



## Development in Asphalt Binder Specifications

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0 pages | null | PAPERBACK

ISBN 978-0-309-43027-2 | DOI 10.17226/22903

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TRANSPORTATION RESEARCH  
**CIRCULAR**

Number E-C147

December 2010

**Development in Asphalt  
Binder Specifications**

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# **Development in Asphalt Binder Specifications**

*Sponsored by*  
Characteristics of Bituminous Materials Committee  
Transportation Research Board

December 2010

**Transportation Research Board  
500 Fifth Street, NW  
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## TRANSPORTATION RESEARCH CIRCULAR E-C147

ISSN 0097-8515

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## Foreword

At the 2008 Transportation Research Board Annual Meeting, the Characteristics of Bituminous Materials Committee sponsored a session on the state of the art in asphalt binder specifications. Leading contributors in this field were invited to present their work as well as more general overviews on specification development addressing the various distresses in asphalt pavements. This circular presents papers covering test methods and specifications for high-temperature permanent deformation, fatigue cracking, and low-temperature cracking. The papers, written after the session took place, are based on the presentations.

Development work in all of these areas continues. The high-temperature test method and specification discussed in the first two papers have been adopted as standards by AASHTO. The high-temperature test method has also been adopted as a standard by ASTM International. These new test methods and specifications are more closely related to field performance and thus will provide agencies with improved tools to specify liquid asphalt products that will effectively meet their needs.

—Robert Kluttz  
*MTE Services, Inc.*



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# New High-Temperature Binder Specification Using Multistress Creep and Recovery

JOHN D'ANGELO

*Federal Highway Administration*

In order to assure positive performance of pavements, highway agencies stipulate the use of specific materials for highway pavement construction. The materials are selected on the results of tests run under set conditions. Many of the tests used in the paving industry are only empirical in nature. Basic tests on the empirical properties of the material have been used over the years and general relationships between tests results and pavement performance have been developed. This system has worked relatively well in the past, but as we move into the future new approaches are needed to assure good performance of the highway system.

The volume of traffic and more importantly the traffic loads have been increasing on our highways. The demand on the transportation dollar increases with the demands on the system. Highway engineers as stewards of the system have to use the best tools available to build long-lasting pavements at the most cost-effective prices. The use of old empirical tests cannot effectively evaluate the new materials such as polymer-modified asphalt that is increasingly being used to construct our roads. There is a tremendous need to develop new performance-based test procedures. These new tests will provide engineers with the information they need to select the most cost-effective materials to maintain the highway system.

## PROBLEM

AASHTO Superpave<sup>®</sup> specification M-320 is the most widely used asphalt binder specification in the United States (1). This specification was developed based on the Strategic Highway Research Program (SHRP) asphalt research, itself based primarily on the study of unmodified asphalt binders. The applicability of the existing AASHTO M-320 specification to modified binders has been questioned by industry and many of the state highway agencies. The use of polymer-modified asphalt binders has grown tremendously in North America due to the increased stress on the highways from higher traffic volumes and heavier loads. As such, many highway agencies have added additional tests to the AASHTO M-320 specification to ensure that a desired modifier is included in the binder. The problem that arises from the use of the resulting Superpave Plus (a.k.a. SHRP+) tests is that in most cases they do not relate to performance, but only indicate the presence of a particular modifier in the binder. Before an agency uses SHRP+ specifications, the implication of their use should be considered very carefully.

## DEVELOPMENT OF THE MULTISTRESS CREEP AND RECOVERY TEST

The objective of this study conducted by the Federal Highway Administration was the development of a PG binder test that is both performance-based and blind to modification type.

The Multistress Creep and Recovery (MSCR) test was developed as a replacement for the existing AASHTO M-320 high-temperature binder test. The results from the MSCR test may

also be used as an alternative to the various SHRP+ tests. In addition to characterizing fundamental properties, the MSCR is an easy-to-use performance-related test.

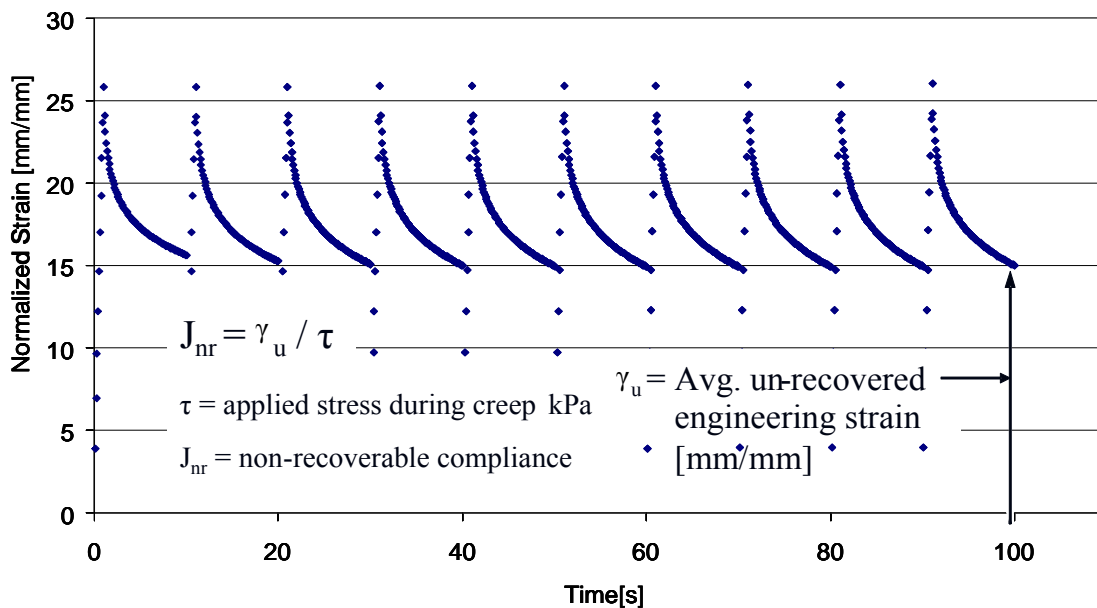
Multiple binders both neat and polymer-modified were evaluated in the development of a new binder test to determine high-temperature rutting property for binders. Equipment for testing of the binders was focused on the existing dynamic shear rheometer (DSR). This equipment has been widely accepted by highway agencies for use in determining rheological properties of binders in specifications. The DSR measures fundamental properties, related to the stress strain response of viscoelastic materials and is ideally suited to evaluate asphalt binders.

The new test uses the 1-s creep loading with the 9-s recovery over the multiple stress levels of 0.025, 0.050, 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, 6.4, 12.8, and 25.6 kPa at 10 cycles for each stress level. The test is started at the lowest stress level and increased to the next stress level at the end of every 10 cycles with no time lags between cycles. The average non-recovered strain for the 10 creep and recovery cycles is then divided by the applied stress for those cycles yielding the nonrecoverable compliance  $J_{nr}$ . The determination of  $J_{nr}$  is shown in Figure 1.

$$J_{nr} = \text{avg. } \gamma_u / \tau,$$

$\gamma_u$  = unrecovered strain from the end of the 9-s recovery portion of the creep and recovery test,  
and

$\tau$  = shear stress applied during the 1-s creep portion of the creep and recovery test.

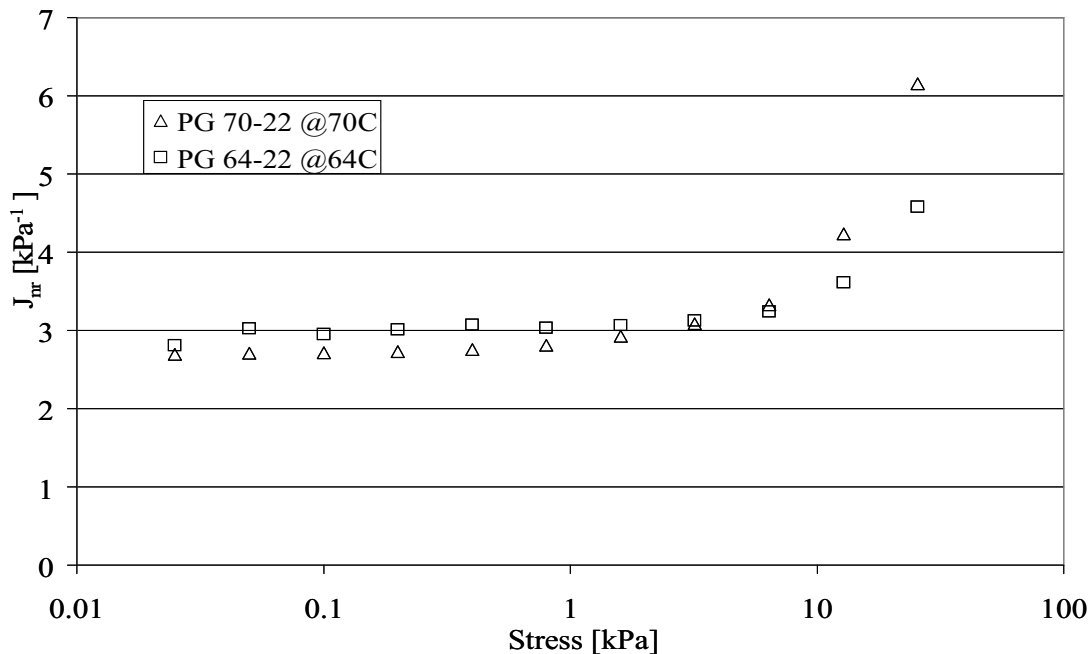


**FIGURE 1** Plot showing determination of  $J_{nr}$  and recovered strain from MSCR test.

Through numerous evaluations the MSCR test has proven to be able to distinguish the difference in rutting potential between various binders, both modified and unmodified (2, 3, 4). The evaluations were performed on multiple types of mix testing from loaded wheel testers and accelerated load testers to actual pavement sections. Stress dependency of the binders was identified which established the criteria to evaluate the different mix tests and their relationship to binder properties. These findings indicate that the nonrecoverable compliance  $J_{nr}$  would be a good replacement for the SHRP high-temperature binder criteria. The next step was to establish specification criteria.

The existing SHRP specification has been extensively validated for neat asphalts. This validation can lead to the assumption that the criteria for neat asphalts in the existing specification do represent their rutting potential. Based on this, the values of  $J_{nr}$ , for neat asphalts tested at their grade temperature, can be compared to the current SHRP grading. The question, that then needs to be addressed is at what stress level should this comparison be made? During this study many neat asphalt binders were tested using the MSCR test procedure. A review of these binders will provide the needed input for the specification development.

The existing SHRP binder specification is based on measurements in the linear viscoelastic range. The  $J_{nr}$  testing of many neat asphalt binders, over a wide range of stress, indicates they behave in a linear fashion typically up to the 3.2 kPa stress level. This behavior can be seen in Figure 2. The figure covers two of the most typical binders used in North America, a PG 64-22 and a PG 70-22. In each case the stress dependency of the binder does not become significant until the 6.4 kPa stress is reached. As the existing high-temperature performance for the SHRP PG grading system for neat asphalts has been well established, the  $J_{nr}$  at 3.2 kPa, which is typically within the linear range for most neat asphalt binders, should provide a reasonable correlation to the performance.



**FIGURE 2** Plot of the nonrecoverable compliance of the ALF PG 70-22 at 70°C and the Mathy PG 64-22 at 64°C .

An evaluation was made of multiple neat asphalt binders to determine if a relationship existed between the existing SHRP grading and the nonrecoverable compliance  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$ . The continuous SHRP binder grade, the temperature where  $|G^*|/\sin \delta$  is  $2.2 \text{ kPa}$ , was determined for each of the neat binders. The corresponding  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  was then determined for each of the neat binders at the SHRP continuous grade. This data is shown in [Table 1](#).

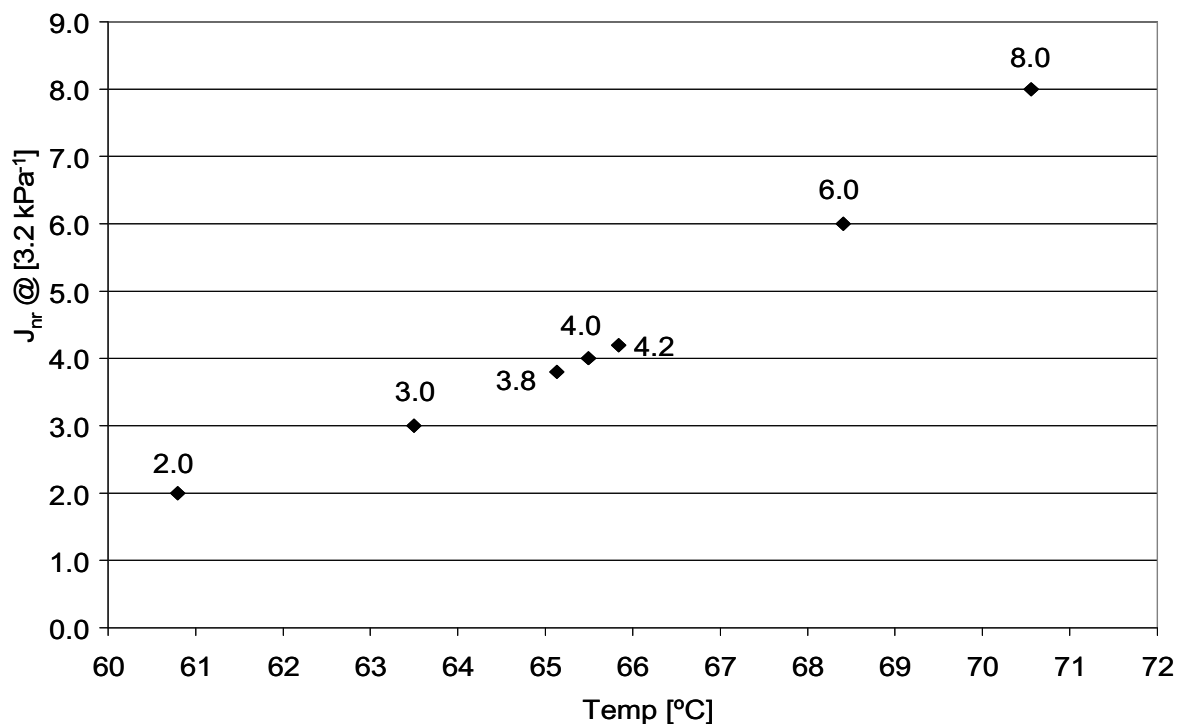
The relationship of  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  to the temperature where  $|G^*|/\sin \delta$  is equal to  $2.2 \text{ kPa}$  provided an average value of  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  of  $4.2 \text{ kPa}^{-1}$ . The coefficient of variation of the  $J_{nr}$  values for all the binders was 12%. This is actually quite good in that the temperature– $J_{nr}$  relationship is a power function as seen in [Figure 3](#). Minor variations in temperature correspond to large variations in compliance values. [Figure 3](#) shows a plot of a neat PG 64-22 over a change in  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  from  $2.0 \text{ kPa}^{-1}$  to  $8.0 \text{ kPa}^{-1}$ , demonstrating the temperature sensitivity of the neat binder. The change in  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  from  $4.2 \text{ kPa}^{-1}$  to  $4.0 \text{ kPa}^{-1}$  would reflect a change in temperature of only  $0.3 \text{ }^\circ\text{C}$ . The change in  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  from  $2.0 \text{ kPa}^{-1}$  to  $4.0 \text{ kPa}^{-1}$  reflects a change in temperature of  $4.7^\circ\text{C}$ , where the change in  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  from  $4.0 \text{ kPa}^{-1}$  to  $8.0 \text{ kPa}^{-1}$  reflects a change in temperature of only  $5.0^\circ\text{C}$ . For this same binder a change in  $|G^*|/\sin \delta$  from  $4.4 \text{ kPa}$  to  $2.2 \text{ kPa}$  produced a change in temperature of  $5.1^\circ\text{C}$  and from  $2.2 \text{ kPa}$  to  $1.1 \text{ kPa}$  produced a change in temperature of  $5.6^\circ\text{C}$ . This again emphasizes the fact that the  $6^\circ\text{C}$  temperature shift used in the SHRP binder specification really does not work even for neat binders.

This analysis indicates a specification with a nonrecoverable compliance  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  of  $4.0 \text{ kPa}^{-1}$  would provide binders with equivalent stiffness to the existing SHRP binder specification for neat binders. A neat PG 64-22 under the existing SHRP specification would be equivalent to a PG 64-22 based on a MSCR specification of  $J_{nr}$  at  $3.2 \text{ kPa}^{-1}$  of  $4.0 \text{ kPa}^{-1}$ . For neat binders the specification grade should not change even though the test and criteria would change.

Though neat binders properties are very similar in a new MSCR specification to the original SHRP binders, polymer-modified binders are likely to have very different properties. Polymer-modified binders were previously shown to be stress dependent. [Figure 4](#) shows a plot

**TABLE 1 Determination of the  $J_{nr}$  Value for Multiple Neat Binders at  $3.2 \text{ kPa}$  and at the Temperature Where  $|G^*|/\sin \delta$  Would Be  $2.2 \text{ kPa}$**

Sample ID	Name	Grade	True Grade	Temp [ $^\circ\text{C}$ ]	$J_{nr}$ [ $3.2 \text{ kPa}^{-1}$ ]
ALF 6727	Control	70-22	72.7-74.2	72.7	4.4
BBRS3	Straight	64-22	66.1-27.3	66.1	4.2
MN county rd 112	Neat Valero	58-28	60.8-33.4	60.8	3.7
MN county rd 112	AshlandM	58-28	60.7-31.4	60.7	4.3
Minn Road	Straight	58-28	61.8-30.8	61.8	3.0
Shandong	Straight	64-22	64.4-23.5	64.4	4.4
BBRS3	Straight	70-22	71.4-24.8	71.4	4.8
BBRS3	Straight	58-28	61.3-30	61.3	4.0
MD project	Straight	64-28	64.8-29.6	64.8	4.6
Citgo	Straight	70-22	71.6-26.9	71.6	4.6
Lion	Straight	64-22	66.7-24.1	66.7	4.5
Average					4.2
Coefficient of variation (%)					12



**FIGURE 3 Plot of the temperature sensitivity of a neat PG64-22 for specific variations in nonrecoverable compliance.**

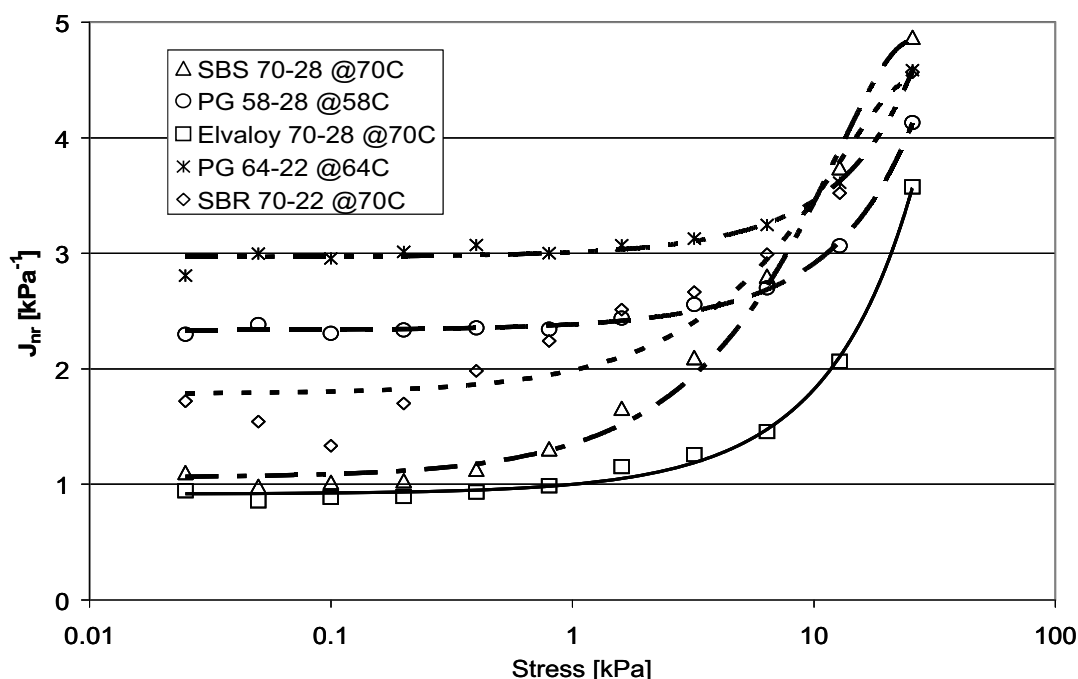
of stress versus nonrecoverable compliance for several different binders at their current SHRP PG grade temperature. The neat binders—the PG 64-22 and the PG 58-28—exhibit linear behavior to beyond the 3.2 kPa stress level. However, the polymer-modified binders, the SBS 70-28 and the SBR 70-22, begin to show nonlinear behavior at much lower stress level of about 0.8 kPa. This is due to the two-phase nature of the polymer-modified binders, with the polymer network suspended in the asphalt continuum. When tested in the lower-stress linear viscoelastic range they appear much stiffer. As the stress increases the polymer chains are extended and start to disentangle. This disentanglement reduces the strength of the binders and modified binders appear to have the same compliance as the neat binders.

Typically polymer-modified binders are used on roadways with high traffic and heavy loads. Under the existing SHRP specifications with higher traffic levels or slower speeds grade bumping is used. In these cases binders with a higher temperature grade would be specified. For example, on a typical freeway in a PG 64 climate if the traffic level were to be above 30 million ESALs one grade bump would be specified, increasing the required grade to a PG 70. At toll facilities or in urban areas where slow moving traffic would be expected the grade would be bumped a second time increasing the required grade to a PG 76. So in the SHRP system the specification criteria would be held constant but the test temperature at which the criteria is required is increased. In this example a high traffic volume urban pavement in a PG 64 climate would require a PG 76 binder that would be tested at 76°C, 12 degrees above the temperature the pavement is likely to ever experience. The MSCR testing would indicate that the binders at these high temperatures are very stress sensitive and not likely to be much stiffer than a neat binder. The other major issue with the SHRP binder specification is that the grade-bumping criteria is based on the concept that time temperature superposition is the same for all binders.

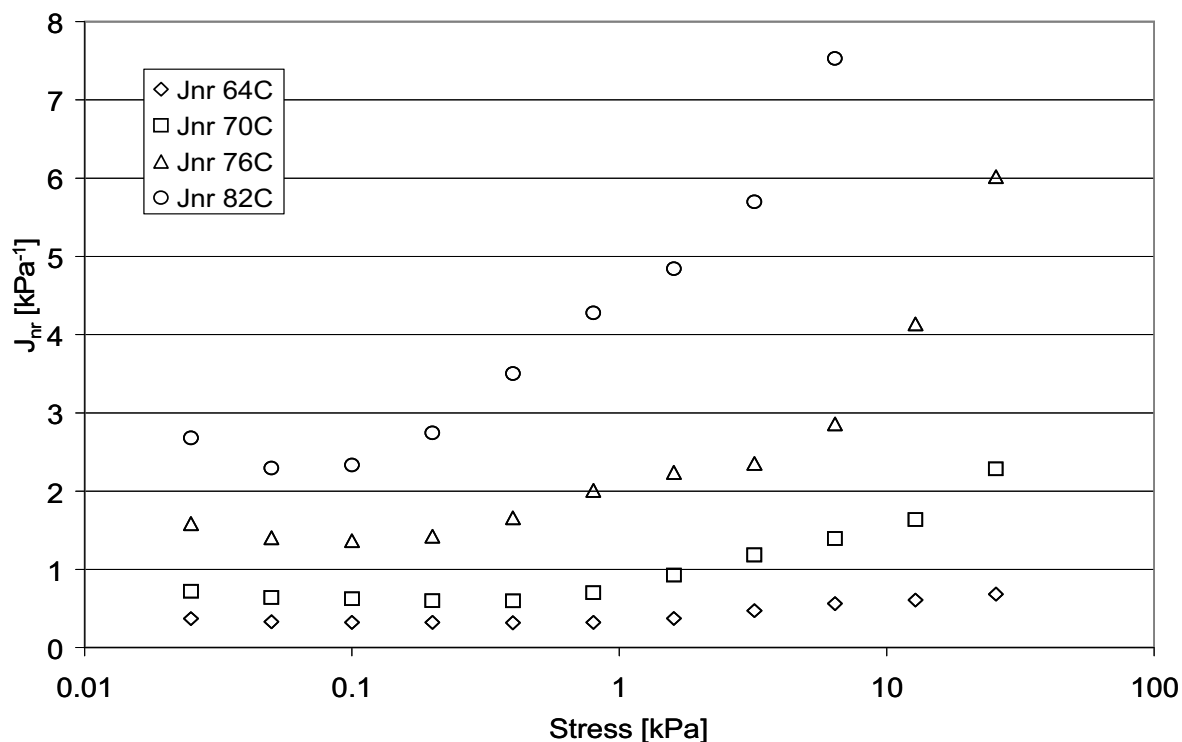
The SHRP grade-bumping concept is actually inappropriate for a high-temperature binder specification. Rutting is a nonlinear high stress and strain phenomenon and testing at temperatures far above the temperature where the stresses and strains will occur will lead to erroneous results that will not correlate to the performance. Figure 5 shows a plot of stress versus nonrecoverable compliance for a typical SBS-modified PG 76-22 over several temperatures. At 76°C the stress sensitivity of the binder is quite pronounced, with the  $J_{nr}$  increasing from 1.36  $\text{kPa}^{-1}$  at 0.1 kPa to 2.35  $\text{kPa}^{-1}$  at 3.2 kPa, a change of 73%. In actuality this binder would most likely be used in a PG 64 climate and never see temperatures of 76°C. At 64°C the  $J_{nr}$  goes from 0.32  $\text{kPa}^{-1}$  at 0.1 kPa to 0.47  $\text{kPa}^{-1}$  at 3.2 kPa, an increase of only 47%. So on an actual roadway this binder would be much less stress sensitive and significantly stiffer than indicated by testing at the higher 76°C. Any new high-temperature binder specification should eliminate the temperature grade bumping and only have testing done at the expected high temperature of the pavement.

## GRADE BUMPING

Grade bumping based on changes in temperature is used in the SHRP binder specification. The concept is based on the idea that increasing the test temperature by 6°C and holding the criteria value the same will basically double the stiffness of the binder. This was developed with the assumption that the temperature susceptibility of all binders is very similar. This has been demonstrated to be inappropriate and will not represent the real high-temperature binder properties at the expected pavement temperature. This leads to the following question: how should increased traffic volume or slower speeds be taken into account in a new specification?



**FIGURE 4** Plot of neat and modified binders showing the relationship between stress and compliance at temperatures where the compliance is between 1.0  $\text{kPa}^{-1}$  and 3.0  $\text{kPa}^{-1}$ .



**FIGURE 5** Plot of stress versus compliance demonstrating the stress dependency for a SBS-modified PG 76-22.

The high pavement temperature does not change with changes in traffic volume so it would be best to change the criteria used to evaluate the binder at that high temperature. Instead of increasing the testing temperature and holding the specification criteria constant, the required compliance value should be reduced to provide a more rut-resistant binder. The original viscosity specifications used a variable viscosity value based on the climate area and the traffic loading. For a new high-temperature binder specification such as the MSCR  $J_{nr}$  value, the existing SHRP pavement temperature criteria should be used, but with a variable  $J_{nr}$  requirement based on traffic volume and speed.

Earlier a relationship was developed between the RTFOT  $|G^*|/\sin \delta$  specification value of 2.2 kPa and the MSCR test  $J_{nr}$  value at 3.2 kPa of 4.0 kPa<sup>-1</sup> at the equivalent temperature. Using a  $J_{nr}$  value at 3.2 kPa of 4.0 kPa<sup>-1</sup> as a basis for a standard paving grade, reductions in  $J_{nr}$  for grade bumping will be made for slow-speed traffic and traffic greater than 30 million ESALs. Using this approach temperature bumping would be eliminated and adjustments to the  $J_{nr}$  criteria would be made. The three mix studies—I-55 in Mississippi, FHWA ALF polymer study, and the MnRoad Hamburg mix study—were used to determine the relationship between  $J_{nr}$  and rutting. Table 2 shows the relationship between  $J_{nr}$  and rutting for the three studies and the change in rutting with changes in  $J_{nr}$ . The ALF mix experiment was an accelerated loading study using an 80 kN wheel load traveling at 19 km/h at a constant 64°C. This is an extreme loading condition far exceeding anything that will typically be experienced on an actual roadway. In this case a reduction in  $J_{nr}$  by 50%, from 4.0 kPa<sup>-1</sup> to 2.0 kPa<sup>-1</sup>, reduced rutting by almost 39%. A further 50% reduction of  $J_{nr}$ , from 2.0 kPa<sup>-1</sup> to 1.0 kPa<sup>-1</sup>, reduced the rutting an additional 32%. The



**TABLE 2 Relationship Between Changes in  $J_{nr}$  and Changes in Rutting for I-55 Mississippi, ALF, and MnRoad Studies**

MS test site			ALF study			MnRoad		
$J_{nr}$ at [3.2 kPa <sup>-1</sup> ]	Rut [mm]	% Change in Rut with Change in $J_{nr}$	$J_{nr}$ at [3.2 kPa <sup>-1</sup> ]	Rut [mm]	% Change in Rut with Change in $J_{nr}$	$J_{nr}$ at [3.2 kPa <sup>-1</sup> ]	Rut [mm]	% Change in Rut with Change in $J_{nr}$
4	13.3		4	27.7		4	10.2	
2	6.4	51.7	2	17.0	38.7	2	5.7	44.2
1	3.0	53.5	1	11.6	31.6	1	3.4	39.7

MnRoad study was performed with the dry Hamburg tester. This testing was done with steel wheel loaded to 703 N done at multiple temperatures. In this case a reduction in  $J_{nr}$  from 4.0 kPa<sup>-1</sup> to 2.0 kPa<sup>-1</sup> reduced the rutting by 44%, and the additional reduction of  $J_{nr}$ , from 2.0 kPa<sup>-1</sup> to 1.0 kPa<sup>-1</sup>, reduced the rutting an additional 40%. The I-55 test section in Mississippi was an actual high-speed, high-volume test site with real traffic distributions and variable annual temperatures. The rutting was measured for 6 years providing a good measure of expected early life field rutting. In this case a reduction of  $J_{nr}$  from, 4.0 kPa<sup>-1</sup> to 2.0 kPa<sup>-1</sup>, reduced rutting by 52% and a reduction  $J_{nr}$ , from 2.0 kPa<sup>-1</sup> to 1.0 kPa<sup>-1</sup>, reduced the rutting an additional 54%. As the loading severity is reduced the percent change in rutting with a change in  $J_{nr}$  increases with the real world sections on I-55 experiencing just over a 50% reduction in rutting. Based on this, a reduction in  $J_{nr}$  of 50% should reduce rutting by approximately 50%.

The standard RTFOT environmental grade in the existing SHRP binder specification,  $|G^*|/\sin\delta$  of 2.2 kPa, has been shown to be equivalent to an MSCR  $J_{nr}$  value of 4.0 kPa<sup>-1</sup> measured at 3.2 kPa. A new high-temperature binder specification would start with PG standard grade which would require a  $J_{nr} = 4.0$  at 3.2 kPa<sup>-1</sup> or less, which would be referenced as a PG 64S-XX. Grade bumping for slow traffic would require reducing the  $J_{nr}$  value by 50% to 2.0 or less, which would be referenced as a PG 64H-XX. Grade bumping for standing traffic would require reducing the  $J_{nr}$  value by 50% again to 1.0 or less, which would be referenced as a PG 64V-XX. In each case the binder would be tested at recommended environmental PG high temperature to evaluate how the binder would react in the pavement not at some artificially high temperature which the pavement will never experience. An example of what the new specification would look like is shown in [Table 3](#).

## SHRP+ REPLACEMENT TEST

The MSCR  $J_{nr}$  value has been shown to be a better indicator of the rut resistance of both neat and polymer-modified binders than the existing SHRP criteria  $|G^*|/\sin\delta$ . However, many highway agencies require polymers in binders for other reasons than rutting. In many cases polymers are required to reduce cracking or to improve durability by reducing ravelling. So even though  $J_{nr}$  is a better indicator of rut resistance of the binder, it alone will not identify the presence of an elastomeric polymer in the binder and cannot be directly used as a SHRP+ to replace the most commonly used Elastic Recovery (ER) test. The percent recovery measured in the MSCR test does measure the elastomeric response of the polymer in an asphalt binder.

**TABLE 3 Recommended New High-Temperature Specification Using the MSCR  $J_{nr}$  Value to Replace the RTFOT  $|G^*|/\sin\delta$**

Original					
DSR $ G^* /\sin\delta$ min 1.0	64				
RTFOT					
64S standard $J_{nr}$ $3.2 \leq 4.0$ tested at environmental temp.	64				
64H heavy $J_{nr}$ $3.2 \leq 2.0$ tested at environmental temp.	64				
64V very heavy $J_{nr}$ $3.2 \leq 1.0$ tested at environmental temp.	64				
PAV					
S grade DSR $ G^* /\sin\delta$ Max 5,000	28	25	22	19	16
H & V grade DSR $ G^* /\sin\delta$ Max 6,000	28	25	22	19	16

The percent recovery measured in the MSCR test directly measures the elastic response of the binder. In the test a creep stress is applied to the binder for 1 s and then removed. The binder is then allowed to recover for 9 s. The amount of recovery has been shown to relate to the amount of elastomeric polymer in the mix and the extent of networking of the polymer in the binder. The difference in the ability of the ER test and the MSCR percent recovery to distinguish the elastic response of polymer-modified binders is further shown in [Table 4](#).

## ELASTIC RESPONSE OF BINDERS

Highway agencies have instituted various SHRP+ tests to assure themselves that there will be an elastomeric polymer in the binder. The MSCR test provides a significant improvement over all of these previously used SHRP+ tests. The MSCR test provides a far greater control over loading and measurement of response than the tests typically used. The recovery portion of the creep and recovery curve in the high-temperature range used to evaluate pavement rutting provides significant information on how a polymer is reacting, in the base binder and how well established the polymer network is.

Neat binders exhibit very little elastic response at typical high pavement temperatures. At the end of the creep loading very little of the accumulated strain is recovered. Polymer-modified binders, however, in many cases exhibit varying amounts of recovery of the accumulated strain after the creep load is removed. The recovery as measured in the MSCR test can be used to determine the structure that has been established in the binder by the polymer. The ER test, extensively used by highway agencies, just provides an indication that an elastomeric polymer is present in the binder.

The ability of the MSCR test to identify the extent and strength of the polymer network in a binder was demonstrated through a comparison of different blending techniques in one base binder. A PG 64-22, produced from a Saudi Light crude oil by Lion Asphalt, was blended with

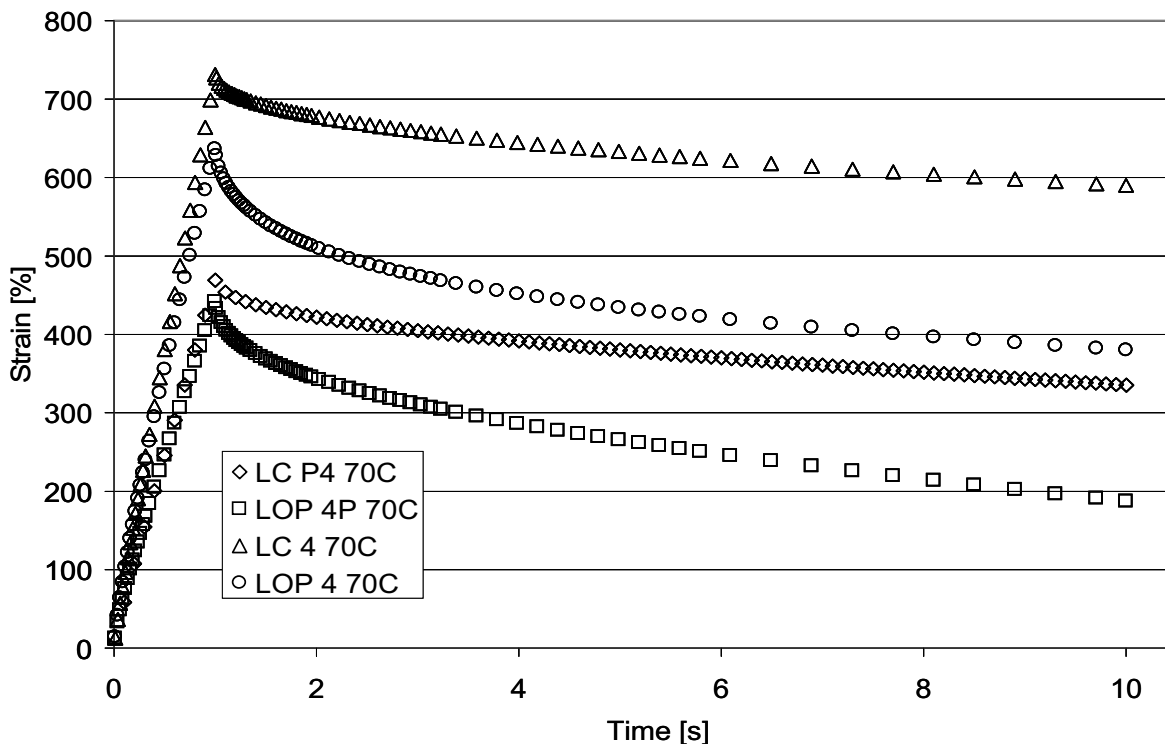
one polymer content, but using different methods and slight variations in polymer type. The Lion asphalt is considered a compatible asphalt in that the polymer mixes easily and will take significant time to separate from the base. The polymer used was a Kraton 1101 SBS linear polymer and a Kraton 1184 SBS radial polymer both added at 4% by weight to the base asphalt. The LC 4 formulation was created with the Kraton 1101 mixed directly with the Lion asphalt at 188°C using a high-speed Silver son mixer. The LC P4 was produced in the same manner except 0.5 % polyphosphoric acid (PPA) was also blended in. Mixing was continued for 2 hours for each binder. The LOP 4 and LOP 4P binders were produced by diluting a concentrate of 15% Kraton 1184 polymer with the Lion asphalt to a final 4% by weight blend. In this case the concentrate was made by adding the polymer into the Lion base at 200°C with a high-speed mixer. The LOP 4P had the additional 0.5% PPA added. Table 4 provides the properties of the various blends.

Each one of the blends was produced from the same base asphalt and had 4% SBS polymer added, however, MSCR testing indicates that the binders are very different. The SHRP binder testing graded the LC 4 and the LOP 4 to have almost exactly the same high-temperature grade with only 0.9 degrees Celsius difference between their continuous grades. This was the same for the LC 4P and LOP 4P where the addition of PPA increased the high-temperature grade to 81°C. The MSCR testing, however, graded the binders differently, indicating the differences associated with the blending differences.

The MSCR  $J_{nr}$  value at 3.2 kPa indicates the LC 4 binder is the weakest. The LC 4P and LOP 4 binders are almost the same with the LOP 4P 37% less compliant than the other three. These results are very different than the SHRP binder grading where the LC 4 and the LOP 4 are equivalent and the LC 4P and the LOP 4P are equivalent. In addition to the differences identified by the  $J_{nr}$  results, very distinct differences of the polymer network in the binder can be seen by evaluating the percent recovery between the peak strain and the final unrecovered strain. These differences are shown clearly in Figure 6.

**TABLE 4 Data on Polymer Modified Binders Indicating Differences Based on Blending Methods**

Sample ID	Continuous Grade	Polymer	Acid	Temp (°C)	$J_{nr}$ 3.2 kPa	Percent Recovery 3.2 kPa	Elastic Recovery
LC	66.7–24.1		0	64	3.1	0	5
LC 4	75.7–22.3	4% linear SBS	0	70	1.9	19.2	73.8
				76	4.6	6.0	
LC 4P	81.2–22.2	4% linear SBS	0.50%	70	1.1	28.4	93.8
				76	2.4	20.6	
LOP 4	76.6–25.2	4% radial SBS from concentrate	0	70	1.2	40.3	86
				76	2.4	37.0	
LOP 4P	81.6–24.5	4% radial SBS from concentrate	0.50%	70	0.7	52.1	91.6
				76	1.4	42.5	



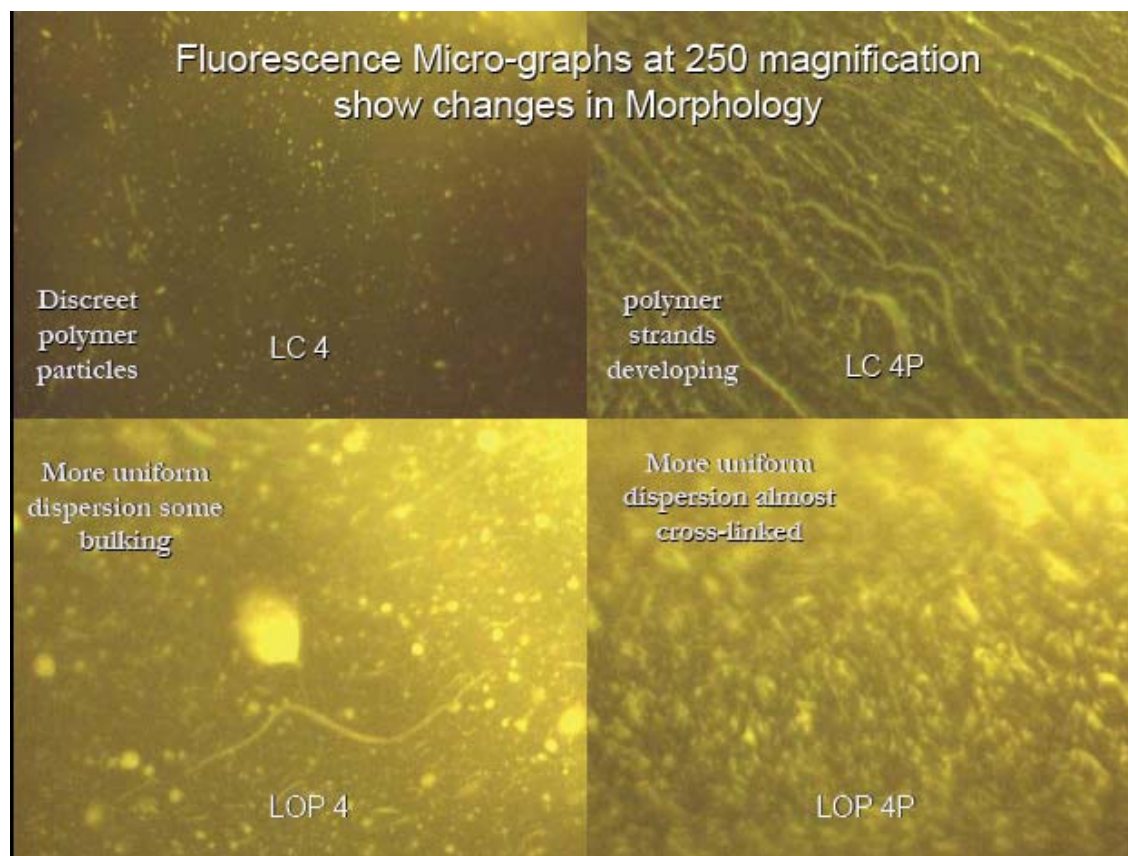
**FIGURE 6** Plot of the creep and recovery cycle for 4% polymer blends of Lion asphalt tested at 70°C.

The LC 4 and LC P4 binders where the polymer was blended in at lower temperatures have much lower percent recovery in the MSCR test than the LOP 4 and LOP 4P binders. The recovery for the LC 4 is 19.2% and the LC P4 is 28.4% at 70°C. The improved processing used to make the LOP 4 and LOP 4P binders increased the recovery to 40.3% and 52.1%, indicating a superior polymer network in the binder. The elastic recovery testing using AASHTO T301 indicated the binders should have similar ranking to the SHRP binder testing. The ER test is used extensively to evaluate the presence of polymer in binders, but does not seem to distinguish between polymer systems. The ER test ranks the LC 4 as the weakest binder with the LC 4P as the strongest.

The improved recovery results shown in the MSCR tests are attributed to the improved polymer network established in the binder by processing at higher temperatures. To verify that the MSCR percent recovery test results actually do reflect the extent of the polymer network in the binders the morphology of the polymer network was evaluated. A Van Guard 1200 ECM microscope with an ultraviolet light source was used to create fluorescence micrographs of the binders shown in Figure 7. A small sample of asphalt binder is placed on a glass plate and viewed through a microscope under ultraviolet light. The maltene fraction of the binder is absorbed into the SBS polymer and reflects the ultraviolet light, indicating the polymer structure (5, 6, 7). The micrograph of LC 4 indicates that the polymer is simply floating in the asphalt. This is identified by the small glowing dots sometimes called the night sky. The LC 4P micrograph shows how the PPA has caused the polymer to start to cross-link, as seen by the long polymer strands in the asphalt. The micrograph of the LOP 4 begins to show the complete networking of the polymer in the asphalt. The LOP 4 shows continuous polymer network with

some larger concentrations of polymer. The LOP 4P micrograph again shows much more extensive polymer network with a leathery look indicating extensive cross-linking and well dispersed concentrations of polymer. This verifies results from the percent recovery measured in the MSCR test.

The AASHTO T301 ER test, the typical test used by many highway agencies to verify the presence of polymer in the asphalt binder did not identify the nature of the polymer structure. The AASHTO test T301 can not identify the nature of the polymer network because the test is performed at 25°C, a temperature where the base binder is significantly stiff enough to provide support for the polymer, thus masking the nature and extent of the polymer network. All four binders had high ER test results well above typical highway agencies requirements. The ER test indicates the LC 4 and LOP 4 should be similar and the LC 4P and LOP 4P should be similar. The MSCR test percent recovery at 76°C indicates the LC 4 and LC 4P are reacting like a neat asphalt where the polymer is only a filler which has stiffened the asphalt to a 76 grade. The percent recovery for the LC 4 and LC4P was only 6% and 20.6%, respectively, while the LOP 4 and LOP 4P had 37% and 42.5% recovery at the higher temperature. These higher recoveries indicate the improved nature of the polymer network in the binder, which is not shown by the ER test.



**FIGURE 7 Fluorescence micrographs of the Lion asphalt with different polymer processing.**

## CONCLUSIONS

To capture the rutting response of the binder in the mix the nonlinear response has to be captured. A new test, Multistress Creep and Recovery, performed with a DSR was developed to capture the nonlinear response of the binder and relate that response to rutting in asphalt mixtures. This affect can not be identified by the existing SHRP binder specification where testing is done in the linear viscoelastic range.

A new high-temperature binder specification was developed using the compliance value  $J_{nr}$  measured in the MSCR test. This new specification does a far better job of relating binder properties for both neat and polymer-modified binders to actual pavement rutting. Testing is done at the same environmental zones established by SHRP but temperature bumping is eliminated. To account for high traffic volumes and or slower moving traffic the compliance value is reduced. For any climate zone all testing would be done at a constant temperature.

The stress dependency of polymer-modified binders is far more complex than neat binders. Polymer-modified binders are in fact two-phase systems and the stress dependency is affected by the stiffness of the base asphalt, the volume of polymer, and the extent of the polymer network in the binder. Non-cross-linked polymer will act like a filler and increase the overall stiffness of the binder, however the stress sensitivity of the binder will be greater than a neat binder with an equivalent stiffness. This is due to the actual lower stiffness of the base binder controlling the polymer particle interaction at higher stress. As the polymer percentage or volume is increased the particle interaction becomes greater, increasing the apparent stiffness of the binder and reducing the stress sensitivity at any particular stiffness. Cross-linking of the polymer increases the stiffening effect of any volume percentage of the polymer and reduces the stress sensitivity of the binder. All of these effects are identified by the MSCR test which improves the ability to relate binder properties of both neat and modified binders to mixture performance.

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## Use of Hamburg Rut Testing Data to Validate the Use of $J_{nr}$ as a Performance Parameter for High-Temperature Permanent Deformation

GERALD REINKE  
*MTE Services, Inc.*

The development and calculation of the nonrecovered compliance test method and data reduction technique has been described elsewhere in this circular. The effort in this discussion is to enumerate the tests MTE performed and the results obtained, and show how these results lead directly to the utilization of nonrecovered compliance as more rational of high-temperature mixture performance than  $G^*/\sin(\delta)$ .

NOTE: In the original developmental work of  $J_{nr}$  by D'Angelo,  $J_{nr}$  was calculated by dividing nonrecovered percent strain by test stress in Pa (e.g., 3,200 Pa). This method of determining  $J_{nr}$  produced  $J_{nr}$  values ranging from high values of 0.4 and higher to low values of 0.1 and lower. At the time of this TRB session, the procedure for determining  $J_{nr}$  had been changed to dividing actual strain (not percent strain) by the test stress in kPa (e.g., 3.2 kPa). This method of calculation produced  $J_{nr}$  values ranging from high values of 4.0 and higher to low values of 1.0 and lower. The work reported in this paper is derived from data obtained and reduced during the time period when  $J_{nr}$  was expressed as decimal values. For that reason when reading the paper and examining the plots bear in mind that the  $J_{nr}$  values reported herein should be increased by a factor of 10 to be consistent with the expression of  $J_{nr}$  values in 2009 and following.

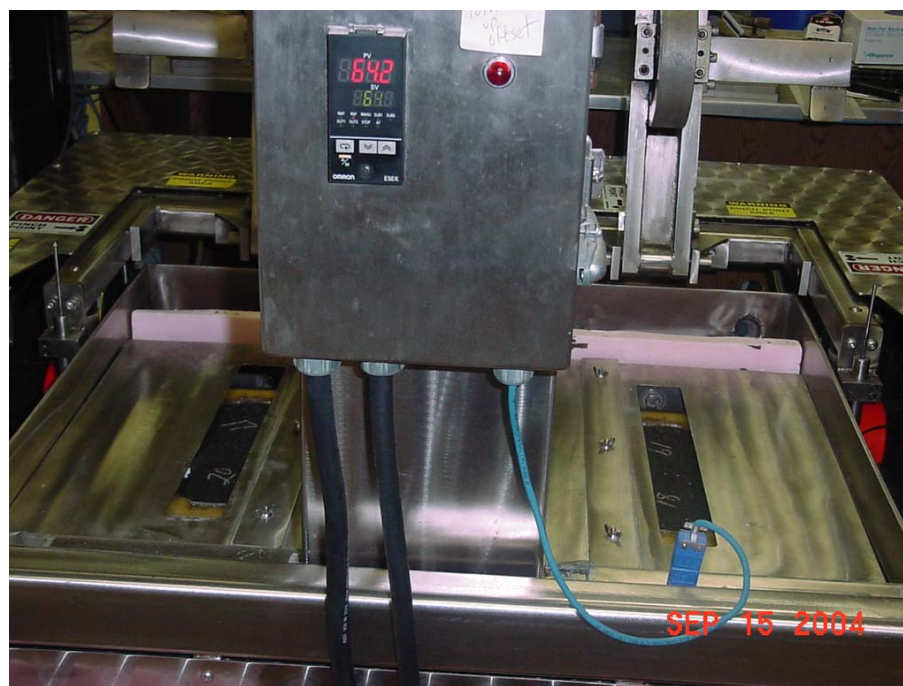
As part of the verification process for the establishment of nonrecovered compliance ( $J_{nr}$ ) as a reasonable parameter for describing the high-temperature performance of bituminous mixtures, MTE used results obtained from its series of Binder Rheology and Rutting Studies (designated in this report as BRRS). Those studies have been ongoing since 2004 and have been reported at ETG meetings and in several publications. In general these studies have sought to identify binder rheological characteristics that correlated to mixture-rutting behavior as determined in the laboratory using an air temperature-controlled Hamburg-type rut tester. Figures 1 and 2 show the arrangement of the test specimens in the rut tester. Two 61-mm thick gyratory pills are cut so that they can be butted against each other to form a single test specimen (Figure 1). Two such combined specimens are tested in the rut tester and the results from each test wheel are averaged to provide an average rut depth value (Figure 2). Details on how the data is reduced are provided in (1).

The experimental work performed is outlined below. Bear in mind that the Hamburg rut test results were the variable against which the  $J_{nr}$  results were correlated.

1. Hamburg rut tests for each mix were conducted at three or four temperatures covering the typical high-temperature PG grade ranges that most agencies specify. Test temperatures ranged from 52.7°C to 74.9°C.



**FIGURE 1** Two gyratory pills, 61 mm in height, at a target air voids of  $7\% \pm 0.5\%$ , are arranged to produce a single test specimen.



**FIGURE 2** Specimens are tested in duplicate to produce a single test result.



2. The test load used was 158 lbs (703-N) unless otherwise noted and the wheel rate was 52 passes per minute.

3. Rut depth in mm at 10,000 wheel passes was used for the data analysis. At warm test temperatures for a given PG binder grade 10,000 wheel passes were generally the maximum number that could be obtained. It was necessary to utilize a metric for all mixes for which a response could be measured.

4. All binder rheological results were obtained at four typical PG grade temperatures. For lower stiffness binders these temperatures were 52°C to 70°C and for stiffer binders these temperatures were 58°C to 76°C. From these test results the binder rheological results were interpolated at the temperature at which the rutting test was conducted. Thus binder properties could be directly correlated to rutting results at specific temperatures.

5. Nonrecovered compliance ( $J_{nr}$ ) values were determined based on the Multistress Creep Recovery test developed by John D'Angelo and described in the first paper of this Circular.

## MATERIALS EVALUATED

The series of binders evaluated for this report are shown in [Tables 1, 2](#) and [3](#) below. The high temperature PG grade of all binders is based on  $G^*/\sin(\delta)$ . Unless otherwise noted all mixes tested were produced with limestone aggregate and based on a 1 million ESAL (Wisconsin DOT E-1) mix design. Unless otherwise noted all rut testing was performed at 703-N (158-lb) test load.

**TABLE 1 BRRS-2 Binders (first series)**

Binder Designation	Modification	PG Grade
PG 58-28	None	PG 59.6-29.8
PG 64-28 SR (straight run)	None	PG 64.7-27.4
PG 64-28C/AS (anti-strip)	PPA+Phosphate Ester antistrip	PG 68.5-31.2
PG 64-28P	Elvaloy + PPA	PG 65.4-30
PG 70-28_3135	Elvaloy (2 types) No PPA	69.8-28.9
PG 70-28P	Elvaloy + PPA	73.2-30.1
PG 76-28P	Elvaloy + PPA	76.8-30.7

**TABLE 2 BRRS-2 Binders (second series)**

Binder Designation	Modification	PG Grade
PG 70-28K	Koch Stylink (SB polymer)	PG 71.6-30.9
PG 58-34O (oxidized)	Oxidized	PG 59.0-35.0
PG 64-34OP (oxidized+polymer)	Oxidized + Elvaloy + PPA	PG 72.4-35.8
PG 58-34P	Elvaloy + PPA	PG 59.4-35.4
PG 64-28C/NO anti-strip	PPA only	PG 64.7-30.9

**TABLE 3 BRRS-3 Binders**

Binder	Modification	True PG Grade
PG 58-28	None	60.4-29.8
PG 64-22	None	66.1-23.5
PG 70-22	None	71.5-25.0
PG 70-22 SBR	SBR latex	70.9-25.7
PG 70-28 SBS	SBS with sulfur cross linking	72.2-30.3
PG 70-28 Elvaloy	Elvaloy + PPA	72.3-30.1
PG 70-28 SBS/EB	SBS with sulfur cross linking + EntiraBond 12	71.5-29.2
PG 76-28 Elvaloy	Elvaloy + PPA	77.2-30.8
PG 76-22 Sealoflex	Sealoflex + PPA	81.9-27.5

## RESULTS

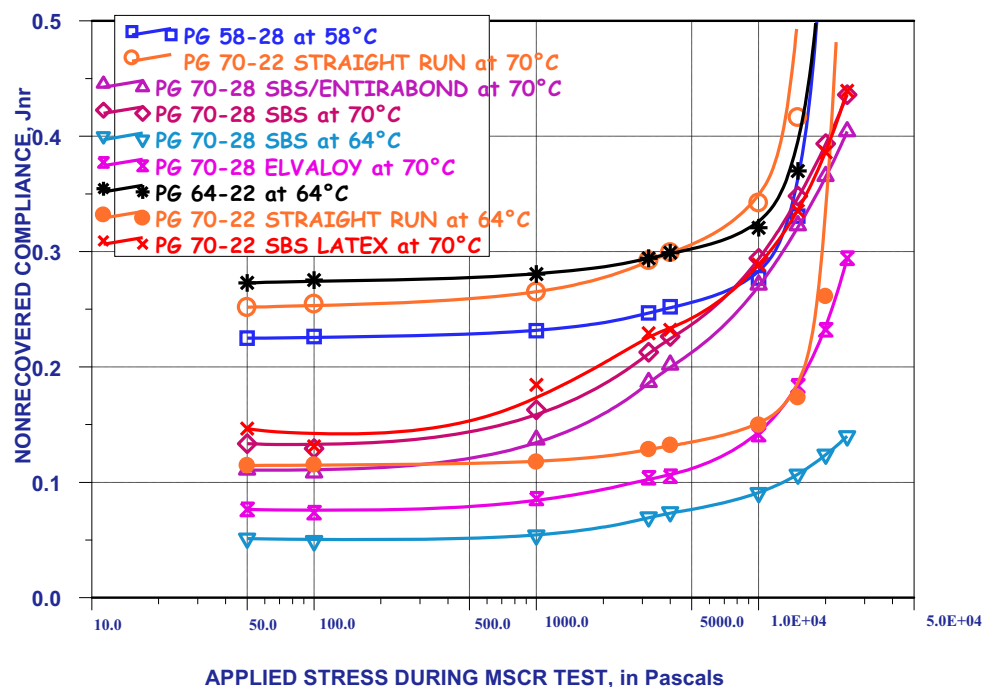
The reason binder nonrecovered compliance is an effective predictor of mixture performance lies in the fact that all binders exhibit a sensitivity to increasing levels of shear stress. This sensitivity to applied stress varies with test temperature, asphalt source, asphalt grade, and type and amount of polymer or other modifying additive. The higher the test temperature the more sensitive to stress is any binder. As with most asphalt characteristics stress sensitivity is asphalt dependent. This source dependency generally requires testing to identify the stress dependency, although in general asphalt with higher levels of asphaltenes will perform better. Most significantly, different polymer types or other types of modifying processes such as PPA or oxidation will impart different levels of stress sensitivity to the binders which they modify.

Figure 3 provides data supporting these claims. While the plot is somewhat busy, the information will repay close reading. Most of the binders were tested at their PG grade temperature and a couple at 6°C below the PG grade temperature. Figure 3 shows a stress sensitivity plot of PG 70-22 straight run asphalt at 70°C (orange open circles) and 64°C (solid orange circles). Two factors of importance are shown: one is the variation of  $J_{nr}$  with temperature. At the 3,200 Pa stress levels  $J_{nr}$  at 70°C is  $\cong 0.3$  while at 64°C the  $J_{nr}$  value is  $\cong 0.15$ . The other factor of importance is the stress at which a rapid increase in  $J_{nr}$  begins to occur. At 70°C this rapid increase begins about 5,000 Pa, while at 64°C the rapid increase begins about 10,000 to 15,000 Pa. This means that mixes made with the PG 70-22 straight run binder can tolerate higher levels of applied stress at 64°C than at 70°C. While this observation may seem like an obvious conclusion to those versed in mix performance, these results make clear why mixes exhibit these characteristics based on the binder grade and mix test temperature. Clearly, aggregate type and gradation will impact permanent deformation behavior, but the observations regarding the impact of binder stress sensitivity remain true for any given binder when combined with any given aggregate.

Further information from Figure 3 can be gleaned by examining the stress sensitivity of the Elvaloy-modified PG 70-28 at 70°C (dark red hour glass), the SBS-modified 70-28 at 70°C (red open diamonds) and the the SBS/Entirabond-modified 70-28 at 70°C (purple upward pointing triangles). While these binders are all PG 70 grades based on  $G^*/\sin(\delta)$ , the  $J_{nr}$  values at 3200 Pa are 0.11, 0.22, and 0.18 respectively. At 15,000 Pa the  $J_{nr}$  values are 0.18, 0.35 and 0.32

respectively. While all three binders exhibit increases in  $J_{nr}$  with increasing stress, these data serve to show that different polymer systems respond differently to the applied stress. It is also worth noting that the SBS and the SBS/Entirabond blends begin to show rapid increase in  $J_{nr}$  above the 1,000 Pa stress level, whereas the Elvaloy blend does not begin to exhibit this rapid increase until 10,000 Pa and higher. This data comparison serves to show that just knowing the PG grade based on  $G^*/\sin(\delta)$  cannot provide information on how the binder will behave in retarding mixture permanent deformation.

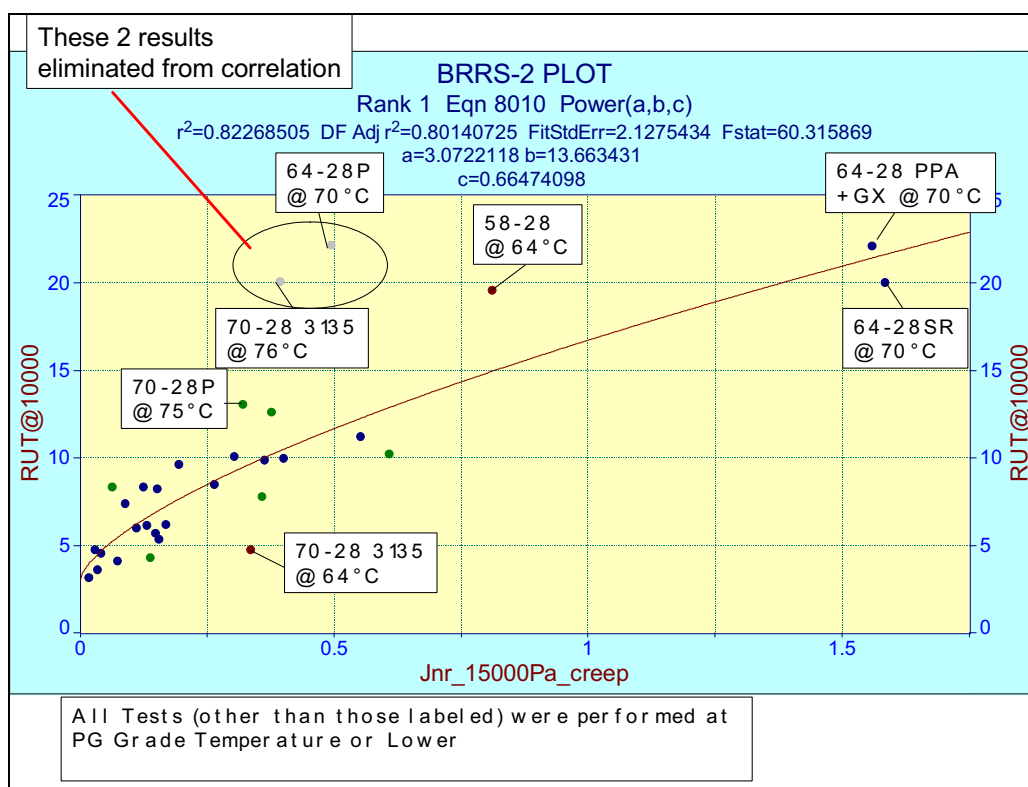
An examination of the stress sensitivity plots for the the PG 58-28 at 58°C (blue open squares) and the PG 64-22 at 64°C (black astericks) in Figure 3 shows that both of these unmodified binders don't begin to exhibit rapid increase in  $J_{nr}$  until a stress level greater than 10,000 Pa. Bearing in mind that the PG 70-28 binders were produced from the PG 58-28 base and the PG 70-22 SBR latex binder (red Xs) was produced from the PG 64-22 base, it should be clear that polymer modification alone is not necessarily sufficient to produce binders that will perform under increased temperature and stress environments. Although all of the PG 70-28 grades and the PG 70-22 grade have  $J_{nr}$  values approximately 50% of the value the original base asphalt at low stress levels when tested at their respective PG grade temperatures, by the time a stress level of 3200 Pa is reached the  $J_{nr}$  of the modified binders (except for the Elvaloy binder) is about 75% of the value of the original base asphalt. By the time the stress level reaches a level of 15,000 Pa the  $J_{nr}$  values in nearly all of the modified binders is the same as those of the base asphalts. While it is true that these materials are being tested at different temperatures, the message is that one should not expect these PG 70-28s or this particular PG 70-22 to perform any better at 70°C in a heavily loaded traffic environment than one would expect a PG 58-28 to



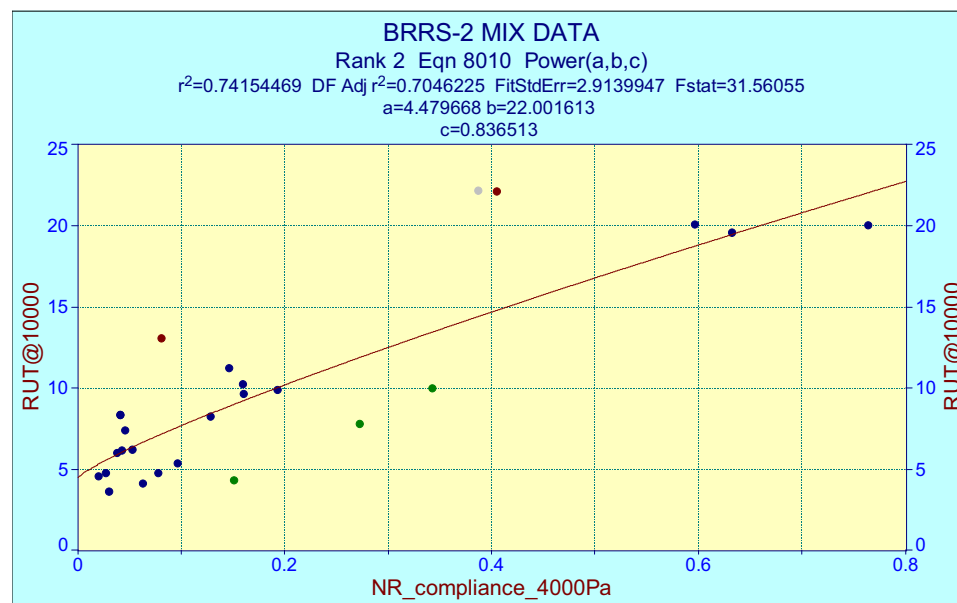
**FIGURE 3** Plot of stress sensitivity of  $J_{nr}$  for selected binders.

perform at 58°C or a PG 64-22 to perform at 64°C in a similar heavy traffic situation. These unmodified binders would never be expected to perform in such situations, but PG 70 grades are used in such situations at warm temperatures just because they are polymer modified. The utilization of nonrecovered compliance now provides a tool for ascertaining which polymer-modified materials can be expected to perform in extreme temperature and traffic environments.

The correlation between nonrecovered compliance and rutting results are examined in Figures 4 through 8. Figure 4 is a plot of mixture rutting using the BRRS-2 binders as a function of nonrecovered compliance at 15,000 Pa. The correlation with two data points removed as shown is 0.82, but the important point to note is that the mixes with high rutting values were obtained when the rutting test was performed at a temperature at least 6°C above the PG grade of the binder used to produce the mix. Given the discussion of the impact of temperature on the stress sensitivity of binders, it stands to reason that asking a binder to perform at an unrealistically high temperature can only lead to excess rutting. Figure 5 is a plot of the same rutting data correlated against nonrecovered compliance determined at 4,000 Pa. While the  $R^2$  is still acceptable at 0.74, this plot in conjunction with Figure 4 makes the point that if the mix is to be exposed to high stress levels in the field then it would be best to determine the  $J_{nr}$  value at a high stress value. Figure 6 is a plot of the 1 million ESAL mix tested in the BRRS-3 study. The  $R^2$  value for this data is 0.81. Although not labeled, the binders with the four highest  $J_{nr}$  values were tested at 6°C above the PG grade of the binder used in their respective mixtures. The most interesting data in this figure however are the comparisons of rutting behavior between the PG 64-22 mix at 64.6°C and the 70-22 SR (straight run) mix at 70.1°C and the PG 58-28 mix at 58.4°C and the PG 70-28 SBS at 70.2°C. An examination of Figure 4 shows that the PG 64-22



**FIGURE 4** Plot of rut depth as a function of  $J_{nr}$  determined at 15,000 Pa.



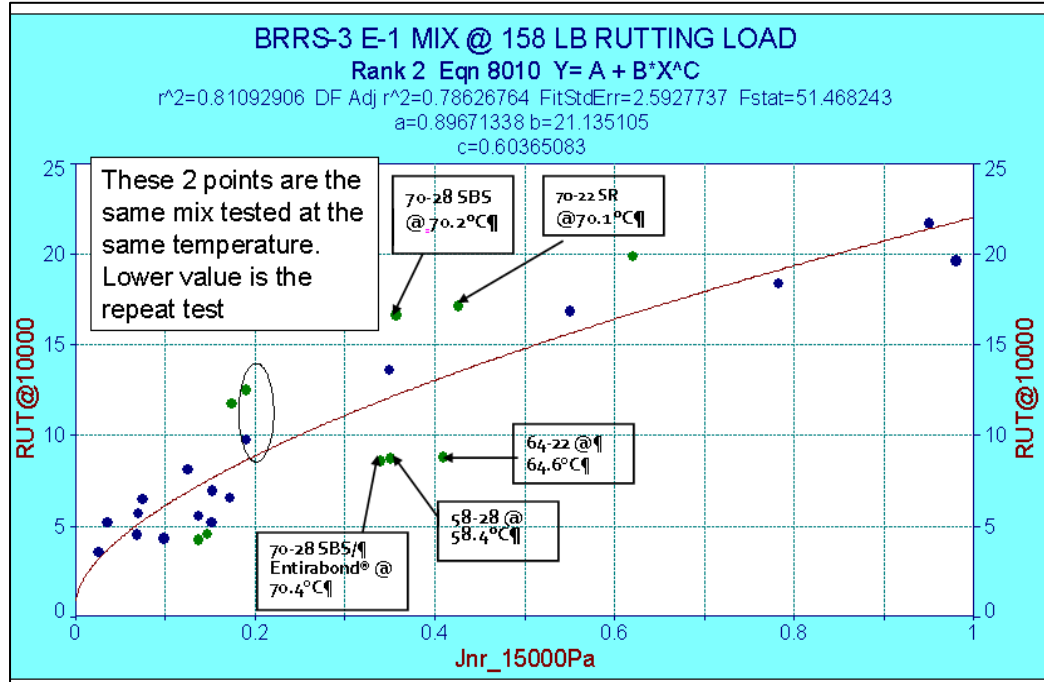
Correlation of  $J_{nr}$  to rutting is better when higher stress is used to determine  $J_{nr}$

**FIGURE 5** Plot of rut depth as a function of  $J_{nr}$  determined at 4,000 Pa.

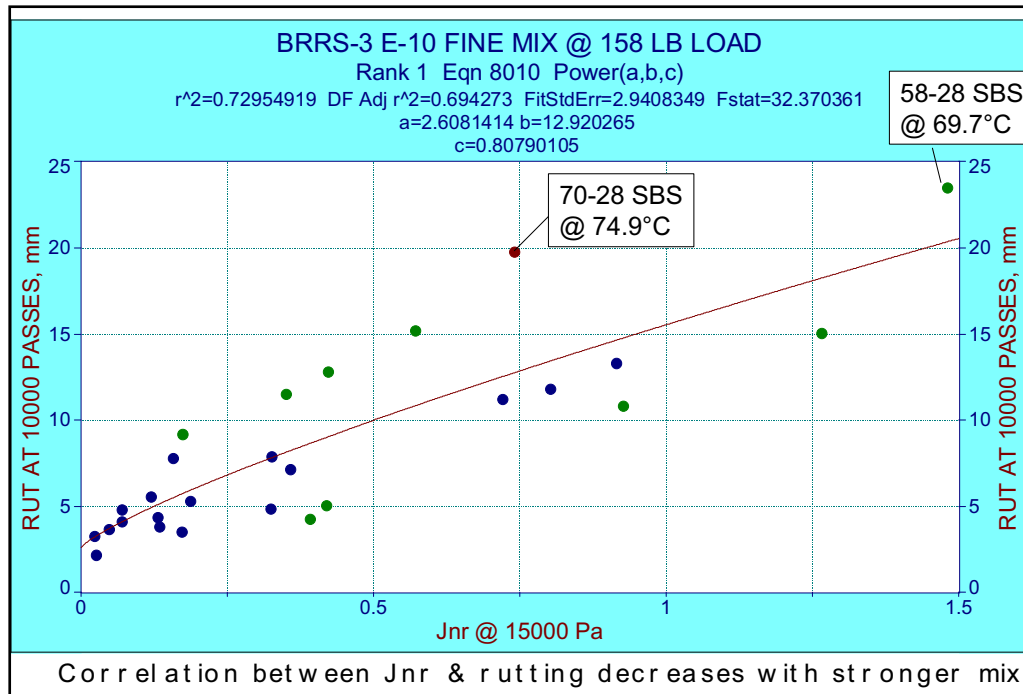
and PG 58-28 are less stress sensitive than the 70-22 SR and the 70-28 SBS at their respective PG grade temperatures. The  $J_{nr}$  values are similar for these two comparisons, but the overall sensitivity of the binders to increasing levels of stress shows that the binders which have rutted to a greater extent are more stress sensitive overall. Figures 7 and 8 are plots of the mixture rutting using the BRRS-3 binders with the 10 million ESAL mix design for fine (Figure 7) and coarse (Figure 8) aggregate blends. Figure 7 shows once again that if a binder is used in a mix well above its grade temperature excess rutting will result. It is also worth noting that in a comparison of Figures 6, 7, and 8, the predicted rut depth for a  $J_{nr}$  value of 0.4 decreases from 13 mm using a E-1 fine aggregate to 9 mm using a E-10 fine aggregate and to 5 mm using a E-10 coarse aggregate. An examination of Figure 8 shows that the rut depth versus  $J_{nr}$  plot is much flatter for the E-10 coarse than for the E-1 fine mixtures.

This leads to the conclusion that as the mix structure improves the impact of the binder properties lessens with respect to rutting behavior.

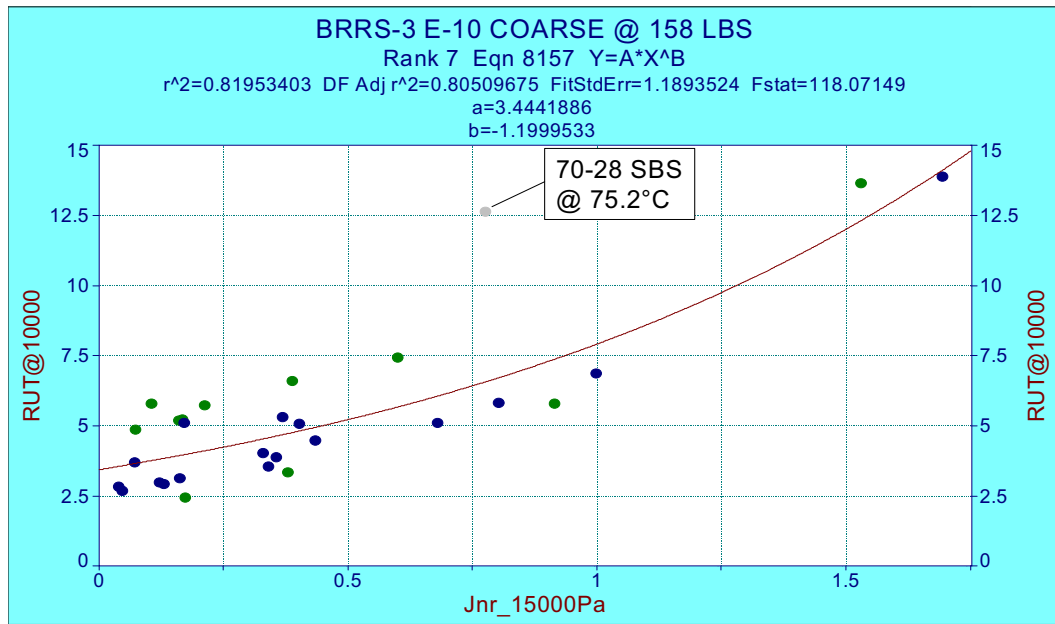
The relative impact of determining nonrecovered compliance at 15,000 Pa versus 4,000 Pa is shown in Figure 9. As should be expected when tested at 15,000 Pa the  $J_{nr}$  values are higher than those determined at 4,000 Pa. However the deviation from the line of equality does not become excessive until the comparison is above a  $J_{nr}$  value above 0.3 at 4,000 Pa. When  $J_{nr}$  is 0.3 at 4000 Pa it is 0.42 at 15,000 Pa. If a binder is being evaluated for performance in a high traffic load environment then a  $J_{nr}$  value of less than 0.2 should be the goal and then it does not matter much as to which stress level is used to make the determination. However, if there is some thought of using a questionable binder in a high traffic load environment then a conservative approach would be to test that binder at 15,000 Pa and aim for a  $J_{nr}$  value of less than 0.4. If only standard or low traffic loads are anticipated then a stress level of 4,000 Pa should perform well.



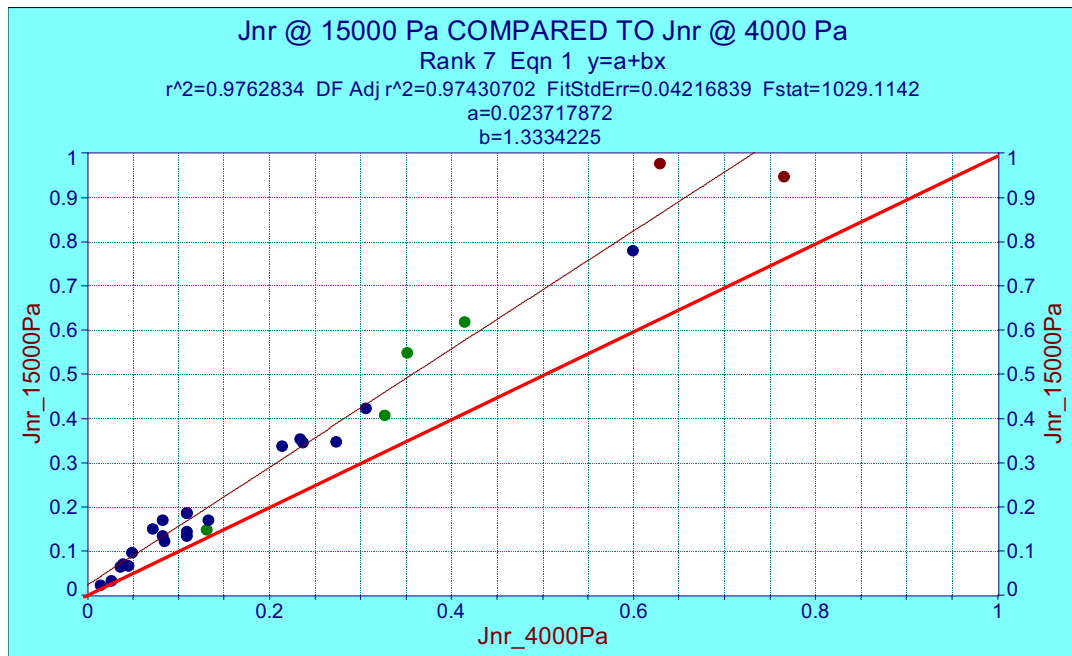
**FIGURE 6** Rut depth as a function of  $J_{nr}$  at 15,000 Pa for E-1 fine mix tested with 703-N (158-lb) load.



**FIGURE 7** Rut depth as a function of  $J_{nr}$  at 15,000 Pa for E-10 fine mix tested with 703-N (158-lb) load.



**FIGURE 8** Rut depth as a function of  $J_{nr}$  at 15,000 Pa for E-10 coarse mix tested with 703-N (158-lb) load.



At conditions where binder becomes more compliant (higher temps, weaker polymer network, lower stiffness base binder), the higher stress has a greater impact on the final  $J_{nr}$  result

**FIGURE 9** Comparison of  $J_{nr}$  at 15,000 Pa to  $J_{nr}$  at 4,000 Pa.

Since this work has been completed the proposed specification limits for  $J_{nr}$  have been altered by the Federal Highway Administration from 0.4 for standard traffic, 0.2 for heavy traffic and 0.1 for very heavy or standing loads. The new values are now 4, 2 and 1 respectively. Any data shown on the plots in this report are still relevant with the exception that the  $J_{nr}$  values would be multiplied by 10. A lower stress level of 4,000 Pa was used rather than the proposed value of 3200 Pa or is now recommended 3.2 kPa. As can be seen from the data in Figure 3 where both 3200 and 4,000 Pa data is shown, the impact of reducing the specification stress level from 4,000 to 3200 has little bearing on the outcome of discussions that occur in this document. It still seems relevant that for the determination of binder formulations for specific projects where heavy loads are to be encountered it would make sense for formulators to use a stress level of 15,000 Pa to determine  $J_{nr}$ . Based on the results in Figure 9, it is clear that to achieve the desired  $J_{nr}$  value at 15,000 Pa, it must also pass the specified  $J_{nr}$  at 3.2 kPa.

## CONCLUSIONS

A laboratory evaluation of the rutting response of different mixes produced using a wide variety of straight run, polymer modified, and modified binders containing PPA (polyphosphoric acid) or modified through air blowing was conducted. Through this investigation it has been possible to demonstrate the effectiveness of binder nonrecovered compliance as an effective tool for evaluating the contribution of these binders to permanent deformation resistance. Outcomes of this work are

1. Understanding the stress sensitivity of a binder is necessary to evaluate its potential to result in a rut susceptible mixture.
2. Knowing the PG grade as determined by  $G^*/\sin(\delta)$  or even knowing the  $J_{nr}$  value at a low stress level is no guarantee of a binder that will resist rutting under heavy traffic loading conditions.
3. Not all modified binders (polymer or otherwise) are created equal. Some polymer systems are more stress sensitive than others. This stress sensitivity needs to be evaluated for each binder source and polymer combination. A given polymer might perform very well in one binder and exhibit poor stress sensitivity in another. As a corollary it is worth remembering the point that a binder, even when modified, remembers its lineage; meaning that if a binder has poor stress sensitivity prior to modification it will not become a superior performing binder after modification. Modifying additives only make up about 1% to 4% of the binder, the remainder is the original binder.
4. It is always better to perform a mixture test to evaluate the rutting resistance of a given mix. In lieu of that the  $J_{nr}$  of the binder used in the mix should be determined at the climatic temperature and at a range of stress levels. This will provide an understanding of stress sensitivity and give assurance that the right amount and type of binder modification has been obtained.

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## **Developments in Intermediate Temperature Binder Fatigue Specifications**

**HUSSAIN BAHIA**

**HAIFANG WEN**

**CARL M. JOHNSON**

*University of Wisconsin, Madison*

It has been acknowledged that the current Superpave asphalt binder specification cannot reliably predict binder fatigue performance due to its reliance on measurements of non-damaged specimens in the linear range of their behavior (1–3). This issue was addressed during NCHRP Project 9-10, which focused on the applicability of the Superpave procedures to modified binders. The subsequent final report, NCHRP Report 459 (2), recommended that a new binder fatigue test be used and a new parameter,  $N_{P20}$ , be introduced. The benefit of  $N_{P20}$  was that it included both the damage and nonlinear characteristics of the binder. However, the testing time required to obtain the parameter was too long to be an efficient specification test. Also, concerns about repeatability and possible failures at boundaries of specimens were raised by users of the test.

Recently, significant research has been performed to identify a suitable accelerated binder fatigue test. An ideal test procedure would deliver pertinent fatigue characteristics without taking longer than 30 minutes to perform. Recent work by Martono et al. has verified the suitability of using the Dynamic Shear Rheometer for performing fatigue testing on binders and introduced the concept of using a stress sweep or a strain sweep as accelerated binder fatigue tests (4). Follow-up work for the verification of these efforts, and the consideration of other suitable accelerated tests has continued in the last 2 years. This document aims to highlight the recent developments and focus on one such test procedure and discuss its ability to indicate fatigue behavior of asphalt.

### **HISTORY OF BINDER FATIGUE TESTING**

The advent of the Superpave binder specification brought about a new mindset in asphalt technology, one in which critical mechanical properties would be measured under conditions that mimic climate and traffic conditions in order to predict the contribution of binders to performance of the HMA in pavements. Because of its highly temperature-dependent nature and the specificity of failures to temperatures, characterization needs to cover the entire range of temperature expected in the field, beginning at the low end for brittle behavior that causes cracking, and ending with high temperature characteristics to indicate rutting resistance. The focus of the characterization at the intermediate temperature has been the fatigue performance. At these intermediate temperatures it is believed that pavements will be in a more flexible condition due to weakening of the subsurface layers, as commonly seen during spring thaw, or due to softening of the asphalt layer itself due to warmer climate than winter conditions. In both cases the asphalt layers are subjected to higher cyclic strain that could cause fatigue.

The Superpave intermediate temperature parameter,  $G^*\sin(\delta)$ , is based on evaluating the amount of energy that the binder can dissipate over the course of one load cycle. By assuming thin pavement sections (with strain-controlled behavior) are most prone to fatigue damage,

minimizing the amount of dissipated energy is desirable so that the material is more prone to recovering the energy from applied traffic loading via elastic recovery, rather than dissipating the energy through permanent deformation or fatigue crack propagation (5).

The parameter  $G^* \sin \delta$  is, however, measured after a relatively small number of load cycles, such that no damage or non-linear behavior will manifest itself and subsequently need to be accounted for (see Figure 1). Many experts agree that fatigue damage resistance cannot be predicted from linear viscoelastic properties alone (2). A number of studies have shown that there is a poor correlation between the linear viscoelastic  $G^* \sin \delta$  and mixture fatigue performance (5, 6). Some amount of damage accumulation needs to be induced in the material in order to make an accurate characterization of fatigue performance. This was accomplished using the time sweep test during the NCHRP 9-10, which can be thought of as a traditional cyclic-loading fatigue test. By applying a sufficient amount of cycles to induce damage, a better fatigue characterization was obtained. As shown in Figure 1, binders can tolerate a large number of cycles with no damage as indicated by the constancy of the  $G^*$  values. This linear range could be followed by a range of nonlinear behavior during which modulus values ( $G^*$ ) will reduce due to damage until damage propagation results in total failure, as indicated by rapid reduction in  $G^*$  values.

NCHRP Report 459 outlined an analysis method that uses the ratio of dissipated energy in order to quantify the amount of damage in a binder specimen. The parameter  $N_{P20}$  was proposed as a revised fatigue parameter, as it represents a 20% reduction in the dissipated energy per cycle, which is assumed to be due to the damage induced. This approach provides a more comprehensive characterization of the fatigue performance of binders. The amount of time required to perform this traditional fatigue testing, however, has been a barrier to its inclusion in specification.

## INVESTIGATION OF ACCELERATED FATIGUE TESTS

To substitute for this time-intensive test, there have been recent attempts to explore alternate “accelerated” fatigue procedures for asphalt binder (7, 8). Much of the research performed on mixture fatigue testing served as a starting point for these binder works. The use of monotonic test procedures in particular to indicate the fatigue performance of mixtures has been

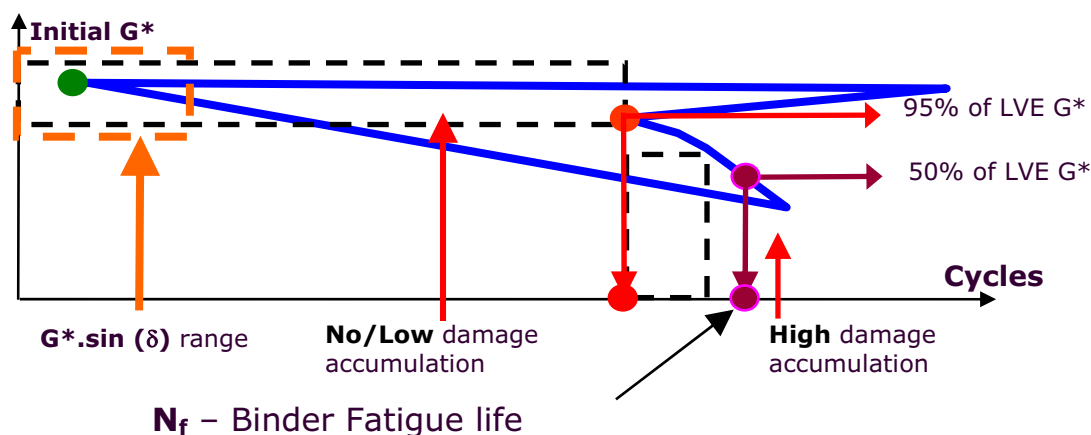


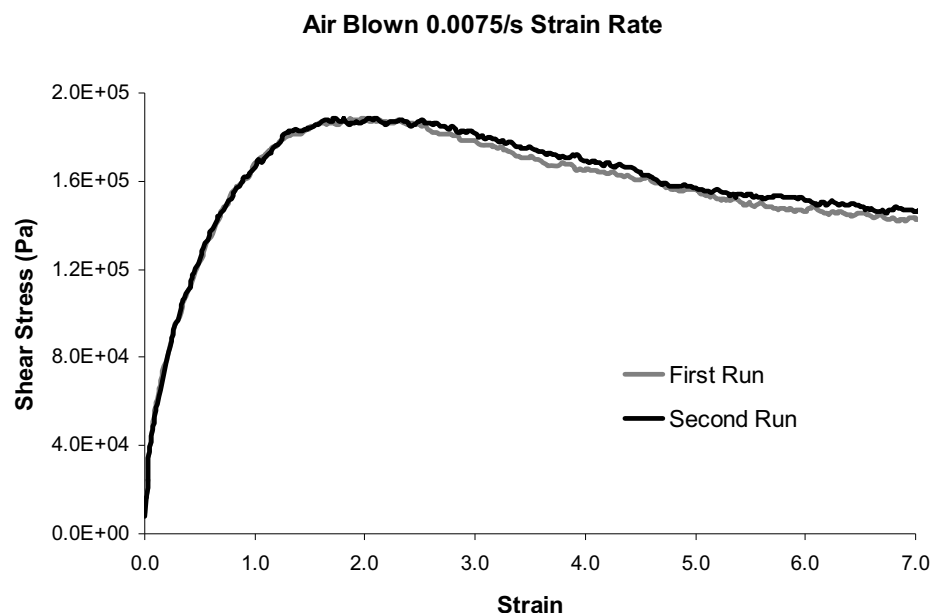
FIGURE 1 Schematic of time sweep test.

prevalent. One of the comprehensive works was conducted by Kim and Wen who used the indirect tensile strength test to measure the fracture energy of specimens used at WesTrack, and found a very good correlation with fatigue performance of the field pavements (9). The concept of using a monotonic test to estimate fatigue was applied to binder testing in a similar manner to the mixtures, except the binder test is performed in shear using the Dynamic Shear Rheometer (DSR). By applying a constant shear rate (usually achieved through the equipment's viscosity procedures), a stress-strain plot (see Figure 2) can be generated that is analogous to those created from other uniaxial monotonic tests performed on mixtures. The following sections cover the details of a study conducted to explore the use of this binder test.

## MATERIALS AND TESTING PLAN

The binders in this study (shown in Table 1) were also used as part of asphalt pavements tested in the FHWA's Accelerated Loading Facility (ALF). The test sections were subjected to repeated wheel loading, with their fatigue performance being recorded as cumulative crack length (m) due to fatigue failure after 100,000 passes. In order to simulate the oxidative state of the binders in the test sections, RTFO-aged binders were used for all tests.

A DSR fitted with an environmental test chamber was used to perform all monotonic shear tests at the ALF testing temperature of 19°C. In order to develop reasonable stress-strain plots, a trial-and-error approach was used to arrive at shear strain rates of 0.005/s, 0.0075/s, and 0.01/s. In order to remain within the applied torque limit of the machine, the 8-mm parallel plate testing geometry was used.



**FIGURE 2** Typical stress–strain output from monotonic shear testing.

**TABLE 1 Binders Selected for This Study**

Binder	Description	PG Grade
PG 70-22	Unmodified straight-run (control)	70-22
Air Blown	Air-blown oxidized binder	76-28
CR TB	Terminally-blended crumb rubber-modified	76-28
SBS LG	Linear-grafted SBS polymer-modified	70-28
Terpolymer	Ethylene terpolymer-modified	70-28

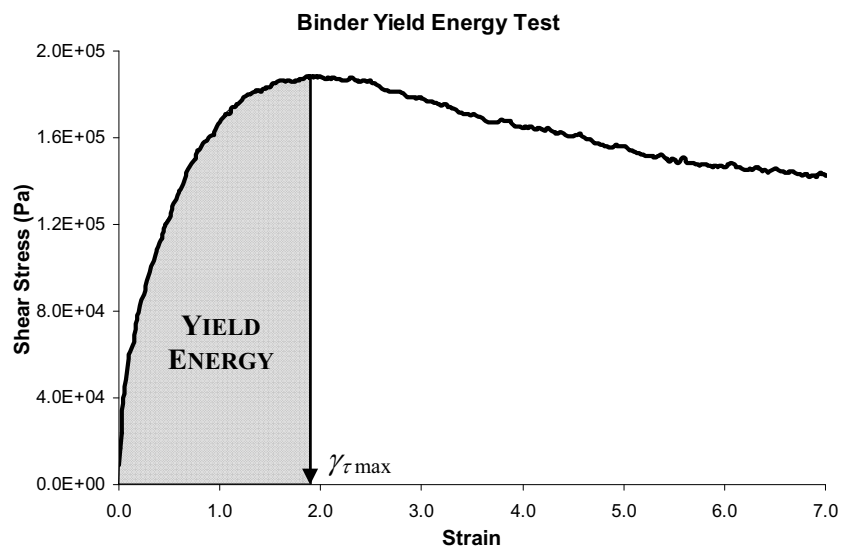
## RESULTS OF MONOTONIC SHEAR TESTING

Prior to any extensive modeling of the monotonic test data, some preliminary analysis was done to explore any apparent relations with the ALF field data. Two parameters were investigated—the area underneath the stress-strain curve up to the point of maximum stress (termed “yield energy”) and the strain at maximum stress ( $\gamma_{\tau \max}$ ).

The term “yield energy” (*YE*) is derived from work done by Kim and Wen as well as Roque on the properties of mixtures tested using indirect tension (9, 10). The area underneath the stress-strain curve until the point of material failure is known as the specimen’s “fracture energy.” However, during binder testing at intermediate temperatures under a constant shear-rate loading, the material never truly appears to fracture (a photograph of a failed specimen is given in Figure 3). Rather, it seems to reach a point where it is unable to take on further stress and simply begins to yield, thus the term “yield energy.” It is assumed at this point the material has failed, so *YE* is calculated as the area under the stress-strain curve bounded by the strain level corresponding to the point of maximum stress (Figure 4).

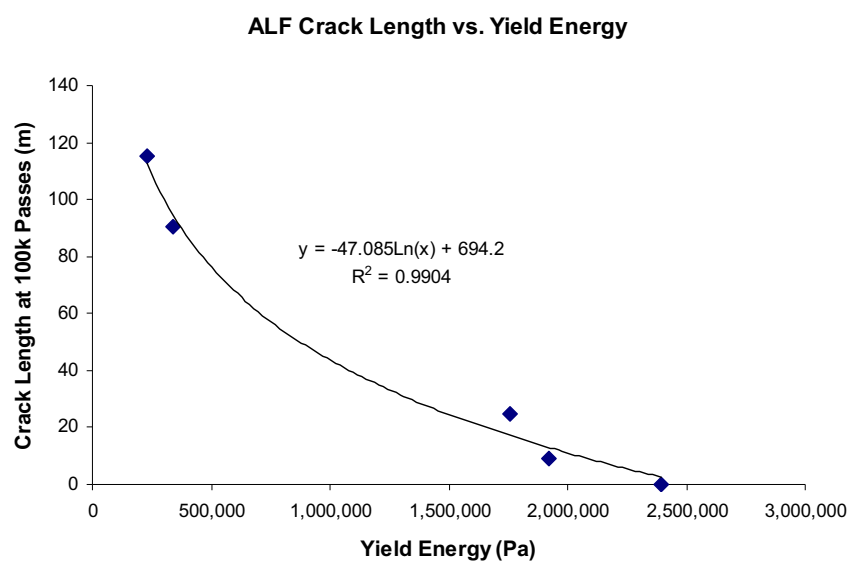


**FIGURE 3 Photograph of failed asphalt binder subjected to constant shear rate loading.**

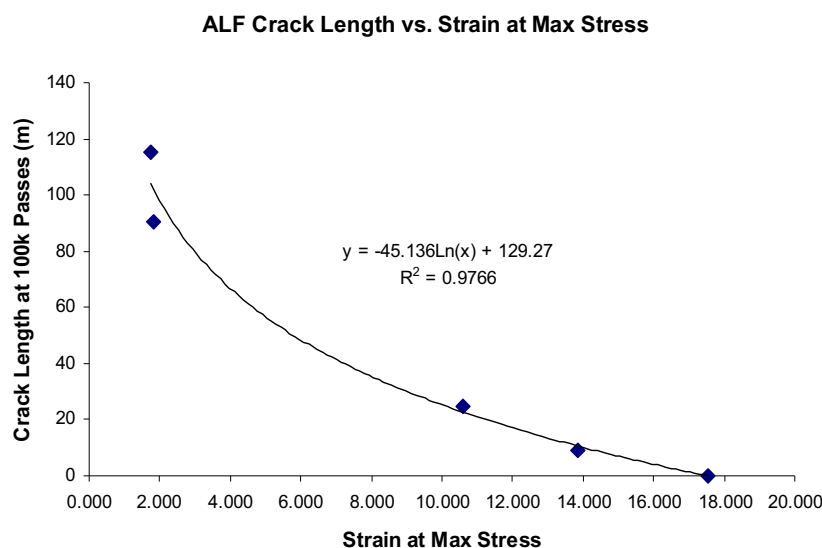


**FIGURE 4** Visual representation of monotonic shear test parameters.

The  $YE$  and  $\gamma_{\tau \max}$  for the ALF binders were then compared to the fatigue cracking data from the ALF test pavements. When plotted against cumulative crack length after 100,000 wheel passes, the  $YE$  and  $\gamma_{\tau \max}$  both show good correlation with the field data. Even though these parameters are based primarily on phenomenological approaches, their ability to accurately rank the performance of the ALF pavements (Figures 5a and 5b) has led researchers involved with this study to rename the constant shear test as the Binder Yield Energy Test (BYET).



**FIGURE 5a** Yield energy ( $\dot{\gamma} = 0.0075 / s$ ) versus ALF fatigue crack length.



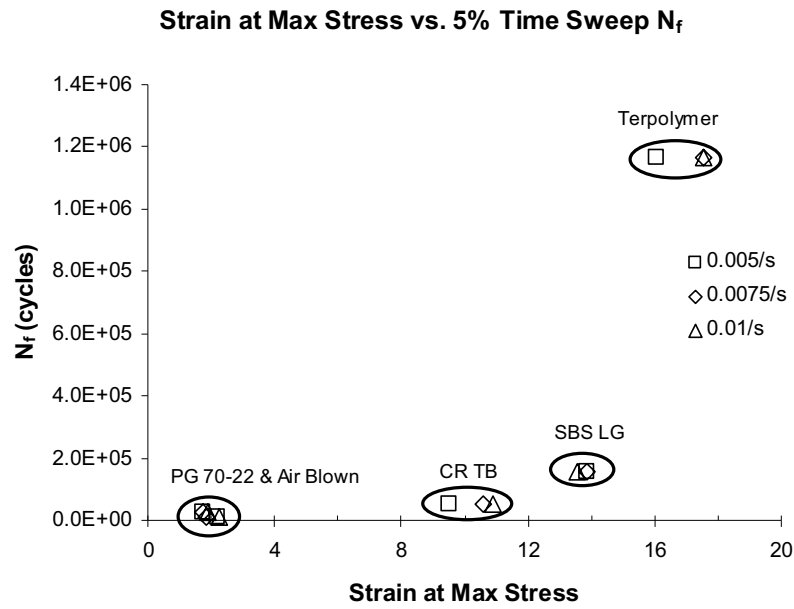
**FIGURE 5b**  $\gamma_{\tau \max}$  versus ALF fatigue crack length.

In addition to comparing BYET results to the ALF performance,  $YE$  and  $\gamma_{\tau \max}$  were also compared to the time sweep results published by Martono (7). The  $N_f$  values obtained from controlled-strain time sweep testing at 5% and 7% strain amplitude compare favorably with the BYET results for  $\gamma_{\tau \max}$  (see Figure 6), showing that the total amount of loading, and thus energy input, that can be applied to a sample before failure correlates to the total amount of deformation the material can withstand before becoming unable to accumulate additional stress.

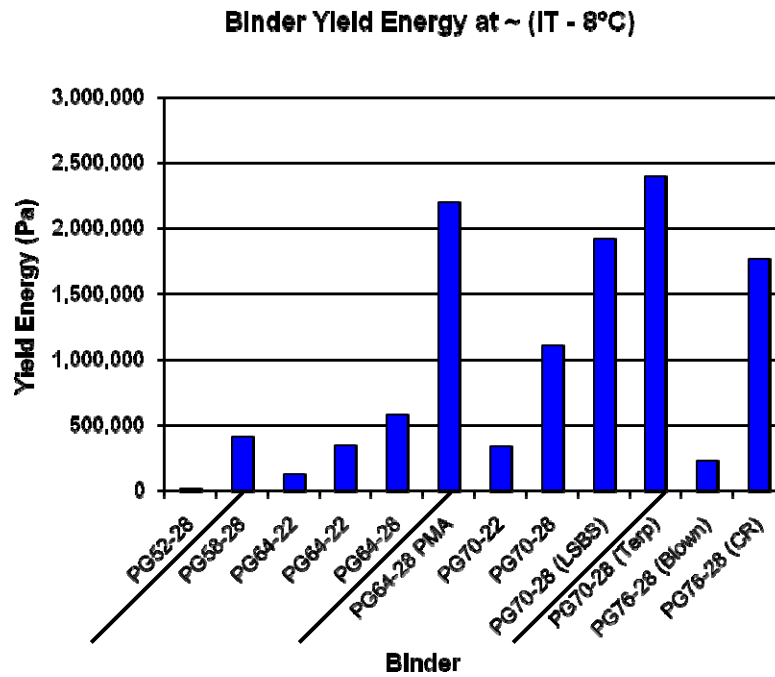
## SPECIFICATION SUITABILITY OF BYET

Due to the ability of the BYET to accurately indicate pavement and binder fatigue performance using parameters that are relatively simple to calculate, a preliminary investigation was pursued as an initial evaluation of the BYET for a specification-type procedure.

In order for a fatigue specification test to be valid, it must be able to discern the damage characteristics between materials at the same initial stiffness values, which  $G * \sin(\delta)$  is unable to do. The ALF pavement sections were tested at 19°C with binders varying in PG grade from 76-28 to 70-22, which corresponds to an approximate binder testing temperature at the Superpave intermediate temperature minus 8°C. Seven additional neat and modified binders of various PG grades from 52-28 to 70-28 were selected for BYET evaluation at the same temperature conditions for comparison to the results from ALF binder testing. A broad range of results can be seen in Figure 7, showing the  $YE$  calculated from BYET is capable of discriminating between the damage resistance of materials when tested at temperatures corresponding to similar values of  $G * \sin \delta$ .



**FIGURE 6** Plot of time sweep  $N_f$  against  $\gamma_{\tau \max}$  from BYET.



**FIGURE 7** Values of  $YE$  for various binders at Superpave intermediate temperature of  $-8^\circ\text{C}$ .



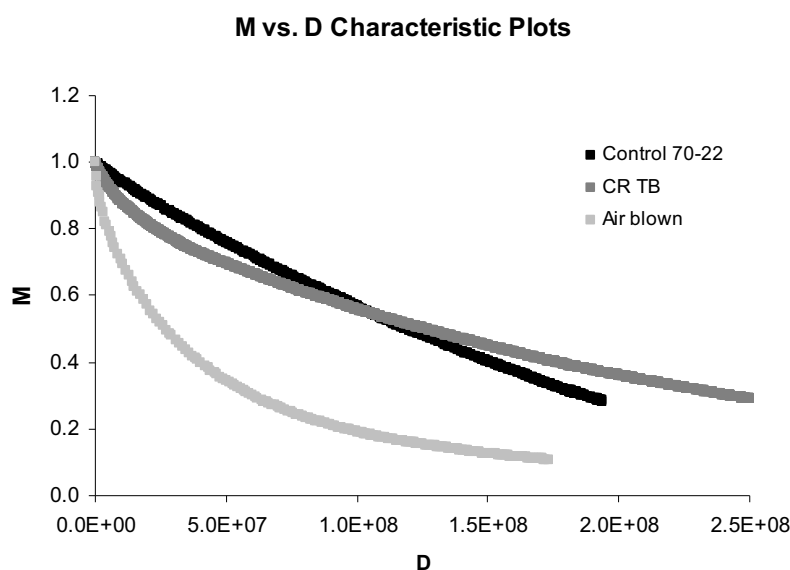
## APPLICATION OF VISCOELASTIC CONTINUUM DAMAGE CONCEPTS

One of the benefits of using the BYET for binders is the existence of models and mathematical formulations to use the results for estimating the effect of pavement structure, traffic speed, and climatic conditions on fatigue life of binders. The models and formulations are called the viscoelastic continuum damage (VECD) concept, which has been developed and used to characterize the behavior of asphalt mixtures (11, 12). The basis of this concept relies on comparing the predicted undamaged behavior of the material to the measured response of the material as it accumulates damage.

Preliminary work on applying the VECD to binders is already underway, with characteristic plots being generated to show the relation between damage intensity ( $D$ ) and material integrity ( $M$ ), as shown in Figure 8. The ultimate goal is to be able to take these characteristic plots and estimate the response of the binders under varying combinations of loading schemes and temperature conditions. This approach is not unique; for example, Kutay et al. have recently shown the ability of the VECD analysis to estimate fatigue behavior of asphalt mixtures subjected to varying conditions of cyclic loading (13).

## CONCLUSION

The Superpave binder specifications and testing methods have achieved a significant step in enhancing quality assurance of asphalt binders for paving applications. It is becoming clear, however, that rheological testing within the small stress or strain is not sufficient to fairly predict binder fatigue performance. Enhanced test procedure and analysis method, that are based on the advancements achieved by the Superpave system appears to be possible, and the needs for better



**FIGURE 8** Characteristic  $M(D)$  curves for the control 70-22, CR TB, and air-blown binders.

predicting of fatigue performance of asphalt binders in HMA could be met. This study indicated that by applying concepts developed for mixtures fatigue testing, a constant shear-rate binder test can be developed. The BYET has shown to be a promising technique for indicating fatigue performance of pavement test sections subjected to accelerated loading in the FHWA ALF. Further validation work needs to be done, but fundamental characterization of the material behavior under monotonic loading is currently being performed. As this work continues, development of the BYET into a specification procedure could be a logical step forward.

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## Low-Temperature Testing and Specifications

MIHAI MARASTEANU  
University of Minnesota

### BACKGROUND

The development of the Strategic Highway Research Program (SHRP) asphalt binder criterion for low-temperature cracking was based on the assumption that the 2-hour mixture stiffness correlated well with the severity of thermal cracking in the field (1). This assumption was extended to asphalt binder creep stiffness (inverse of creep compliance) obtained in low-temperature creep tests. To expedite the testing process the time-temperature superposition principle was used to show that, for asphalt binders in general, the stiffness at 60 s at  $T_1$ °C is approximately equal to the stiffness at 2 hours at  $T_1-10$ °C. To keep the PG binder specification to a reasonable level of simplicity the effects of physical hardening were not considered, although one of the major findings during SHRP was the significant effect of physical hardening on binder physical properties.

The slope at 60 s of the stiffness versus time curve on a double logarithmic scale, the  $m$ -value, was introduced as an additional parameter to control the rheological type of asphalt binders and to eliminate heavily blown asphalts, which in fact were associated with poor fatigue performance. This additional criterion was based on the idea that a low  $m$ -value corresponded to slower relaxation of the thermal stresses that build up at low temperatures, which was detrimental for performance.

A simple fracture test, called the direct tension test, was also required as part of the original SHRP binder specification. A dog bone-shaped specimen was pulled with a constant strain rate and the tensile fracture stress and strain were obtained. A second critical temperature was obtained as the temperature at which the failure strain was 1%. The 10°C shift was also applied to this temperature. Due to the low repeatability of the results, the direct tension test was made optional in the most recent version of the specifications. However, fracture experiments are known to be less repeatable than other material characterization experiments that do not involve fracture; in addition, test data indicates that the repeatability issue is significant only for certain types of binders, which indicates that poor repeatability may be a material property or a specimen preparation problem and not a testing problem.

The most recent addition to asphalt binder low-temperature specification is AASHTO MP1a standard (AASHTO 2002) which uses a thermoviscoelastic model to calculate thermal stresses using asphalt binder creep compliance. First, the experimental creep compliance data is converted to relaxation modulus using Hopkins and Hamming's (2) algorithm that numerically solves the convolution integral

$$\int_0^t E(t') D(t-t') dt' = t \quad (1)$$

where  $E(t)$  is the relaxation modulus and  $D(t)$  is the creep compliance, both in tension. The converted data are then fitted with CAM model for relaxation modulus (3):

$$E(t) = E_{\text{glassy}} [1 + (t/t_c)^w]^{-w/v} \quad (2)$$

where  $E(t)$  is the relaxation modulus at time  $t$ ;  $E_{\text{glassy}}$  is the glassy modulus equal to 3GPa;  $t_c$  is the crossover time; and  $w$ ,  $v$  are shape parameters in the model.

The thermal stress is then calculated by solving the following hereditary integral:

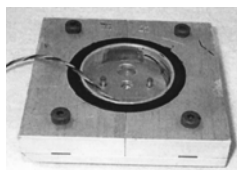
$$\sigma(\xi) = \int_0^{\xi} \frac{d(\alpha \Delta T)}{d\xi'} E(\xi - \xi') d\xi' \quad (3)$$

in which  $\alpha\Delta T$  represents the thermal strain and  $\xi$  represents the reduced time that takes into consideration that the time scale changes with the change in temperature. For simplicity of calculations, the horizontal shift factor is modeled as  $a_T = 10^{C_1 - C_2 T}$ , and a constant rate of temperature drop of 2°C/h was assumed. A pavement constant of 18 is used to convert binder thermal stresses to mixture thermal stresses. The stress curve is intersected with binder strength curve obtained from the direct tension test to obtain a critical temperature ( $T_{\text{CR}}$ ) below which transverse cracking occurs in the pavement.

The low temperature criterion was also developed based on experimental and field data for unmodified asphalt binders for which the  $S$ ,  $m$ , and failure strain temperatures were similar. For modified binders these temperatures can be significantly different. A number of researchers investigated the differences between plain and modified binders and proposed additional tests to better evaluate the fracture resistance of asphalt binders at low temperatures as described next.

## ABCD TEST METHOD

An innovative method that directly measures the critical cracking temperature of asphalt binders was recently proposed by Sang-Soo Kim (4). Using the significant difference between the coefficients of thermal expansion of asphalt binders and common metals, the asphalt binder cracking device (ABCD) was developed to simulate thermal cracking of asphalt pavement in the laboratory. The ABCD, shown in Figure 1, consists of a metal ring, a strain gauge glued to the inner side of the ring, an environmental chamber, and a data acquisition system. By monitoring the strain gauge reading, the development of thermal stress within the asphalt binder during cooling can be calculated and the cracking temperature can be directly determined. Compared with the current specification methods the ABCD offers a number of advantages. The most significant advantage is the simplicity of the test, both in terms of specimen preparation and conducting the test, and the possibility of running multiple specimens at the same time. Further comparison of the thermal stress development and critical temperature obtained with the ABCD method and the current test methods is needed to validate this new test method.



**FIGURE 1 Asphalt binder cracking device (ABCD).**

## FRACTURE MECHANICS-BASED TESTS ON ASPHALT BINDERS

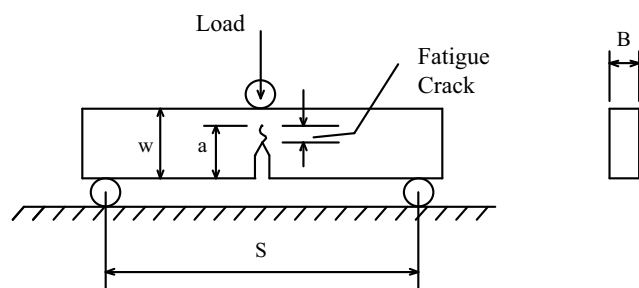
An increasing number of researchers have realized the limitations of predicting fracture properties based on the strength concept and started to investigate the application of fracture mechanics to the behavior of asphalt materials. Their research is based on well-documented fracture studies performed on other engineering materials such as steel, rock, concrete, polymers.

One of the best documented fracture tests is the standard single edge notched bend (SENB) beam test (5). The geometry of the SENB specimen is shown in Figure 2. A crack starter notch is machined at the middle point of the span from the bottom side of the beam. The beam is symmetrically supported by two rollers and a load is applied at the top side of the beam.

The existence of a crack results in significant stress concentration in the vicinity of the crack tip. A parameter, the stress intensity factor ( $K$ ), is used to characterize the stress field in the vicinity of the crack tip in the linear elastic fracture mechanics (LEFM) analysis. A subscript is used to denote the mode of fracture under which  $K$  is derived. For example,  $K_I$  means the stress intensity factor of Mode I fracture. Due to the fact that plane strain, transitional plane strain to plane stress, and full plane stress exist simultaneously, there are strict requirements for specimen geometry in this test method. The use of the recommended standard specimen, shown in Figure 2, results in fracture toughness values,  $K_{Ic}$ , that are reproducible to within 15% by different laboratories.

The accuracy with which  $K_{Ic}$  describes the fracture behavior depends on how well it characterizes the stress and strain field around the crack tip.  $K_I$  is calculated using the LEFM approach. However, materials, such as metals, crack with a plastic zone ahead of the crack tip, and therefore may not meet the LEFM assumptions. If the size of the plastic zone is small compared to the specimen dimensions, its effect on  $K_I$  can be ignored.

Lee and Hesp (6) and Lee et al. (7) used the SENB method to calculate the fracture toughness ( $K_{Ic}$ ) of asphalt binders. The experiment was controlled by the displacement of the crosshead at a speed of 0.01 mm/s. Based on test results from both plain and modified asphalt binder, they observed that the addition of modifier increased the fracture toughness of asphalt binders at similar creep stiffness values. To estimate the fracture energy, multiple beam specimens with different initial notch lengths were tested. The graphical method used in Dongre's work (8) was employed to calculate the fracture energy. Based on the analysis of the fracture energy, they concluded that modifiers significantly improved the fracture resistance of



**FIGURE 2** Single edge notched bend test ( $B = 0.5w$ ,  $s = 4w$ , and  $a = 0.45$  to  $0.55w$ ).

asphalt binders. Their work indicated that fracture properties of asphalt binders had more power in distinguishing different asphalt binders, especially modified asphalt binders, with respect to the low-temperature performance.

Hoare and Hesp (9) tested SENB asphalt binder specimens of different sizes. They kept the  $a/w$  ratio at 0.2 and varied the size of specimen. Three sets of dimensions were tested and the analysis of experimental results showed that no significant difference in fracture toughness had been found between specimens with different sizes. They concluded that plane strain condition required in ASTM E399 had been reached in their fracture test on asphalt binder.

Anderson et al. (10) used the same approach to obtain the fracture toughness of 14 types of asphalt binders: one plain base binder and 13 modified ones, and checked the effectiveness of characterizing binders' low-temperature cracking resistance with different grading methods. Specimens were stored at  $-20^{\circ}\text{C}$  before being removed from silicone molds and conditioned at the same temperature for 18 hours. A razor blade was used to sharpen the initial notch immediately before testing and an optical microscope was used to measure the crack length. The test was performed at the same crosshead speed of 0.01 mm/s and  $K_{Ic}$  was determined using the failure load. The authors found that the 14 asphalt binders were discriminated much better by the fracture toughness than the Superpave criteria with respect to the resistance to thermal cracking. This conclusion was in agreement with the conclusion from Hesp's work.

Olard et al. (11) studied the temperature and loading rate effects on the measurement of fracture toughness and fracture energy using SENB specimen of asphalt binder. They tested five types of asphalt binder, both plain and modified, at four crosshead speeds of 0.01 mm/s, 0.05 mm/s, 0.25 mm/s and 1 mm/s at temperatures ranging from  $-23^{\circ}\text{C}$  to  $-4^{\circ}\text{C}$ . They used beam specimens with the same dimensions ( $25 \times 12.5 \times 175$  mm) and an initial notch of 5 mm long. They, however, used different procedures to calculate fracture parameters. They followed closely to the ASTM standard to check the nonlinearity of the load–displacement curve before calculating the fracture toughness. Their results indicated that only few specimens showed nonlinearity in the temperature range they tested. They also developed an equation to facilitate the computation of fracture energy. Based on their experimental data, it was found that the fracture toughness was less dependent on the temperature and loading rate than was the fracture energy. Their data also indicated that there seemed to be a lower bound value of fracture energy of asphalt binders, which was reached at the glassy and brittle state of asphalt binders.

Another test geometry that has received considerable attention in the past years is the double edge notched tension (DENT). The DENT test configuration is presented in Figure 3. Similar to SENB geometry a polynomial expression is available to calculate the critical stress intensity factor  $K_{Ic}$  for Mode I of loading in such a specimen configuration (12).

Dongre (13) was the first to use the DENT configuration with the standard direct tension testing machine. Based on the LFM analysis he concluded that this geometry could be successfully used to measure  $K_{Ic}$  and  $J_{Ic}$  of asphalt binders. However, he recommended the use of  $J_{Ic}$  concept and elastic plastic fracture mechanics analysis to characterize the fracture behavior of asphalt binders based on his observation that the calculated  $K_{Ic}$  values depended upon the initial crack depth.

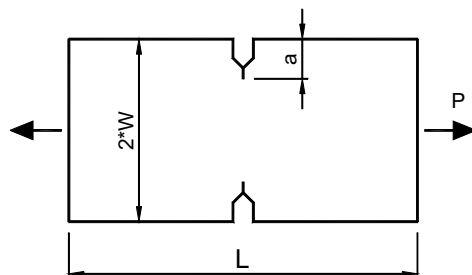
A detailed investigation of the fracture properties of asphalt binders at low temperatures was performed by Gauthier and Anderson (14). In their work they used both the SENB and the DENT configurations to evaluate the applicability of linear elastic, elastic plastic, and time-dependent (viscoelastic) fracture theories to asphalt binders at lower service temperatures. Based on the results obtained on eleven asphalt binders, they found that linear elastic fracture

mechanics analysis is applicable only at temperatures below the glass transition temperature. Above the glass transition temperature, neither elastic nor elastic plastic fracture mechanics are applicable to asphalt binders because in this temperature regime they behave as time-dependent viscoelastic materials and time-dependent fracture mechanics models must be used. They also indicated that the fracture parameters determined on notched specimens provide a different low-temperature ranking of asphalt binders than the PG specification. They concluded that the direct use of fracture mechanics parameters in a purchase specification is not realistic at this time.

The DENT method proposed by Gauthier and Anderson was also used, with a few modifications to the specimen preparation procedure, by Li et al. (15) and Zofka and Marasteanu (16). In their investigations, the authors showed that the fracture strain and stress values obtained for notched specimens are approximately 3 times lower than the fracture stress and strains obtained on direct tension specimens. They concluded that using notched specimen strength values may eliminate the need for the empirical “pavement constant” used in the thermal stress calculations required to determine the critical temperature.

### DISCUSSION ON THE VALIDITY OF THE LOW-TEMPERATURE CRITERION ASSUMPTIONS

The development of the low-temperature criterion was based on Readshaw’s limited study that showed that mixture creep stiffness at 2 hours correlated well with instances of transverse cracking (1). The validity of this correlation was extended to asphalt binders creep stiffness. The impracticality of a 2-hour test for specification purposes led to the use of the time–temperature superposition principle to develop an approximate relationship that would reduce the testing time to more practical values. The analysis of the shift factors for the eight core asphalts used in the SHRP research indicated that they had an almost identical relationship with temperature. Based on this observation, it was then assumed that the creep stiffness at 2 hours loading time at the minimum service temperature could be reasonably approximated by the creep stiffness after 60 s loading at 10°C above the minimum service temperature. However, actual 2-hour tests were not performed and the experimental validity of the assumption was not checked. In order to keep the specification to a reasonable level of complexity, the effects of physical hardening, one of the key findings in SHRP low-temperature research, were not taken into account.



**FIGURE 3 Double edge notched tension (DENT) specimen configuration.**

This equivalence was extended to an additional parameter, the  $m$ -value, which represents the slope of the creep stiffness as a function of time on a double logarithmic scale. According to Christensen, one of the authors of the original SHRP specifications, the  $m$ -value limit was included in the specification not to control thermal cracking, but to control the rheological type of the asphalt and prevent heavily blown asphalts, suspected to be prone to severe fatigue cracking, from being used.

Marasteanu et al. (17) investigated the validity of the equivalence of stiffness and  $m$ -value, after 2 h and 60 s at temperatures 10°C apart, for nine plain and modified asphalt binders used in the construction of asphalt pavements in Ontario. The effect of physical hardening on the time-temperature equivalence factor was also analyzed. In this research the authors used the BBR experimental data obtained using two pieces of software that captured 500 s and 3 h respectively of creep and recovery data. Tests were conducted on two replicate specimens at two temperatures,  $T_1$  and  $T_1-10^\circ\text{C}$ , and two conditioning times, 1 h and 3 days, respectively. Linear viscoelastic conditions were observed for all tests with the exception of the 3-h tests performed after 1 h of conditioning time for which physical hardening effects were significant.

The comparison of the stiffness at 60 s loading at  $T_1$  and the stiffness at 2 hours loading at  $T_1-10^\circ\text{C}$  showed significant differences between the two for all asphalts investigated. The 60 s stiffness values were significantly higher than the 2-h values and differences ranged from 32% to 66%. When converted to deviations from the 10°C equivalence factor the differences ranged from 1.5°C to 3°C. These differences were much reduced for tests performed after conditioning for 3 days that were less affected by physical hardening; the values were within 5% of each other for six of the asphalt binders and varied by less than 18% for all of them. In terms of deviation from the 10°C equivalence factor they were less than 0.7°C for all binders with the exception of one binder for which the deviation was 1.5°C. The comparison of the  $m$ -values showed a similar trend. The values were very different from each other and differences ranged from 14% to 21% for 1 h conditioning and from 10% to 23% for 3 days conditioning. In contrast to the stiffness comparison, the conditioning time did not reduce the  $m$ -value differences. Based on their results the authors concluded that the equivalence factor is asphalt specific and conditioning time specific.

The same set of asphalt binders was used by Basu et al. (18) to investigate how deviations from the validity of the time-temperature superposition principle affect asphalt binder low-temperature limits. The analysis showed that physical hardening has a significant effect on the limiting temperatures and on the  $T_{CR}$  values. The creep stiffness limiting temperature was less affected and showed an increase ranging between 2.2°C and 3.8°C. However, both  $m$  and  $T_{CR}$  show significant differences between the 3-day and the 1-h conditioned binders, with values as high as 12.8°C. The authors concluded that given the large differences between the various grading methods, the issue of which binder property to use for specification purposes and whether physical hardening plays a significant role could only be resolved through comprehensive field validation studies. Such a study is in progress in Ontario, Canada.

Marasteanu (19) investigated the relation between the stiffness and the  $m$ -value calculated from the BBR experimental data and the development of thermal stresses in asphalt pavements. The current specification procedure of selecting the highest of the two limiting temperatures given by  $S(60\text{ s})$  and  $m(60\text{ s})$  was also investigated. The analysis showed that thermal stress development is primarily controlled by the stiffness of the binder; a significantly softer binder results in lower stresses than a stiffer binder regardless of their  $m$ -value. As a consequence, binders that have a much higher  $m$ -value limiting temperature than the  $S$  limiting



temperature are most likely disadvantaged by the current specification with respect to low-temperature cracking properties. However, at similar levels of  $S$ , the  $m$ -value can significantly affect thermal stress development. Lower  $m$ -values, which are undesirable according to the current specification because of lower relaxation rates, result in slower development of thermal stresses, which is a desirable property. Therefore, the benefit of a high  $m$ -value for the low-temperature performance is not obvious. For climates where the temperature stays at reasonable low values for extended periods of time the higher  $m$ -value binders may be better performers as they allow more relaxation to occur. For climates characterized by extremely cold temperatures it is not obvious that higher  $m$ -values result in better performance as thermal stresses develop faster and can result in cracking occurrence before relaxation can occur.

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