(0092-14-20)
TPF-5 (302)
Modified Binder (PG+) Specifications and Quality Control Criteria

Task Report:
Work Area #2

Erik Lyngdal
Cheng Ling
Remya Varma
Ying Li
Professor Hussain Bahia
University of Wisconsin - Madison

Dan Swiertz
Consultant

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1. Introduction

The aim of the Pooled fund 0092-14-20 project is to provide essential information to state Department of Transportations (DOT) for five partner state agencies (Wisconsin, Ohio, Kansas, Idaho and Colorado) to support evaluation and standardization of PG plus (PG+) specifications. Based on needs and goals of each partner DOT, the main objectives of the proposed pooled fund research include:

1. Perform detailed assessment of current PG+ and modified binder quality control procedures in partnering states in terms of reliability, applicability, and relevance to performance and quality of modified asphalt binders.
2. Use a range of modified binders representative of the products currently specified by partner states to develop unified test procedures and specification criteria based on products placed in the field.
3. Improve product quality and reliability through ruggedness studies and development of precision and bias statements for selected tests.
4. Introduce consistency to current products supplied by elimination or reduction of differences in modified binder acceptance tests and criteria throughout member states.
5. Validate and establish relevance of suggested PG+ and quality control procedures in terms of mixture performance.

To meet the aforementioned objectives, the project was broken down into the four primary **Work Areas**. After completion of each work area, researchers are expected to produce task reports that document the work performed in the respective work area. This document is a task report that addresses the objectives in **Area #2** of the Pooled Fund work plan.

2. Candidate Replacement & Supplemental Test Methods

The primary consideration of this research is to provide guidance to the partner states on the usage and/or replacement of specific PG+ tests used in their respective specifications as identified in **Task Report #1**. It is therefore important to note that the interpretation of the data in this report is objective based. That is to say the decision on which test method(s) states choose to implement depends on whether partner states wish to simply replace a test currently in use (for example T301 elastic recovery) or implement a test method that simulates pavement performance in a given temperature range (for example Linear Amplitude Sweep). Research in the third phase of this project will attempt to determine the relationship between the binder properties shown in this report and mixture/field performance. Two primary considerations were made in selecting candidate replacement and supplemental test methods for this research: (1) the repeatability of the results (i.e. can we trust the data), and (2) the relevance of the results (i.e. are the test methods fundamentally sound and do the results make sense). Methods meeting both of these criteria are considered candidates for correlation with mixture performance.

Task Report #1, which was delivered earlier in the project, included a summary of the shortcomings of the PG+ tests used currently by the partner Stets in the pooled fund study, and offered candidate replacement test methods. A summary of the primary and secondary candidate replacement test methods is shown in Table 1.

Table 1. Candidate Replacement Test Methods

Current PG + Test Method	Associated Partner State	Candidate Replacement Test	Engineering Property Addressed
Phase Angle	OH, WI	**Primary:** ER-DSR **Secondary:** MSCR, %R	Elasticity (recovery of strain)
Elastic Recovery (T301)	ID, KS, OH, WI, CO	**Primary:** ER-DSR **Secondary:** MSCR, %R	Elasticity (recovery of strain)
Ductility (T51)	CO, OH	**Primary:** BYET, Strain at Peak Stress **Secondary:** None	Intermediate temperature strain tolerance
Toughness and Tenacity (D5801)	CO, OH	**Primary:** BYET, Strain at Peak Stress **Secondary:** BYET, Yield Energy	Intermediate temperature strain tolerance and resistance to deformation

In addition to the test methods shown in Table 1, which may not sufficiently address fatigue damage/cracking and thermal cracking in pavements, two supplemental test methods were introduced in **Task Report #1** for consideration by the partner states. A summary of these supplemental test methods is shown in Table 2.

Table 2. Supplemental Test Methods

Supplemental Test Method	Parameter Measured	Engineering Property Addressed
Linear Amplitude Sweep (TP101)	Cycles to Failure	Fatigue cracking
Single Edge Notched Bending (SENB-BBR)	Fracture Strength, Fracture Toughness	Thermal cracking potential

To establish whether candidate replacement test methods adequately capture the respective engineering property addressed, and therefore offer a suitable replacement to the current PG+ methods, correlations between the candidate test methods and the current PG+ methods were evaluated. Asphalt binders supplied by the partner states were tested using the candidate replacement test methods as well as current PG+ methods. Modified binder testing data collected by the Western Cooperative Test Group (WCTG) and analyzed by the research team is also included because it: (1) increases the number of data points available for correlation, and (2) represents a wider range of modification types in the analysis, making the conclusions more robust. Where appropriate, both binder groups are plotted on the same charts, and the correlation coefficients drawn from the pooled fund binder group and WCTG binder groups are delineated for clarity.

3. Asphalt Binder Elasticity Results

One of the primary objectives of this work area is to compare and contrast different test methods that measure the elasticity of asphalt binders. According to the partner state survey, the primary objective of the elasticity tests is to indicate the presence of elastomeric modification. Two parameters are generally specified across the United States to indicate the presence of an elastomer using elasticity: phase angle and AASHTO T301 elastic recovery. In **Task Report #1**, a comprehensive literature review of these two procedures indicated that there are inherent problems with using these elasticity tests to indicate the presence of elastomeric modification. Three general shortcomings were discovered: (1) elastomeric modification does not ensure that performance is equal to or better than binders that utilize other types of modifiers/additives, (2) elastomer indication tests do not directly address a specific mode of failure because it is not clear how elasticity contributes to rutting or fatigue resistance, and (3) more time and money are required to use an additional testing apparatus (for T301 elastic recovery only).

Despite the unclear relationship between asphalt binder elasticity and asphalt mixture performance, elastomeric binder modification has been empirically linked to an increase in performance of asphalt mixtures. It is therefore desirable to identify test methods that capture the elasticity of asphalt binder in order to provide an indication of presence or quality of elastomeric modification. In this section of the report, candidate test methods are compared with current Phase Angle and AASHTO T 301 Elastic Recovery (ER T301) test methods. Candidate methods include the Multiple Stress Creep and Recovery (MSCR) Percent Recovery (%R) and the Elastic Recovery using the DSR (ER DSR) procedure. The objective of the elasticity testing in this study was to determine if the MSCR %R or ER DSR procedure can be used as an alternative to the current PG+ plus procedures.

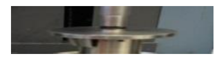
3.1 Evaluation of Phase Angle & Elastic Recovery T301

The phase angle measured at the high temperature PG of the asphalt is used by two partner states to indicate the presence or quality of elastomeric modification. Asphalt binders exhibit a viscoelastic response to loading; one defining characteristic of viscoelastic materials is that under cyclic loading (as with the DSR) a phase lag occurs between the resulting strain produced by a given stress or vice versa. For a perfectly elastic material, the phase lag (angle) is 0 degrees, whereas for a viscous fluid the phase angle is 90 degrees, or completely out of phase. Therefore, if a given asphalt binder has a lower phase angle, it can be said that the mechanical response of the binder is more elastic, hence the specification of an upper limit on phase angle for the partner states.

Phase angle is not a static measurement for a given asphalt binder, but rather depends on the testing temperature (similarly on loading rate); for unmodified binders the phase angle will increase with increasing test temperature. Elastomeric modification, with styrene-butadiene-styrene (SBS) in particular, is expected to reduce the phase angle at a given temperature. However, as more modification technologies become available, the extent (magnitude) of elastic behavior becomes more difficult to quantify when using the phase angle alone. Different types of elastomeric modifiers may result in very similar phase angle measurements yet offer unique and beneficial performance characteristics in the mixture. If the intention of the phase angle is to identify elastomeric polymers, a false-positive may result. Similarly, the non-linear viscoelastic nature of many modified asphalt binders tested using standard Superpave PG specifications could lead to erroneous rankings of modified binders using this approach. It is thus more desirable to directly measure elasticity using a strain recovery test like the ER DSR procedure or MSCR %R.

The ER DSR procedure introduced in **Task Report #1** was developed at the University of Wisconsin – Madison to provide a direct alternative to the AASHTO T301 elastic recovery procedure which uses a smaller sample, is less operator dependent, and conserves sample geometry during the test. The current procedure includes testing in the dynamic shear rheometer using the 8 mm plate at 25 °C. The shear rates and loading times were calculated to mimic those in the T301 procedure. A draft standard in AASHTO format is included in the Appendix of this report. A table that compares the two procedures is shown in Table 3.

Table 3. Comparison on ER DSR and ER T301 Procedures

Test Parameter	ER T301	ER DSR
Loading Time	120 sec.	120 sec.
Load Type	Tensile	Shear
Relaxation Time	1 hr.	1 hr.
Test Temperature (°C)	25 ± 0.5	25 ± 0.5
Sample Geometry	Bone Shape 	Cylindrical 
Unloading Procedure	Manually Cut	Zero Stress Applied by DSR
Conditioning Time	90 min.	20 min.
Conditioning	Water Bath	Water/Air/Peltier Plate
Data Acquisition	Manual Readings	Automatic

3.1.1 Comparison of Phase Angle and ER-DSR

Figure 1 shows the correlation between ER DSR measured at 25 °C and Phase Angle at the high temperature of the PG grade for both data sets. Fundamentally the trend indicated in the plot makes engineering sense: as the phase angle increases, the elastic recovery decreases, indicating less elastic behavior. Fitting linear trend lines indicate that both data sets show similar regression slopes and coefficients of determination, suggesting a high level of reliability in the analysis procedure. However, there is significant scatter at the intermediate range of phase angle values between 60-65 degrees. The scatter shows that a range of ER-DSR values between 55 and 75 % corresponds to phase angle of 63-65 degrees.

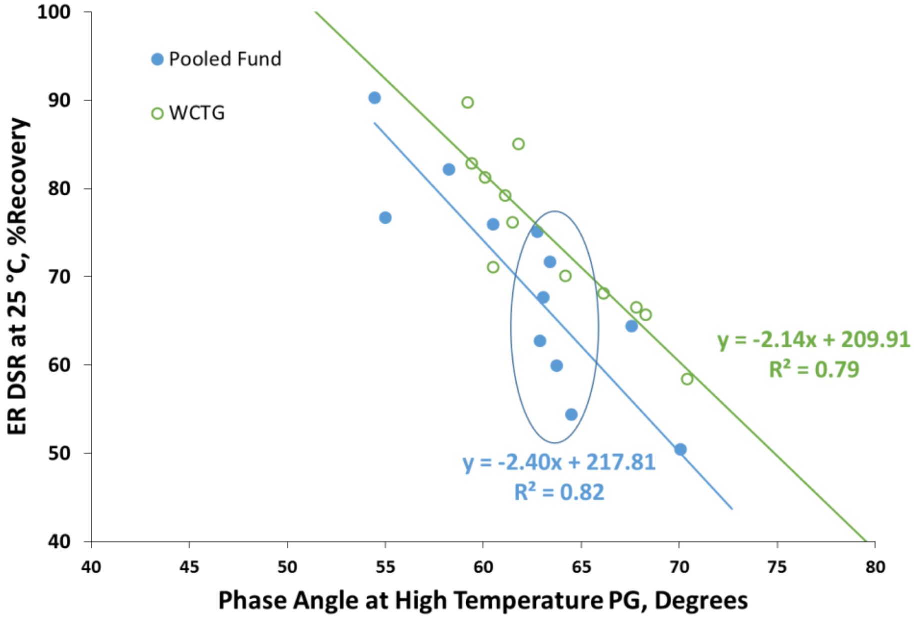


Figure 1. Correlation between Phase Angle and ER DSR results for the Pooled Fund and WCTG data sets.

The relatively strong correlation between phase angle and ER DSR is surprising for two reasons: (1) the large difference in testing temperature between the two test methods and (2) the nature of the test method. As previously mentioned, the phase angle is measured at high temperature PG grades while the ER DSR procedure is measured at 25 ˚C. Material properties of asphalt binder are expected to change at different rates for different types of polymer modification. Also, the phase angle is derived from a dynamic loading test method while the ER DSR method is measured from constant displacement rate test method. The correlation observed implies that the binder modification types were likely similar for the data set.

However, as previously mentioned, differences of nearly 20% in elastic recovery (data circled) were noted for samples showing nearly identical phase angles. This suggests the ER DSR is a more robust test for quantifying the degree of elasticity (in terms of elastic recovery) in modified binders, and using phase angle alone could be misleading. It is unknown how other binder modification types, such as non-SBS elastomeric modification or plastomeric-elastomeric hybrid modification, would fit into this analysis. Nevertheless, for the binders sampled in this study by the partner states, the correlations suggest that ER DSR could be a favorable replacement for phase angle and may be a better representation of elasticity in similarly modified binders.

3.1.2 Phase Angle Limits and Specification Effectiveness

Interestingly, the partner states using phase angle specify a maximum phase angle of 73-80 degrees, depending on the high temperature PG of the asphalt. Nearly all of the data analyzed falls well below the specified phase angle requirements, suggesting that current limits only determine if a binder is modified, but do not tell much about the type or the effectiveness of modification. An additional binder requirement may be needed to control modification levels, or identify the modification type. Using the current phase angle requirements and the pooled fund data correlation only, the associated lower limit on ER DSR percent recovery would be 28.2 -

45.1% for a phase angle of 80 degrees and 73 degrees, respectively. Both partner states that use phase angle as a PG + test also use T301 elastic recovery with a minimum limit of 60-65% recovery. Figure 2 shows the correlation between phase angle and T301 elastic recovery as well as ER DSR and T301 elastic recovery.

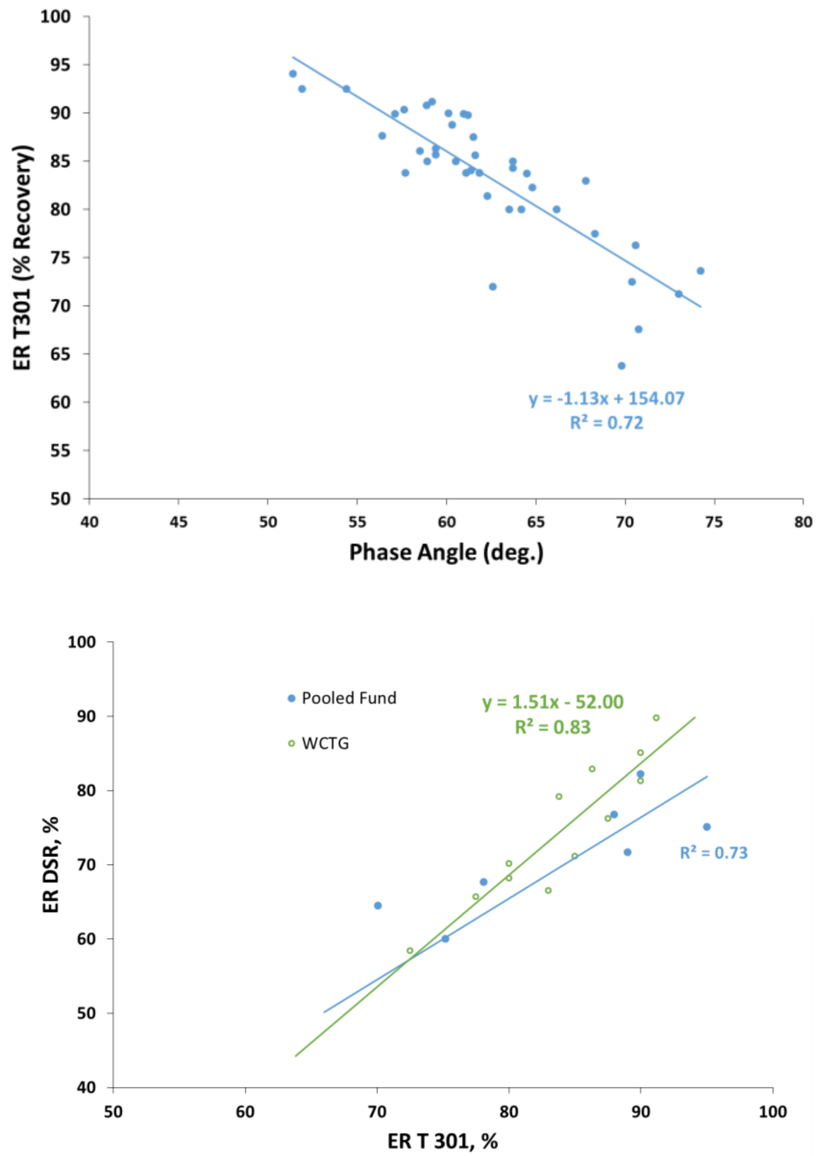


Figure 2. Correlation between T301 recovery and phase angle (top) and T301 recovery and ER DSR recovery (bottom).

The top plot in Figure 2 clearly shows that T301 elastic recovery is closely related to the phase angle measurement and by association the ER DSR measurement, as shown in the bottom plot. Although the coefficient of determination for the pooled fund data on the right-hand plot is less than that of the WCTG data, the data is generally included in the spread of the overall data set. Since the WCTG data represents binders sampled at the same time from the same source, a higher correlation is expected (i.e. less variability). It should be noted that similar correlations made in the literature found a coefficient of determination as high as 0.97 between the ER DSR and ER T301 for samples prepared and tested by the same operator in the same laboratory [1].

This data suggests that the current system of specifying both phase angle and T301 recovery is redundant to some extent as all binders that passed the phase angle requirements also passed the elastic recovery requirements. Since T301 recovery is similarly correlated with ER DSR, it can be reasonably concluded that the ER DSR procedure can directly replace both the phase angle and T301 recovery procedures with appropriate modification to the elastic recovery limits if that requirement is to be maintained by the partner states. If ER DSR is used, a reduction in the required ER DSR of approximately 30% is suggested to maintain the equivalent level of T301 elastic recovery.

3.2 MSCR Percent Recovery (%R)

Using the MSCR %R to evaluate elasticity in modified binder is attractive because it allows the user to gain more information (Jnr, stress sensitivity) for a given binder using fewer test methods and it directly measures elastic response in terms of strain recovery. Unlike the phase angle, which is measured at small strain in the linear range of behavior, the MSCR is considered a damage characterization test. MSCR testing directly ranks binders based on their damage resistance properties and therefore avoids the issue of non-linearity when testing complex polymer modified asphalt binder systems.

To evaluate possible correlations between Phase Angle and MSCR %R, testing results were plotted against one another as shown in Figure 3. There is a relatively poor relationship between MSCR %R and Phase Angle for the WCTG data set, although the fundamental relationship is logical. Since the pooled fund data is again included within the spread of WCTG data but includes far fewer data points, it can be reasonably assumed that using a larger data set for the pooled fund binders would result in a similar coefficient of determination as is shown for WCTG. It should be mentioned that plotting the data using 0.1 kPa or 10 kPa creep stresses resulted in nearly identical coefficients of determination and thus are not shown here.

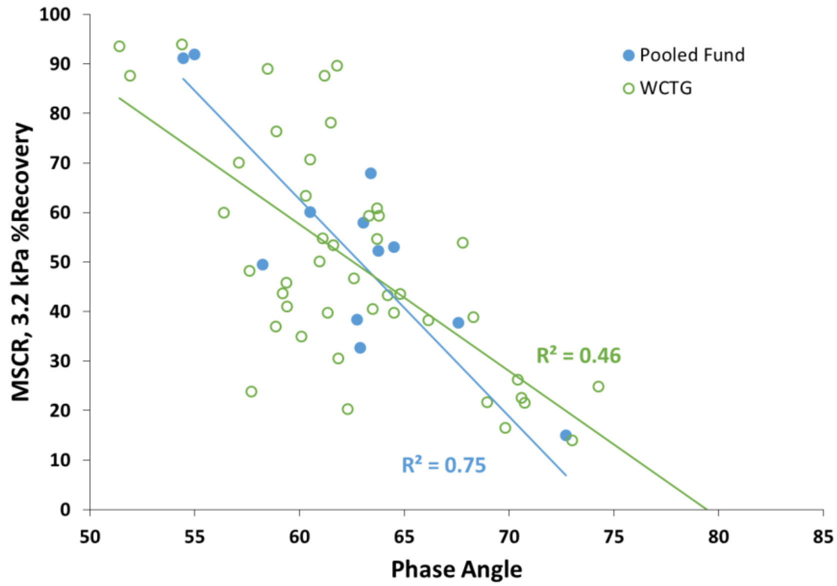


Figure 3. Correlation between Phase Angle and MSCR %R for asphalt binders tested as part of the Pooled Fund and WCTG.

Given that both of these properties are intended to indicate binder elasticity and are measured at the same temperature, it is surprising to see a lower correlation between MSCR %R and phase angle in comparison to the phase angle and ER DSR test measured at 25 °C. There are two potential reasons that explain lack of a correlation: (1) differences between creep and dynamic testing, and (2) stress dependence (non-linearity) of modified asphalt binder. To measure phase angle, an oscillatory load is applied to asphalt binder within the linear range (12% for unaged binder and 10% strain for RTFO binders), whereas a creep load is applied repeatedly for 20 cycles with each including a 1 second loading and 9 seconds unloading during the MSCR procedure. The loads applied to asphalt binder for the 3.2 kPa stress level used in the MSCR can result in strains well above 100% strain, which is generally within the non-linear region of behavior for most modified asphalt binders.

MSCR %R was also compared with T301 elastic recovery. Figure 4 shows the correlation between T301 recovery and MSCR %R at 3.2 kPa. Although a logical trend exists, there is a poor correlation between T301 recovery and MSCR %R for both the WCTG and pooled fund data set. A much lower coefficient of determination value was determined for the pooled fund data set. The difference in correlation can likely be attributed to the amount of data included into each data set. Interestingly, the range in MSCR %R is nearly three times larger than the range in recovery values for T301; whereas MSCR %R ranges from 14% to 94% (a spread of 80%), the T301 recovery ranges from 63.8% to 94.1% (a spread of 30.3%). This suggests a binder modified to meet T301 specification may still perform poorly (based on elastic recovery) at high temperature under the assumption the binder will not recover strain upon unloading (i.e. will rut). The poor correlation is somewhat expected as the two test methods are run at widely different temperatures and employ different loading techniques. This is elaborated on below.

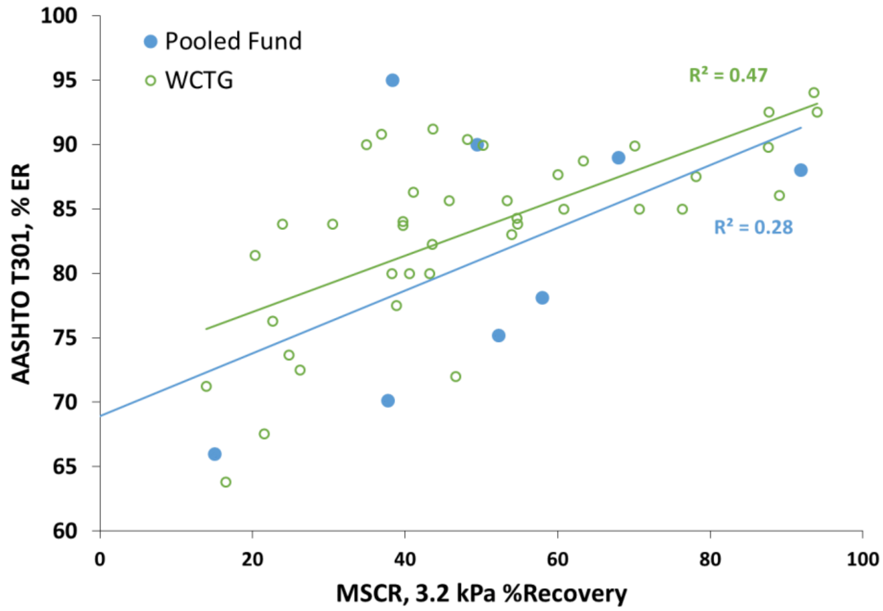


Figure 4. Correlation between ER T301 and MSCR %Recovery at 3.2 kPa for both Pooled fund and WCTG binders.

There are two primary explanations for the poor correlation between T301 and MSCR %R: (1) the MSCR procedure is conducted at the high PG temperatures and (2) the nature of each test method. Binder modification can affect binder elasticity differently at different temperatures for a given binder. An increase in elasticity due to binder modification at high temperatures may not provide a proportional increase in elasticity at 25 ℃, and vice-versa. Binders modified to meet T301 specification may not show the same ranking at high pavement temperatures.

The nature of the test method also suggests that a low correlation should exist between the two methods. The MSCR procedure measures elastic recovery using creep loading and recovery while the T301 procedure measures the elasticity recovery after a constant displacement of loading and recovery. In a creep and recovery procedure, the stress applied to the binder sample is controlled. For a constant displacement and recovery procedure, the stress felt by the asphalt binder during loading will change depending on the modification type and sample cross section, which is controlled by the Poisson's Ratio of the material being tested. The stress felt by the asphalt binder prior to recovery may not be proportional for the different types of testing procedures. Another primary difference between these tests is the time allowed for recovery. The data collected for both data sets shows a strong linear correlation between the Jnr and %R measured at 3.2 kPa (Figure 5). The majority of binders showing a low (under approximately 25% MSCR %R), the non-recoverable creep compliance, Jnr, is relatively high (above 2 kPa), and the deformation for a given cycle was likely higher. Since the recovery time is fixed at nine seconds, perhaps a significantly smaller percentage of the deformation was recovered than if a longer recovery time (as with the T301 procedure) was allowed. Hence specifying recovery using T301 does not necessarily correlate to recovery at high temperature using the MSCR.

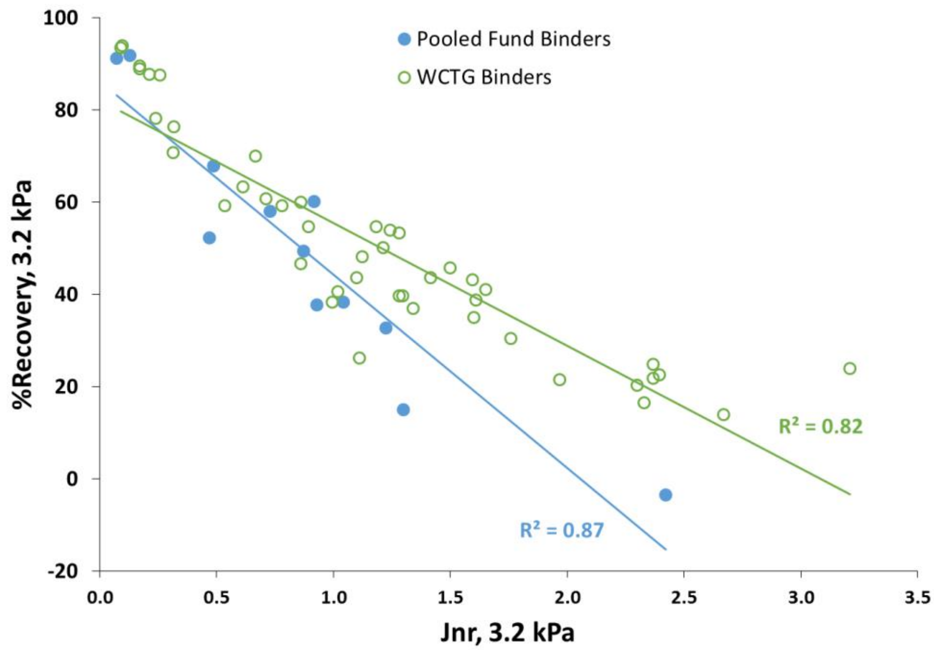


Figure 5. Jnr and % Recovery measured at 3.2 kPa.

These results imply that if performance at high temperature (i.e. reduction of permanent deformation) is desired, a high temperature test method that measures that performance should be used. For this data set, Figure 5 suggests that this would equate to specifying a minimum elastic recovery or specifying a maximum Jnr to limit deformation under a given traffic loading since the two are linearly correlated.

The data set shows that the MSCR %R cannot reliably be used to replace the T301 procedure; elasticity in the T301 procedure does not adequately predict high temperature elastic recovery using the MSCR. The decision should be made by each partner state whether the objective is simply to replace a given test method (i.e. T301) or implement a performance based test method like the MSCR. If a measure of elasticity (either quantifying amount or quality of modification) is desired by the partner states, the ER DSR procedure is suggested because it provides logical rankings and correlates well with existing methods using the binder set representative of the partner states. If instead a test method simulating actual high temperature pavement distress is desired, the MSCR should be considered because it allows for measurement of Jnr, which is a true measure of resistance to permanent deformation and % R is an indicator of elasticity. A more detailed investigation of the MSCR test as it applies to the sample set of binders is presented in the next section.

4. MSCR Implementation

In addition to the potential of the MSCR to indicate the presence of an elastomer using %R, there are other aspects of the MSCR procedure that can be used to predict the performance or quantify the quality of asphalt binder: non-recoverable creep compliance (Jnr) and percent Jnr difference (% Jnr Diff). Jnr quantifies the resistance of an asphalt binder to permanent deformation at high temperature, and has been used by some jurisdictions to replace current traffic PG grade "bumping." % Jnr Diff indicates the stress dependence of asphalt binder by calculating the percent difference between the Jnr at 0.1 kPa and 3.2 kPa. Please refer to **Task Report #1** for a more comprehensive overview of the MSCR implementation process and procedure. The intent of this section is to apply the knowledge gained from the MSCR literature review, covered in **Task Report #1**, to the data collected for the pooled fund binders to support implementation of the MSCR procedure.

4.1 Results When Using the MSCR Test to Classify Pooled Fund Binders

Each Pooled Fund binder was tested for MSCR according to the AASHTO TP 70 procedure and performance graded according to AASHTO M 332. The following sections are intended to show how each pooled fund binder would be graded if the MSCR was implemented in its full form. First, the MSCR results were graded to classify the respective binder traffic level as standard, heavy, very heavy or extremely heavy traffic. Next, each binder was analyzed for elastomer indication with the %Recovery-Jnr curve. Refer to **Task Report #1** for more background on the history and justification for selecting test conditions and limitations.

4.1.1 Traffic Level

Jnr traffic level specifications were intended to replace the current PG grade "bumping" system. Instead of increasing the high temperature PG grade for higher traffic levels, lower Jnr values are specified for higher traffic levels. Four traffic levels exist for Jnr: 4.5 kPa^{-1}, 2.0 kPa^{-1}, 1.0 kPa^{-1}, and 0.5 kPa^{-1} for standard, heavy, very heavy and extremely heavy traffic, respectively. In addition to the traffic level limits, a stress sensitivity parameter, called % Jnr Difference (% Jnr Diff), of 75% is specified and calculated as the % difference between the 0.1 kPa and 3.2 kPa stress level Jnr. Table 4 shows the MSCR grading result for each binder provided by the Pooled Fund members.

Table 4. MSCR Classification of Binders

Binder Grade	Jnr, kPa^{-1} (3.2 kPa)	MSCR Designation	% Jnr Difference	Pass/Fail
WI 58-34	0.603	V	12.1	Pass
WI 64-34	0.916	V	129.6	Fail
WI 70-28	1.23	H	53.7	Pass
OH 64-28	0.930	V	30.6	Pass
OH 70-22	0.469	E	276.7	Fail
OH 76-22	0.728	V	43.5	Pass
KS 64-28	0.074	E	-16.5	Pass
KS 64-34	0.488	E	-0.7	Pass
KS 70-28	1.04	H	46.6	Pass
CO 76-28	0.871	V	332.3	Fail
CO 64-28	2.422	S	13.5	Pass
ID 70-28; 13474	1.299	H	27.0	Pass
ID 70-28; 13435	0.133	E	-3.0	Pass

Results of the testing indicate that the majority of the asphalt binders provide a high resistance to rutting. Six of the nine binders met the MSCR requirements for very heavy or extremely heavy traffic volumes; both V and E asphalt binders are graded for 30 million ESALS of traffic. Given that all of the binders were tested at the high temperature PG grade, most of these binders do not need additional polymer to account for high levels of traffic. In fact, if a DOT implements the current MSCR PG grading standard, these results imply that less polymer may be necessary in comparison with current PG "bumped" binders. For example, if a DOT requires a PG 58E-28 instead of a PG 70-28, the amount of polymer required to meet the lower Jnr requirement may be less than the amount polymer required to meet the G*/sinδ at 70 ℃. However, Jnr is believed to be more representative of the failure mechanism for high temperature rutting in the actual pavement.

4.1.2 Elastomer Indication

The second aspect of the MSCR procedure is the elastomer indication with the %Recovery-Jnr curve. Elastomer indication with the MSCR AASHTO M 332 procedure is best understood visually in Figure 6. Pooled fund and WCTG binders were both plotted with the %Recovery-Jnr curve in Figure 6. If the data point in the figure is above the %Recovery-Jnr curve, then the binder is assumed to have been modified with an elastomer, while if the data is below the %Recovery-Jnr curve then, the binder is assumed to not contain an elastomer. Note that for Jnr values greater than 2.0 kPa, there is no minimum %R requirement.

All but three binders from both the pooled fund and WCTG data passed the requirement for indication of an elastomer. In addition, for both the WCTG and pooled fund binder data, there is a strong linear correlation between Jnr and %Recovery. This result is expected for binders with similar modification types. These results suggest that current binders being produced will pass the MSCR specification with an equivalent or lower content of elastomeric polymer, and is most likely the result of modifying binders to meet another specification requirement that requires more polymer (i.e. T301).

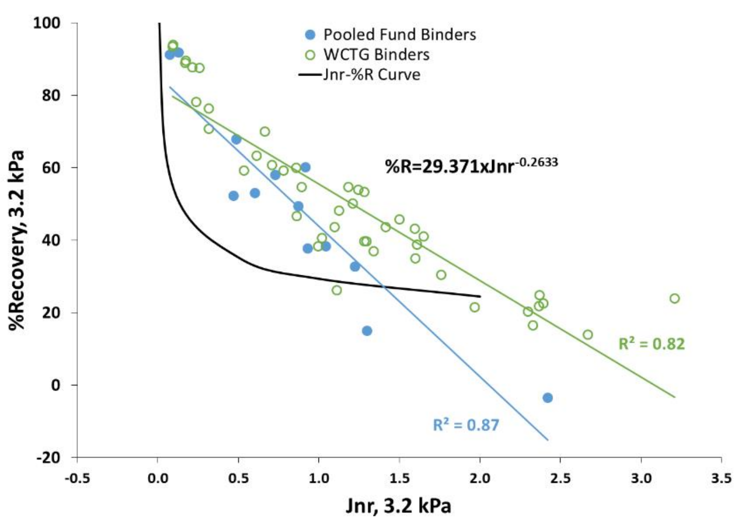


Figure 6. Elastomeric indication chart per AASHTO 332 with current data

4.1.3 Stress Dependence

When conducting creep and recovery tests on viscoelastic materials, such as asphalt, the Jnr is expected to remain constant for stress levels within the linear-viscoelastic region. When the creep stress level increases outside the linear viscoelastic region, the Jnr generally increases with increasing stress. In the MSCR procedure, the 0.1 kPa stress level is expected to be within the linear viscoelastic region for binders tested at the high PG temperature, while the 3.2 kPa stress level is expected to be in the non-linear region of material behavior. Therefore, the 3.2 kPa Jnr should always be equal to or larger than the 0.1 kPa Jnr. Depending on the base asphalt and modification type, the % Jnr Diff can range widely for different asphalt binders. To ensure that asphalt binders do not exhibit a high stress sensitivity, a maximum % Jnr Diff of 75% is placed on the Jnr from 0.1 kPa to 3.2 kPa in the MSCR standard AASHTO M 332. To understand how this specification limitation applies to the binder provided by Pooled Fund members, each binder was tested for MSCR at three stress levels: 0.1, 3.2 and 10 kPa. Then the % Jnr Diff was measured between each subsequent stress level (i.e. % Jnr difference between 0.1 and 3.2 kPa and % Jnr diff between 3.2 and 10 kPa). Each Pooled Fund binder % Jnr Diff values are compared in Figure 7. Each binder was labeled with the first letter of the respective partner state and the PG grade for identification.

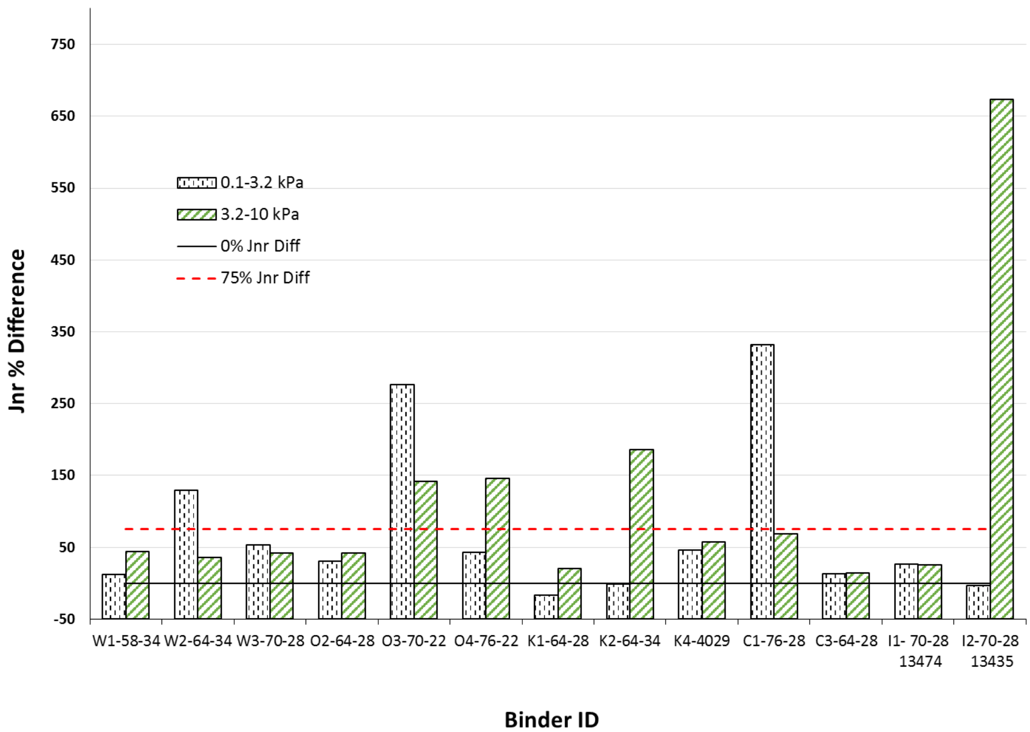


Figure 7. % Jnr Diff values for each Pooled Fund binder. Where 0.1-3.2 kPa data represents the % Jnr Diff between stress levels of 0.1 and 3.2 and 3.2-10 kPa data represents the % Jnr Diff between stress levels of 3.2 and 10 kPa.

As shown in Figure 7, there is a wide range of potential outcomes depending on the binder type. Three of the 0.1-3.2 kPa % Jnr difference values fail the % Jnr Diff parameter and have a 0.1-3.2 kPa % Jnr Diff that is significantly greater than the 3.2-10 kPa % Jnr Diff. For each of these binders, one was graded to withstand very heavy traffic and two were graded for extremely heavy traffic according to AASHTO M 332. Therefore, the asphalt binders meet requirements for large volumes of traffic, but fail the specification due to high stress sensitivity. These results are not logical, given that the same three binders have 0.1-3.2 kPa % Jnr Diff values greater than the corresponding 3.2-10 kPa % Jnr Diff values. As previously stated, as the stress level increases, the Jnr is also expected to increase. Therefore, the 0.1-3.2 kPa % Jnr Diff would be expected to be lower than the 3.2-10 kPa % Jnr Diff. A potential cause of the stress sensitivity failures can be found in the test procedure itself. For "E" and "V" binders, the Jnr at 0.1 kPa is extremely low, even approaching the resolution limits of the DSR, so when the % Jnr Diff is calculated, a very large number usually results.

It is therefore concluded that before % Jnr Diff is considered for implementation, state specifications should be written to allow for high performing binders with respect to Jnr alone, and waive the % Jnr Diff parameter.

5. Ductility

Two partner states currently use the ductility test (AASHTO T 51) at 4 °C to evaluate strain at failure for modified asphalt binders. **Task Report #1** provides a detailed critical analysis of the ductility test using available literature and justified the use of a DSR based procedure for measuring an analogous binder property. A DSR based procedure is desirable as DSR based procedures generally exhibit less variability, require less material, and are easier to run relative to temperature bath based test methods such as ductility and T301 elastic recovery. The DSR based approach introduced in **Task Report #1** is called the Binder Yield Energy Test (BYET). In the BYET test a sample of asphalt is tested by applying a constant strain rate and monitoring the resultant stress. The strain rate is calculated to be equivalent to the strain rate used in the ductility test. The full details of the test procedure can be found in Clopotel [1], and a summary of the test method in AASHTO format is provided in the Appendix. A typical output of the test is shown in Figure 8 below.

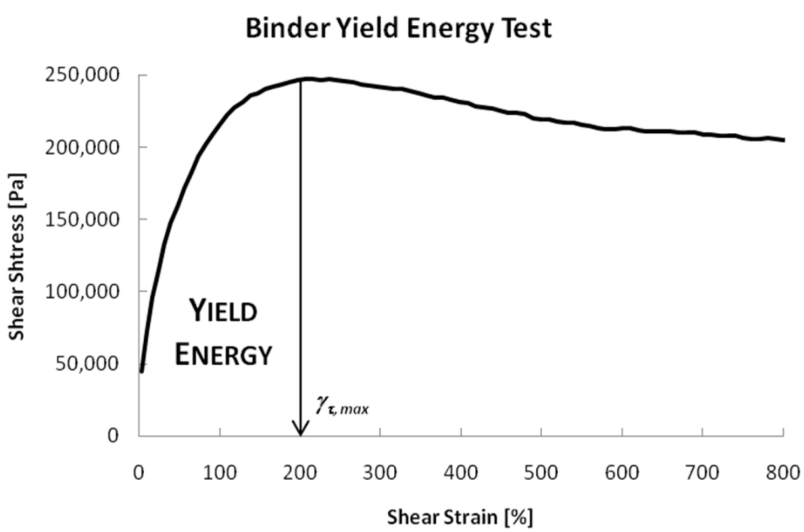


Figure 8. Typical output for BYET test

The output of the BYET test is used to calculate the strain at maximum stress, which is the peak of the stress-strain curve. This strain represents the apparent failure of the specimen under constant engineering strain, similar to the reported elongation at failure specified by the ductility test. Previous research has shown a strong correlation between the strain and maximum stress parameter and the ductility measured using the ductilometer [1]. The principle advantage of the BYET from an engineering standpoint is that the sample geometry remains essentially unchanged throughout the test. This eliminates the confounding effects that changes in geometry can have on ductility results, providing a more robust way to compare binders with different types of modification. The BYET test has been shown to more consistently identify the presence of elastomers in binder and produce a more logical ranking of binders modified with different types of polymers. Furthermore, the BYET strain at maximum stress matches very closely with the ductilometer results for neat binders, indicating the method is adequately capturing the same property [2], as shown in Figure 9.

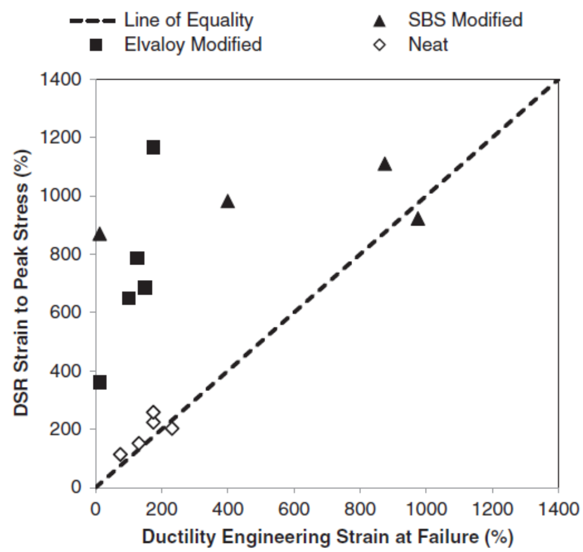


Figure 9. Comparison of BYET Strain at Peak Stress with T51 Strain at Failure [2].

To determine whether the BYET strain at maximum stress parameter can be used as a direct replacement for strain at failure in the T51 ductility test, WCTG data is used since the data set for the pooled fund binders does not include ductility. These results are shown in Figure 10.

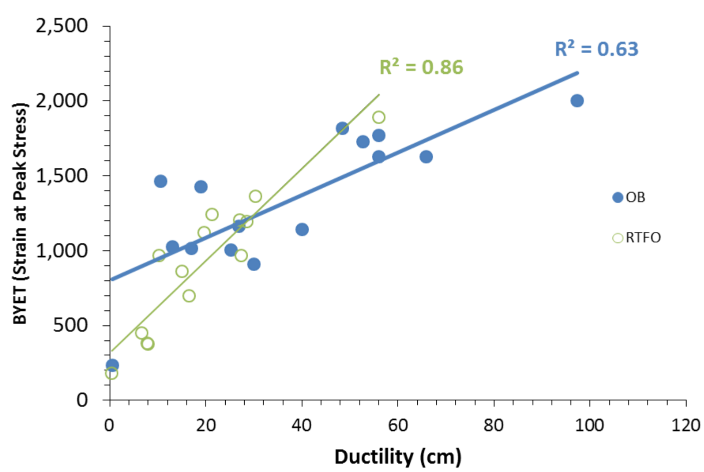


Figure 10. Comparison of BYET Strain at Peak Stress with Ductility at 4 °C.

The data shown in Figure 10 indicates a strong linear correlation between T51 ductility and BYET strain at maximum stress for RTFO aged binder and a moderate correlation for original binder. The trends are both logical with increasing T51 ductility correlating with an increasing BYET strain at maximum stress. If a replacement to the T51 ductility is desired, the BYET strain at maximum stress parameter appears to be a viable alternative based on the available literature and the results shown in Figure 10. The BYET test also has several methodological advantages that make it an attractive replacement. It should be noted that this analysis does not necessarily address pavement performance at intermediate temperature. Almost all of the literature available relating ductility to performance is based on unmodified binders; studies with modified binders have generally concluded that the ductility test cannot reliably predict pavement performance [2]. It is for this reason that the researchers recommend considering a damage characterization test at intermediate temperature (e.g. the linear amplitude sweep test) if partner states wish to implement a performance predictive test measure for intermediate temperature pavement distress.

6. Toughness and Tenacity

The toughness and tenacity test measures an asphalt binder's ability to withstand a tensile force under a constant deformation rate. This test is often used to characterize the elastomeric properties of asphalt binders. This test was first introduced by Benson to better characterize rubberized asphalt binders, and as the test gained more popularity in the 1970s, began to be used as an indication of polymer modification [3, 4]. From an engineering standpoint, the toughness and tenacity test suffers from the same shortcomings as the ductility test; the true stress experienced by the sample grows exponentially as the sample cross section is reduced. Since the base asphalt chemistry and type of modification will both influence the Poisson's Ratio and stress relaxation rate of the sample, the change in cross section and resultant stress will not be consistent between binders. Thus, comparing samples with different polymer loading (both type and quantity) may change ultimate ranking of samples.

Since the toughness and tenacity test shares many similarities with the ductility test up until the point of apparent 'yield', the BYET test is similarly considered in this analysis. Before that analysis is presented, an important note about the strain rates used during testing should first be made. The BYET test was designed to have an equivalent strain rate to the T51 ductility procedure (ran at 5 cm/min). The toughness and tenacity test is run at 50 cm/min. Since these strain rates are an order of magnitude apart, it is assumed the true stress in the sample will be significantly different for the BYET and toughness and tenacity test for a given strain. It is nevertheless desirable to use the same BYET procedure to replace T51 ductility and toughness and tenacity. For this testing the BYET was run at 25 °C, the same temperature used for toughness and tenacity as correlations drawn from the BYET test at 4 °C were inconclusive.

The toughness of an asphalt sample is defined as the area under the force-displacement curve up until the point of apparent maximum stress and a tangent drawn from that point to the displacement axis. This is best shown graphically in Figure 11. The parameter of interest from the BYET test is the yield energy, which is defined as the area under the shear stress-strain curve up until the point of apparent maximum stress. Given the similarities between the two metrics, and since the engineering stress is a function of the force applied (load), the two parameters should be related to some degree.

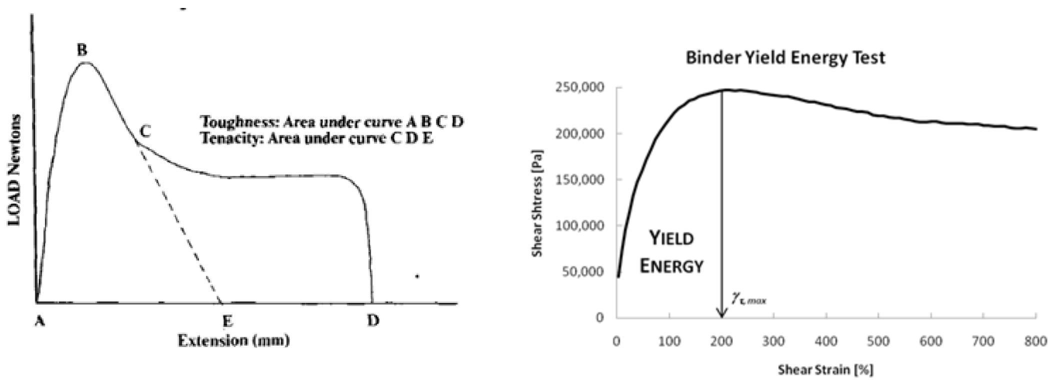


Figure 11. Definition of toughness, tenacity, and yield energy.

Figure 12 shows the correlation between toughness and yield energy at 25 °C for available WCTG samples. The data has been fit with both linear and power law regression for comparison purposes. The trend in the data is logical, as the yield energy of the binder increases, so too does the toughness. Although the correlation is relatively poor, given the differences in strain rates between the two tests, unknown modification types and polymer loadings, and somewhat arbitrary means for defining the toughness, the results are promising.

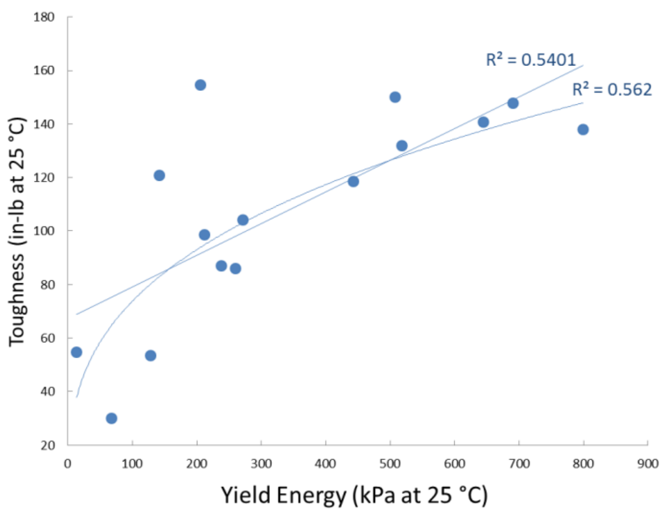


Figure 12. Toughness and Yield Energy at 25 °C for WCTG binder set.

The tenacity of an asphalt sample is defined as the total area under the force-displacement curve excluding the toughness region (see Figure 11). The BYET test, however, is not run until complete failure of the sample, and often the shear stress in the sample plateaus after several 1000% strain. It is therefore difficult to draw analogies between the BYET test and the tenacity parameter. Interestingly, however, the toughness and tenacity appear to be linearly related for the WCTG data set, as shown in Figure 13. Although this relationship cannot be confirmed for all modification types, for this data set the correlation suggests a redundancy in reporting both values. As an artifact of the trend shown at left in Figure 13, the yield energy should therefore relate to tenacity in the same manner as toughness. Figure 13 (right) displays this relationship for the data set; for modification types that show a linear relationship between the toughness and tenacity parameters, the BYET yield energy can reasonably be used to predict both metrics using a power law function.

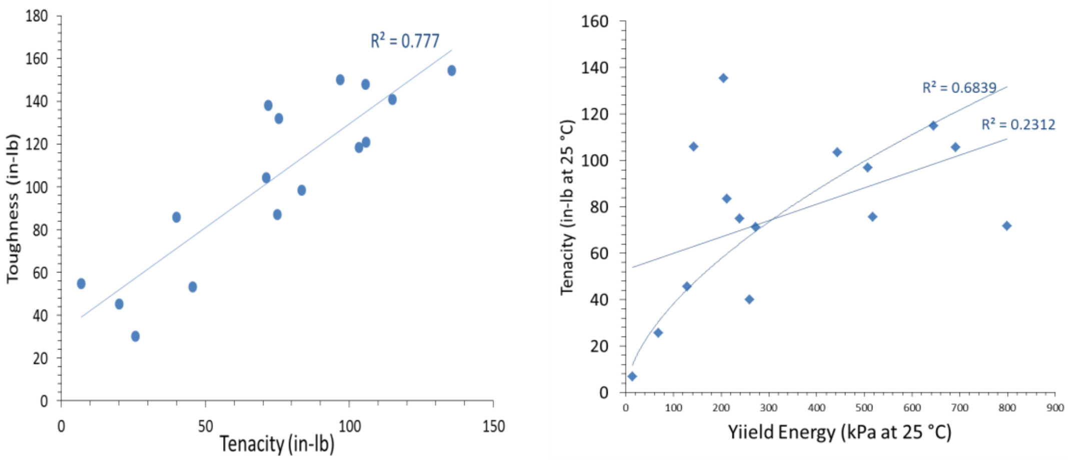


Figure 13. Toughness and tenacity for WCTG samples (left); Tenacity and yield energy at 25 °C (right)

The data suggests the toughness parameter can reasonably be estimated using the BYET yield energy at 25 °C. If the tenacity parameter remains of interest to the partner states, results from representative materials should be plotted in a fashion similar to Figure 13. If a linear correlation exists, it is suggested that the yield energy parameter from the BYET run at 25 °C be used to estimate minimum specification limits. If the toughness and tenacity are linearly related, specifying one (i.e. toughness), will satisfy the other.

7. Supplemental Test Methods

7.1 Intermediate Temperature – Linear Amplitude Sweep

Modified asphalts are expected to enhance the performance characteristics of pavement at intermediate temperatures; fatigue cracking behavior is more problematic at intermediate temperatures. Characterization of modified binders using the standard Superpave PG system at intermediate temperatures has generally proven to be ineffective for predicting pavement distress at these temperatures. Damage resistance testing is recommended for characterizing modified binder performance since it ranks binders based on their failure properties and minimizes the non-linearity effects that can confound standard PG results when using modified binders. The Linear Amplitude Sweep (LAS) test is one such test that has shown promise in characterizing modified asphalt binder fatigue damage resistance.

The LAS test (AASHTO TP101) is a DSR based test that was developed at the University of Wisconsin-Madison by Johnson et al. and modified by Hintz to indicate asphalt binder fatigue [5, 6]. LAS testing has been proposed as an accelerated method to measure asphalt binder fatigue. To conduct a LAS test, an 8 mm sample is placed in the DSR at intermediate temperature ranges. A frequency sweep first measures the asphalt binder linear viscoelastic properties. Next, a strain amplitude sweep is run from 1% strain to 30% strain at a constant frequency of 10Hz [6], during the test the strain amplitude is increased linearly. After completion of the test, viscoelastic continuum damage mechanics are used to analyze the DSR data; resulting in a fatigue life equation. Derivation of a fatigue life model gives an equation that directly relates the expected traffic load to the expected fatigue life of an asphalt binder or pavement. A typical fatigue law equation is described by the following equations and depicted in Figure 14 [5, 6, 7].

$$N_f = A * (Load\ Amplitude)^B$$

Where, A and B are derived material properties. A visual representation of the parameters is shown in Figure 14.

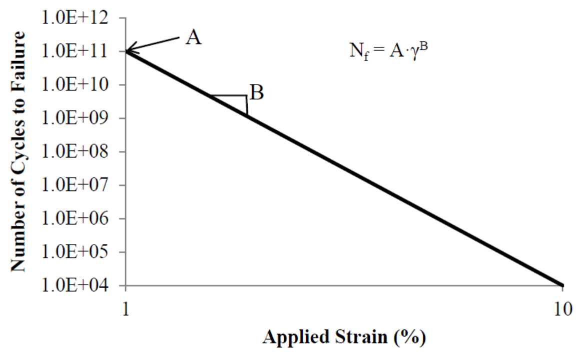


Figure 14. Fatigue law on log-scale taken from Hintz [6].

In order to derive the fatigue law from the LAS test data, the accumulated damage at each strain amplitude must be monitored. In general, the amount of energy dissipated after each strain amplitude in the DSR relates to damage accumulation. Hintz et al used viscoelastic continuum damage theory (VECD) to develop an analysis template that converts material properties measured in the DSR to the fatigue law parameters A and B. Based on the predicted strain experienced by an asphalt binder in service, the resulting fatigue life can be estimated.

The aforementioned analysis template can be found at the following website: http://uwmarc.wisc.edu/. A recent study funded by the Wisconsin Highway Research Program (WHRP) used the LAS test to characterize fatigue cracking resistance of modified binders in Wisconsin with success. Results of the study showed that by selecting appropriate structural parameters (i.e. strain in the pavement layer) in the LAS procedure, a logical and strong correlation between Cycles to Failure (Nf) and percentage of cracking in the field. Conversely, the Superpave G*sinδ parameter showed a very low correlation for the same pavements. A summary of this data is shown in Figure 15 [8].

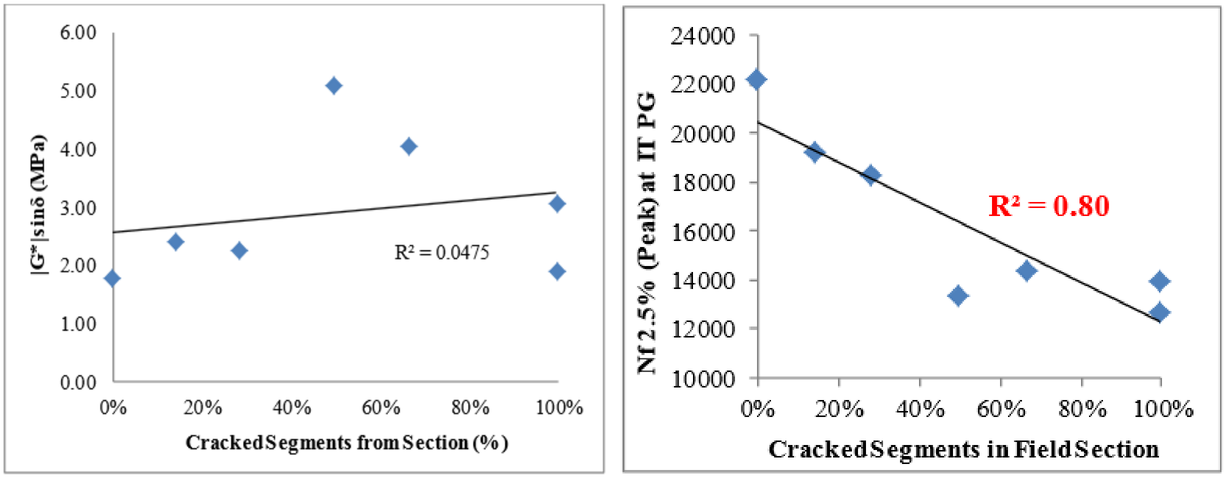


Figure 15. Comparison of Cycles to Failure Measured by LAS test to field fatigue cracking in Wisconsin [8].

The same WHRP study offered a preliminary specification based on Nf based on the expected traffic level, shown in Figure 16. In the current study it is recommended that the LAS Cycles to Failure (Nf) parameter similarly be used in addition to the standard Superpave G*sinδ parameter to evaluate fatigue cracking potential. The binders supplied by the partner states will be evaluated using the current LAS procedure and results compared against the mixture testing to be conducted in Phase 3 and ultimately compared to field performance. From there, acceptable limits on Nf from Figure 16 can be adjusted by defining minimum acceptable field performance and back calculating the associated Nf level.

ESALs	**WisDOT Mix Design Category**	Minimum LAS N_f Required (PAV Condition)
300,000	E-0.3	28000
1,000,000	E-1	30000
3,000,000	E-3	33000
10,000,000	E-10	41000
30,000,000	E-30	67000
ESALs	**AASHTO MP-19 Grade**	Minimum LAS N_f Required (PAV Condition)
3,000,000	S	33000
10,000,000	H	41000
30,000,000	V, E	67000

Figure 16. Preliminary specification based on Cycles to Failure in LAS test from [8].

Another possible test for intermediate temperature fatigue is the BYET test. The Yield energy at the intermediate temperature has been found to correlate well with Fatigue cracking at the ALF facility. Figure 17 shows the composite Score and relationship between fatigue measured on ALF and the various parameters. As shown the BYET gave the second highest correlation value.

Binder Test for Fatigue Cracking	Comparative Data	$1\text{-}p_{Reg}$	τ_K	$1\text{-}p_{\tau K}$	R	Composite Score
Critical Tip Opening Displacement	Axial Fatigue	99%	1.00	99%	0.95	0.99
	ALF Cracking	100%	1.00	99%	0.98	
Binder Yield Energy	Axial Fatigue	94%	0.80	96%	0.87	0.88
	ALF Cracking	90%	0.80	99%	0.80	
Time Sweep	Axial Fatigue	89%	0.80	96%	0.79	0.88
	ALF Cracking	95%	0.80	96%	0.88	
Failure Strain in Low Temperature Direct Tension Test	Axial Fatigue	92%	0.60	88%	0.83	0.81
	ALF Cracking	93%	0.60	88%	0.85	
Superpave $\|G^*\|\sin\delta$	Axial Fatigue	84%	-0.60	88%	-0.73	0.75
	ALF Cracking	78%	-0.60	88%	-0.66	
Large Strain Time Sweep Surrogate	Axial Fatigue	85%	-0.40	76%	-0.74	0.67
	ALF Cracking	78%	-0.40	76%	-0.67	
Essential Work of Fracture	Axial Fatigue	53%	0.40	76%	0.43	0.55
	ALF Cracking	60%	0.40	76%	0.50	
m-value from Low Temperature Bending Beam Rheometer	Axial Fatigue	63%	0.40	76%	0.52	0.54
	ALF Cracking	47%	0.40	76%	0.38	
Stress Sweep	Axial Fatigue	89%	-0.40	76%	-0.79	0.69* *Incorrect trend direction*
	ALF Cracking	83%	-0.40	76%	-0.73	

Figure 17. Correlation between BYET and Bottom up Cracking at the ALF facility of the FHWA [9].

7.2 Low Temperature – Single Edge Notched Bending

Thermal cracking remains a significant concern in the Upper Midwest and indeed in all regions where asphalt pavement is exposed to very low, rapidly falling pavement temperatures. The Bending Beam Rheometer (BBR) has traditionally been used to evaluate the low temperature properties of asphalt binder as they apply to the Superpave PG system. The Single-Edge Notch Bending (SENB) test has been developed by researchers at the University of Wisconsin-Madison to help provide a more robust low temperature specification system, especially for modified binders. Asphalt binders that can maintain low stiffness and high relaxation properties at low service temperatures have a higher resistance to thermal cracking; this was the logic behind the BBR Superpave specification. However, linear viscoelastic stiffness and relaxation do not indicate any fracture properties of an asphalt binder. Thermal cracking is directly related to both viscoelastic and fracture mechanics of asphalt binders [10]. The SENB test measures fracture properties of asphalt binders in a simple three point bending apparatus at low PG temperatures.

Marasteanu et al. conducted a study that attempted to correlate BBR, m-value, SENB parameters and low temperature mixture tests [11]. Results showed that there is not a clear correlation between SENB and BBR. Both SENB and BBR stiffness showed a relatively high correlation with the glass transition temperature of asphalt mixtures. The study validated the idea that both viscoelastic stiffness and fracture properties are necessary to characterize thermal cracking resistance of asphalt binders.

To conduct a SENB test, PAV aged asphalt binder is poured into a mold with the same dimensions to the BBR test, with an induced notch at the center of the span. Figure 18 provides a schematic of a typical SENB specimen [11].

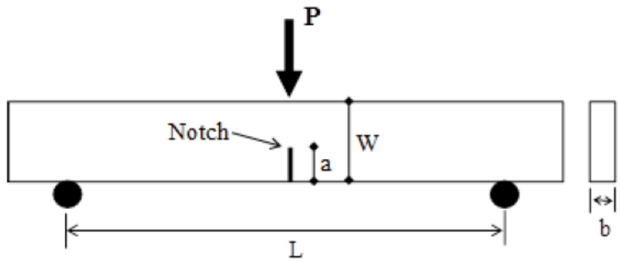


Figure 18. Schematic of SENB testing apparatus. Where P is the point load on the opposite side of the notch [10].

A point load is applied to the opposite side of the notch at a constant displacement rate in a bath at typical Superpave PG low temperatures (PG LT +10 ˚C). LVDTs and a load cell are used to measure the force and deflection until the sample fails. Three fracture parameters can be calculated from the force displacement data: fracture toughness, K_{IC}, fracture energy, G_f, and displacement at the maximum load. Each parameter can be calculated using the following equations:

$$\boldsymbol{K_{IC} = \frac{PL}{bW^{3/2}} * f(\frac{a}{W})}$$

$$\boldsymbol{G_f = \frac{W_f}{A_{lig}}}$$

Where, K_{IC} is the fracture toughness, G_f is the fracture energy, W_f is the work or area under the SENB force displacement curve, A_{lig} is the area of the ligament and all others are geometric constants shown in Figure **.**

A draft standard of the SENB test is currently under consideration by AASHTO for provisional implementation. The test has been evaluated using a range of LTPP section binders with success. The same WHRP study that outlined the use of the LAS test similarly used the SENB test to characterize thermal cracking in Wisconsin. Results indicated a strong correlation between both deflection at peak load and failure energy and thermal cracking PCI deduct values, which is a relative indicator of the thermal cracking present in the existing pavement. The results are shown in Figure 19.

In the current study it is recommended that the SENB fracture energy and displacement at maximum load be used in specification of binders for thermal cracking resistance. Figure 20 provides a preliminary specification using these parameters with binders from the recent WHRP study as well as the Transportation Pooled Fund study on Low Temperature Cracking sponsored by WisDOT. The binders supplied by the partner states will similarly be evaluated using the SENB procedure and results compared against the proposed specification and against mixture testing to be conducted in Phase 3 and ultimately compared to field performance. From there Figure 19 can be modified if needed to include the findings of this study.

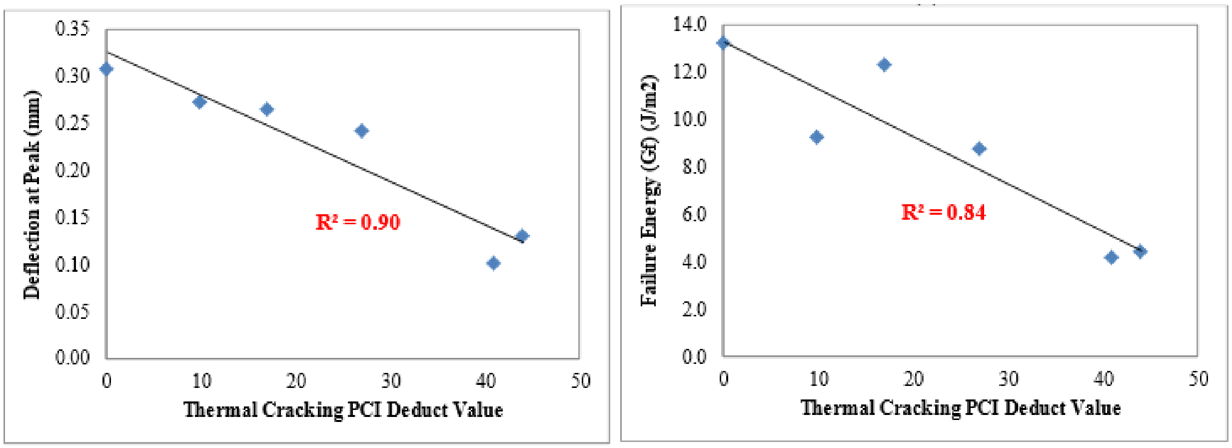


Figure 19. Correlation between deflection at peak load (left) and failure energy (right) with thermal cracking PCI deduct [8].

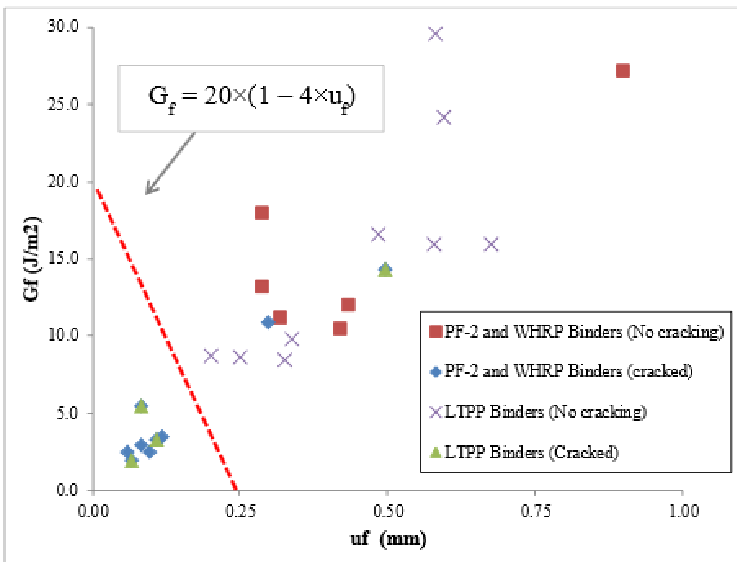


Figure 20. Proposed preliminary BBR-SENB specification based on RTFO aged conditions. Binders in the lower left triangle would fail the specification [8].

8. Summary of Significant Findings & Recommendations

In this phase of the study, asphalt binders supplied by the partner states were evaluated using current PG+ and recommended replacement PG+ test methods. In addition, binder testing results from the Western Cooperative Testing Group (WCTG) were included to make the analysis more robust.

Two supplementary PG+ test methods were also introduced to evaluate resistance to fatigue cracking and low temperature thermal cracking. Table 5 is a summary of the preliminary recommendations as they pertain to the respective PG+ test methods being replaced and/or considered for implementation. As mentioned in Section 2, the interpretation of the recommendations shown in Table 5 is objective based; the decision on which test method(s) states choose to implement depends on whether partner states wish to simply replace a test currently in use (for example T301 elastic recovery) or implement a test method that simulates pavement performance in a given temperature range. A more detailed description of each test method follows Table 5.

Table 5. Summary of Preliminary Replacement and Implementation Test Methods

Engineering Property or Distress	Partner State Objective	Preliminary Recommendation	Justification
High temperature elasticity (recovery of strain)/permanent deformation	Replace Phase Angle	ER DSR	Correlates well; apparent better differentiation between modified binders
	Replace ER T301	ER DSR	DSR-based; less material intensive; provides logical ranking
	Presence of Elastomer	MSCR %R	High temperature test; established standard; obtain more information from one test
	Address high temperature pavement deformation (rutting)	MSCR %R or MSCR Jnr	Damage characterization test; DSR-based; significant literature correlating test to field performance
Intermediate temperature elasticity (fatigue cracking)	Replace T51 ductility	BYET Strain at Max. Stress 5 °C	Logical ranking of modification types; good correlation between tests; no change in sample geometry; less material intensive; easier to run
	Replace toughness and tenacity	BYET yield energy at 25 °C.	No change in sample geometry; easier to run; widely available
	Address intermediate temperature (fatigue) cracking potential	*Linear Amplitude Sweep (LAS) Cycles to Failure	Damage characterization test; DSR-based; evidence of correlation to field performance in Wisconsin
		BYET yield energy at intermediate PG	Easy to run; same geometry as current G*sinδ; widely available; correlates well to full scale testing in ALF
Low temperature cracking potential	Address thermal cracking potential	*BBR-SENB	Damage characterization test; evidence of correlation to field performance in Wisconsin

*Denotes a supplementary test method for consideration

- **Phase Angle**: Findings suggest that the ER DSR procedure is a more robust method for quantifying the degree of elasticity (in terms of elastic recovery) for the modified binders used in this study. Although the ER DSR procedure is run at 25 °C and the phase angle at the high temperature PG, a strong linear correlation exits between the two methods. Based on the analysis, the ER DSR procedure can be used as a direct replacement for phase angle.

- **Elastic Recovery (AASHTO T301)**: The ER DSR procedure was found to correlate reasonably well with the AASHTO T301 elastic recovery results. All binders tested in this study that passed the phase angle requirement also passed the elastic recovery requirements. The ER DSR procedure can directly replace the T301 recovery procedure with appropriate modification to the elastic recovery limits. If ER DSR is used, a reduction in the required elastic recovery of approximately 30% is suggested to maintain the equivalent level of T301 elastic recovery.

- **MSCR Implementation**: The results in this report suggest that if performance at high temperature (i.e. reduction of permanent deformation) is desired, a high temperature test method that measures that performance should be used. The binders supplied for this study were tested in the current MSCR procedure with the following findings:

    - The MSCR %R cannot reliably be used to replace the T301 procedure; elasticity in the T301 procedure does not adequately predict high temperature elastic recovery using the MSCR. The decision should be made by each partner state whether the objective is simply to detect elastic response and ensure use of a specific amount and type of polymer, or implement a performance based test method like the MSCR.
    - All but three binders from both the pooled fund and WCTG data passed the requirement for indication of an elastomer. However the limits proposed for %R in the current TP70/T350 are not calibrated to any specific amount or type of polymer and they are not related to performance.
    - The WCTG and pooled fund binder data show that there is a strong linear correlation between Jnr and %R. These results suggest that current binders being produced will pass the MSCR specification with an equivalent or lower content of elastomeric polymer, and is most likely the result of modifying binders to meet another specification requirement that requires more polymer (i.e. T301).
    - For the current binder set, the linear correlation between Jnr and %R suggests that the concept of the MSCR procedure could therefore be satisfied by specifying either a maximum Jnr or a minimum %R for a given binder.
    - State specifications should be written to allow for high performing binders with respect to Jnr alone, and waive the % Jnr Diff parameter. For the binders supplied for this study, especially the “E” and “V” binders, the Jnr at 0.1 kPa is extremely low, even approaching the resolution limits of the DSR, so when the % Jnr Diff is calculated, a very large number results. It is therefore concluded that the % Jnr Diff parameter does not hold significant meaning for these binders. Without a solution for the 0.1 kPa Jnr values, and calibration against actual performance, this requirement cannot be considered ready for implementation.

- **Ductility at 4 °C (AASHTO T51)**: The DSR-based BYET test strain at maximum stress parameter appears to be a viable alternative to ductility at 4 °C. A strong linear correlation was found between T51 ductility and BYET strain at maximum stress consistent with the exiting literature, suggesting direct replacement of the T51 ductility test is possible with the BYET test. The BYET test also has several methodological advantages that make it an attractive replacement to T51, including consistent sample geometry during testing, lower sample quantity requirements, more accurate temperature control (and ability to run at any test temperature easily), and logical ranking of modified binders. It should be noted that this analysis does not necessarily directly address pavement performance at intermediate temperature.

- **Toughness and Tenacity (ASTM D 5801)**: The BYET test yield energy parameter shows a logical trend with binder toughness; as the yield energy of the binder increases, so too does the toughness. For the binder set tested in this study, the toughness and tenacity parameters were strongly linearly correlated, and as a result the tenacity parameter was also correlated with the BYET yield energy. Although the correlations are relatively poor, given the differences in strain rates between the two tests, unknown modification types and polymer loadings, and somewhat arbitrary means for defining the toughness and tenacity, the results are promising. Overall, the data suggests the toughness parameter can reasonably be estimated using the BYET yield energy at 25 °C. If the tenacity parameter remains of interest to the partner states, results from representative materials should be collected and analyzed to determine if the yield energy parameter from the BYET run at 25 °C can be used to estimate minimum specification limits. If the toughness and tenacity are linearly related, specifying one (i.e. toughness) parameter should satisfy the other.

- **Fatigue Cracking Resistance:** The Linear Amplitude Sweep (LAS) test (AASHTO TP101) is suggested for evaluation of fatigue cracking resistance of asphalt binders on the basis of recently published findings which show a high correlation between the LAS cycles to failure and actual fatigue cracking reported for field sections. A suggested preliminary specification is offered and will be evaluated after Phase 3 mixture testing and field evaluation of the present binders. In addition, BYET can be used as it showed very good correlation with fatigue cracking measured at the Accelerated Loading Facility ALF at FHWA. The limits for acceptance can be derived from correlations of that study.

- **Thermal Cracking Resistance:** The BBR-SENB test is suggested for evaluation of thermal cracking potential of asphalt binders based on the same study referenced for the LAS test. The SENB test is a more practical a repeatable alternative to the original DTT test from the original Superpave specification and more accurately characterizes modified binder thermal cracking potential relative to using stiffness and stress relaxation rate (m-value) alone. The two parameters of interest in the SENB test are the deformation at maximum load and the fracture energy. A preliminary specification is offered and will be evaluated again after Phase 3 testing is complete. The challenge in using this test is the availability of equipment. Partner states should consider setting up of SENB in their laboratories if there is sufficient interest.

9. Future Work Plan

Phase 3 of the work plan will address two primary research interests: (1) establish ruggedness and multi-laboratory precision of candidate replacement tests based on the decision of the partner states on which tests they wish to pursue and (2) establish correlation between candidate replacement tests and supplementary test methods and mixture performance.

Information regarding ruggedness and multi-laboratory precision work plans was originally provided in the Pooled Fund Progress Update presented to the partner states in May, 2015. At this time, partner states will need to make a decision on which tests they wish to pursue. Ruggedness and multi-laboratory precision will follow ASTM E 1169 and ASTM C 670, respectively. It will be the responsibility of the UW research team to conduct ruggedness testing on associated test methods. For the recommended replacement or supplementary methods presented in this report, this includes the ER DSR and BYET test methods. For this testing, four factors will be investigated: laboratory, DSR model, operator, and binder type. It is suggested that multi-laboratory precision be evaluated by the partner states each selecting three laboratories. Each laboratory will be sent four binder samples for evaluation by the candidate test method.

The second goal of Phase 3 is to establish correlations between candidate or supplementary binder test methods and mixture performance. Table 6 outlines the preliminary research plan with justification for this work. Partner states are expected to provide their input into which tests they would like to see run. Given limited material quantities, it is not feasible to run all tests on all mixtures.

Table 6. Preliminary Mixture Research Plan

Engineering Property or Distress	PG+ Binder Test Method	Replacement/Supplementary Binder Test	Candidate Mixture Performance Test with Justification
High temperature elasticity (recovery of strain)/permanent deformation	Phase Angle	ER DSR	**Primary:** Flow Number (FN) FN is well-researched in upper Midwest with data available from prior WHRP studies. Can also get dynamic modulus from same sample. **Secondary:** Hamburg Wheel Tracking Test is popular across U.S.; specification limits exist in many areas.
	ER T301	ER DSR	
	Presence of Elastomer	MSCR %R	
	Address high temperature pavement deformation (rutting)	MSCR %R or MSCR Jnr	
Intermediate temperature elasticity (fatigue cracking)	Replace T51 ductility	BYET Strain at Max. Stress 5 °C	**Primary:** Semi-Circular Bending Crack propagation test; easy to test; common across U.S. **Secondary:** Mixture fatigue; True fatigue testing
	Replace toughness and tenacity	BYET yield energy at 25 °C.	
	Address intermediate temperature (fatigue) cracking potential	*Linear Amplitude Sweep (LAS) Cycles to Failure And/or the BYET	
Low temperature cracking potential	Address thermal cracking potential	*BBR-SENB	**Primary:** Semi-Circular Bending See above **Secondary:** Asphalt Thermal Cracking Analysis Intuitive; relates to performance **Tertiary**: Disc-Shaped Compact Tension Popular, easy to test

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11. Appendix

Standard Method of Test for

Measuring Asphalt Binder Yield Energy and Elastic Recovery Using the Dynamic Shear Rheometer

AASHTO Designation: T XXX-15

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1. SCOPE

1.1. This test method covers the Binder Yield Energy test (BYET) for evaluation of asphalt binders' resistance to yield-type failure under monotonic constant shear-rate loading using the Dynamic Shear Rheometer (DSR). This test procedure can also be adapted for performing surrogate test procedure using the Dynamic Shear Rheometer (DSR) in place of the conventional ductility test (AASHTO T 51), and the Elastic Recovery test (ASTM D 6084). The test method can be used with unaged material and material aged using AASHTO T 240 (RTFOT) and/or AASHTO R 28 (PAV) to simulate the estimated aging for in-service asphalt pavements.

1.2. The values stated in SI units are to be regarded as the standard.

1.3. *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

---

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:
M 320, Standard Specification for Performance Graded Asphalt Binder
T 51, Standard Method of Test for Ductility of Bituminous Materials
T 240, Effect of Heat and Air on Rolling Film of Asphalt (Rolling Thin-Film Oven Test)
R 28, Standard Method of Test for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
T 300, Standard Method of Test for Force Ductility Test of Asphalt Material
T 315, Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)

2.2. *ASTM Standards*:
D 8, Standard Terminology Relating to Materials for Roads and Pavements
D 2872, Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)
D 6084, Standard Test Method for Elastic Recovery of Bituminous Materials by Ductilometer
D 6521, Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
D 7175, Standard Test Method for Determining the Rheological Properties of Asphalt Binder

Using a Dynamic Shear Rheometer

3. TERMINOLOGY

3.1 Definitions

Definitions of terms used in this practice may be found in Terminology D 8 determined from common English usage, or combinations of both.

4. SUMMARY OF TEST METHOD

4.1 The binder is prepared either at an unaged condition or using Test Method AASHTO T 240 (ASTM D 2872) (RTFOT) to represent short-term aging of asphalt pavements, or further aged using AASHTO R 28 (ASTM D 6521) to simulate long-term aging of asphalt pavements. The sample is prepared consistent with Test Method AASHTO T 315 (ASTM D 7175) (DSR) using the 8-mm parallel plate geometry with a 2-mm gap setting.

4.2 For the binder yield energy procedure the sample is tested in monotonic shear using a constant strain rate. The sample is continuously loaded until peak shear strength is achieved and the sample has yielded. The results can also be used to estimate ductility or forced ductility using the strain and stress at the peak of the yield curve.

4.3 For measuring the binder Elastic Recovery, after 2 minutes of monotonic shear using a constant strain rate, the sample is allowed to recover for 30 minutes before calculating the percent strain recovery.

5. SIGNIFICANCE AND USE

5.1. This method is intended to evaluate the performance of binders at intermediate temperatures in terms of resistance to yielding and in terms of elastic recovery. The "yield energy" of the sample can be used to identify the relative performance of different materials in terms of resistance to fatigue or extreme loading damage.

5.2 This method also provides a simple and more repeatable alternative to conducting the ductility test (AASHTO T51) and elastic recovery test (ASTM D 6084), using a standard Dynamic Shear Rheometer and a small sample size. The stress-strain response curve from the yield test, as well as the strain recovery from the elastic recovery test can be useful in identifying the presence of modifiers in binders and their potential benefits in improving ductile behavior of binders.

Note: It is to be noted that the relationship between ductility and elastic recovery to pavement performance in not known and there is no clear evidence that having higher ductility or higher elastic recovery improve pavement performance. The significance of this test is to replace the use of ductilometer used for ductility, forced ductility and elastic recovery with simpler and more repeatable tests in the DSR.

6. PROCEDURE

6.1. *Condition the asphalt binder in accordance with T 240 (RTFOT) for short-term performance, or follow with R 28 (PAV) for long-term performance.*

6.2. *Sample preparation* – The sample for the test is prepared following T 315 for 8-mm plates.

The temperature control also follows the T 315 requirements. This test may be performed on the same sample that was previously used to determine the rheological properties in the DSR on PAV residue as specified in M 320.

6.3. *Test protocol* – Two variations of the BYET test protocols are hereby described: Method A describes the procedure for measuring the binder yield energy, and a surrogate test procedure for the conventional ductility test (AASHTO T 51). Method B describes the Elastic Recovery test (ASTM D 6084).

6.3.1. *Method A: Binder Yield Energy and Ductility* – The prepared sample is tested at the desired test temperature at which a constant strain rate of 2.315 % s^{-1} is applied to the sample. Both stress (τ, Pa) and strain (γ, %) are recorded at a sampling rate of one data point every two seconds. The test is concluded once the material achieves 4167% strain (30 minutes).

For estimation of the Ductility the prepared sample is tested at the desired test temperature (usually 4 or 25°C) at which a constant strain rate of 2.315% s^{-1} is applied to the sample. Both stress (τ, Pa) and strain (γ, %) are recorded at a sampling rate of one data point every two seconds. The test can be concluded once the material achieves a strain of 2778% (1200 seconds) (Ref. 11.1). The strain at the peak stress can be used as a measure of the ductility.

6.3.2. *Method B: DSR- Elastic Recovery* - The prepared sample is tested at the desired test temperature (usually 25°C) at which a constant strain rate of 2.315 % s^{-1} is applied to the sample until a strain of 277.78 % is achieved. After this a recovery step is carried out by applying a 0.0 kPa shear stress to the sample for 30 minutes. Both stress (τ, Pa) and strain (γ, %) are recorded at a sampling rate of one data point every two seconds throughout the test (Ref. 11.2-11.3).

> Note 1 – The DSR strain rate is selected to be approximately equivalent to the 5 cm/min Ductilometer deformation rate used in AASHTO T 51 and ASTM D 6084 (Ref. 11.1-11.2)
>
> Note 2 – The applied strain rate can be proportionally adjusted if different equivalent Ductilometer deformation rates are desired (e.g. a rate of 0.463%s^{-1} would be equivalent to 1 cm/min in the ductilometer).
>
> Note 3 – The strain limit of 2778% is chosen to be equivalent to the maximum elongation of 100 cm of the ductility sample (with an effective length of 36 mm) in the ductilometer following T 51.

7. CALCULATION AND INTERPRETATION OF RESULTS

7.1. *Method A: Binder Yield Energy* – For the results of the binder yield energy test, the data should be analyzed as follows:

7.1.1. For the first data point, (τ_{i-1}, γ_{i-1}), the area, A_{i-1}, under the stress-strain curve at that point is calculated as

$$A_{i-1} = (\tau_{i-1})(\gamma_{i-1}) / 2$$

7.1.2. For subsequent data points, the area under the curve is calculated as the sum of the trapezoidal areas between each data point, known as the incremental energy, until the point of maximum shear stress (τ_{max}, $\gamma_{\tau,max}$). The total area is recorded as the Yield Energy (Figure 7.1), which is calculated using the formula below:

$$Yield\ Energy = A_{i-1} + \sum_{i=1}^{N} \left(\frac{\tau_i + \tau_{i-1}}{2}\right)(\gamma_i - \gamma_{i-1})$$

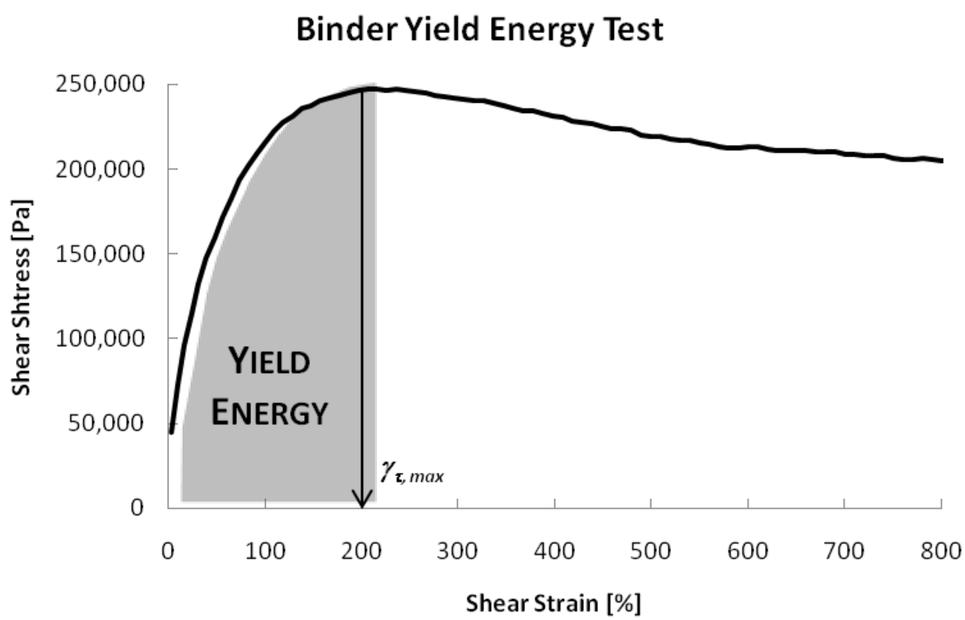

FIGURE 7.1 – Visual representation of Binder Yield Energy Test parameters.

7.2. *Method A* – For the ductility test, the data should be analyzed as follows:

7.2.1. Determine the maximum shear stress (τ_{max}) and the strain corresponding to the maximum shear stress, report this as ductility ($\gamma_{\tau,max}$).

Note 4 – It is possible to analyze data to achieve parameters comparable to those reported for the force ductility test (AASHTO T 300). For this the full shear stress vs. shear strain curve must be analyzed.

7.3. *Method B* - For the elastic recovery test, the data should be analyzed as follows:

7.3.1 Using the strain after 1800 seconds of recovery (γ_2) and the strain at the end of the loading step (γ_1 = 277.78%), the elastic recovery of the sample (Figure 7.2) is calculated using the formula below:

$$Elastic\ Recovery = \frac{\gamma_1 - \gamma_2}{\gamma_1} \times 100$$

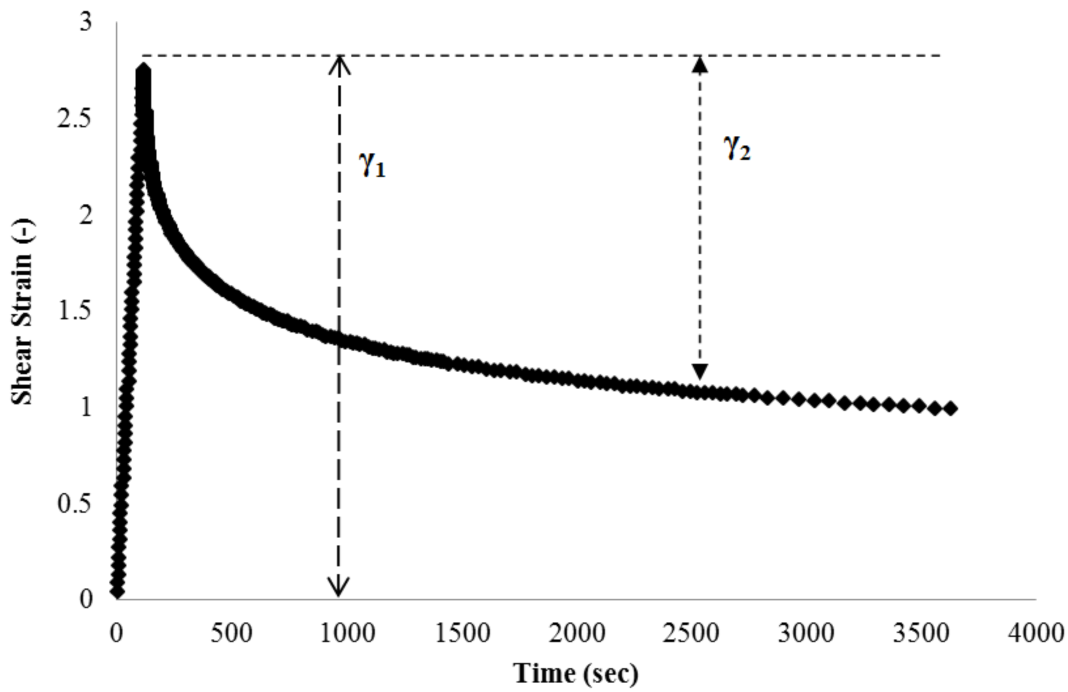

FIGURE 7.2 – Typical γ(t) curve for the elastic recovery test in the DSR

8. REPORT

8.1. *For Method A (Binder Yield Energy and Ductility) report the following, if known*:

8.1.1. Sample identification,

8.1.2. PG Grade and test temperature, nearest 0.1°C

8.1.3. Maximum shear stress, τ_{max}, kPa.

8.1.4. Shear strain at maximum shear stress, $\gamma_{\tau,max}$, %.

8.1.5. Yield Energy, MPa.

8.2. *For Method B (DSR- Elastic Recovery) report the following, if known*:

8.2.1. Sample identification,

8.2.2. PG Grade and test temperature, nearest 0.1°C

8.2.3. Shear strain at maximum shear stress, $\gamma_{\tau,max}$, %.

8.2.4. Elastic Recovery, %.

9. PRECISION AND BIAS

9.1. Two replicate tests are recommended for every material at each temperature tested.

9.2. Test precision and bias is to be determined upon results of inter-laboratory testing.

10. KEYWORDS

10.1. Asphalt binder, Yield, Energy, Dynamic Shear Rheometer, Elastic Recovery, Ductility, Ductilometer.

11. REFERENCES

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Standard Method of Test for

Estimating Damage Tolerance of Asphalt Binders Using the Linear Amplitude Sweep

AASHTO Designation: TP 101-12-UL

1. SCOPE

1.1. This test method covers how to determine asphalt binders' resistance to fatigue damage by means of cyclic loading employing systematically, linearly increasing load amplitudes. The amplitude sweep is conducted using the Dynamic Shear Rheometer at the intermediate pavement temperature determined from the performance grade (PG) of the asphalt binder. The test method can be used with binder aged using AASHTO T 240 (RTFOT) and AASHTO R 28 (PAV) to simulate the estimated aging for in-service asphalt pavements.

1.2. The values stated in SI units are to be regarded as the standard.

1.3. *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:
M 320, Standard Specification for Performance-Graded Asphalt Binder
T 240, Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test)
R 28, Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
T 315, Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)

2.2. *ASTM Standards*:
D 8, Standard Terminology Relating to Materials for Roads and Pavements
D 2872, Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)
D 6521, Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)
D 7175, Standard Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer

3. TERMINOLOGY

3.1 Definitions

3.1.1 Definitions of terms used in this practice may be found in Terminology D 8, determined from common English usage, or combinations of both.

4. SUMMARY OF TEST METHOD

4.1 Asphalt binder is first aged using Test Method AASHTO T 240 (ASTM D 2872) (RTFOT) to represent short-term aging of asphalt pavements, or the material may be further aged using AASHTO R 28 (ASTM D 6521-08) prior to testing in order to simulate long-term aging of asphalt pavements. A sample is prepared consistent with Test Method AASHTO T 315 (ASTM D 7175-05) (DSR) using the 8-mm parallel plate geometry with a 2-mm gap setting. The sample is tested in shear using a frequency sweep to determine rheological properties. The sample is then tested using a series of oscillatory load cycles at systematically increasing amplitudes at a constant frequency to cause accelerated fatigue damage. To quantify damage tolerance a rigorous viscoelastic continuum damage approach is used to calculate fatigue resistance from rheological properties and amplitude sweep results.

5. SIGNIFICANCE AND USE

5.1. This method is intended to evaluate the ability of an asphalt binder to resist fatigue damage by employing cyclic loading at increasing amplitudes in order to accelerate damage. The characteristics of the rate of damage accumulation in the material can be used to indicate the fatigue performance of the asphalt binder given pavement structural conditions and/or expected amount of traffic loading using predictive modeling techniques.

6. APPARATUS

6.1. Use the apparatus as specified in T 315.

7. PROCEDURE

7.1. Condition the asphalt binder in accordance with AASHTO T 240 (RTFOT) for short-term performance, or condition the asphalt binder in accordance with AASHTO T 240 (RTFOT) followed by AASHTO R 28 (PAV) for long-term performance.

7.2. *Sample preparation* – The sample for the Linear Amplitude Sweep is prepared following AASHTO T 315 for 8-mm plates. The temperature control also follows the AASHTO T 315 requirements.

> Note 1: In accordance to AASHTO T 315 provisions, it is suggested that spindle and plate temperature be raised to 64°C or higher before insertion of the asphalt sample to ensure sufficient adhesion is achieved, especially for highly modified and/or aged asphalt binders. Such provisions have been shown to prevent delamination in the majority of binders tested.

7.2.1. This test may be performed on the same sample that was previously used to determine the rheological properties in the DSR on PAV residue as specified in M 320.

7.3. *Test protocol* – Two types of testing are performed in succession. The first, (a frequency sweep), is designed to obtain information on the rheological properties, and the second (an amplitude sweep), is intended to measure the damage characteristics of the material.

7.3.1 *Determination of "alpha" parameter* – In order to perform the damage analysis, information regarding the undamaged material properties (represented by the parameter α) must be determined. The frequency sweep procedure outlined in Section 6.3.1.1 is used.

7.3.1.1 *Frequency sweep* –Frequency sweep test data is used to determine the damage analysis "alpha" parameter. The frequency sweep test is performed at the selected temperature, and applies oscillatory shear loading at constant amplitude over a range of loading frequencies. For this test method, the frequency sweep test is selected from the DSR manufacturer's controller software, employing an applied load of 0.1% strain over a range of frequencies from 0.2 – 30 Hz. Data is sampled at the following 12 unique frequencies (all in Hz):

0.2	0.4	0.6	0.8	1.0	2.0	4.0	6.0	8.0	10	20	30

Complex shear modulus [|*G**|, Pa] and phase angle [δ, degrees] are recorded at each frequency, as shown below.

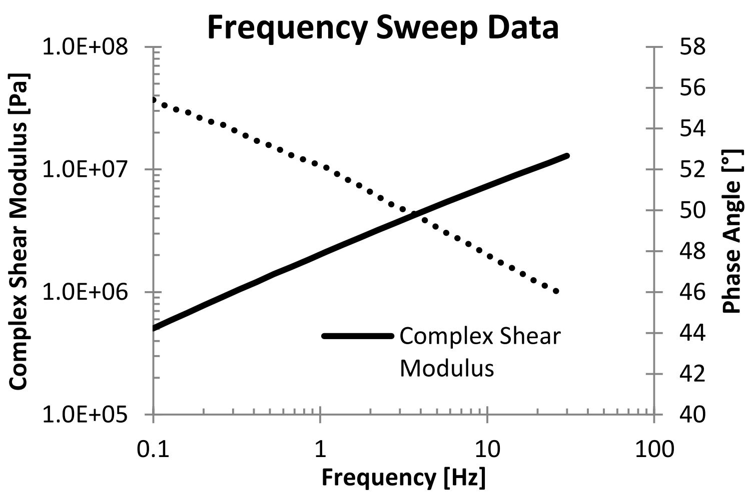

Figure 1– Example output from frequency sweep test.

7.3.2. *Amplitude sweep* – The second test is run at the selected temperature using oscillatory shear in strain-control mode at a frequency of 10 Hz. The loading scheme consists of a continuous oscillatory strain sweep. Strain is increased linearly from 0.1 to 30% over the course of 3,100 cycles of loading for a total test time of 310 sec. Peak shear strain and peak shear stress are recorded every 10 load cycles (1 sec), along with phase angle [δ, degrees] and Complex shear modulus [|G*|, Pa].

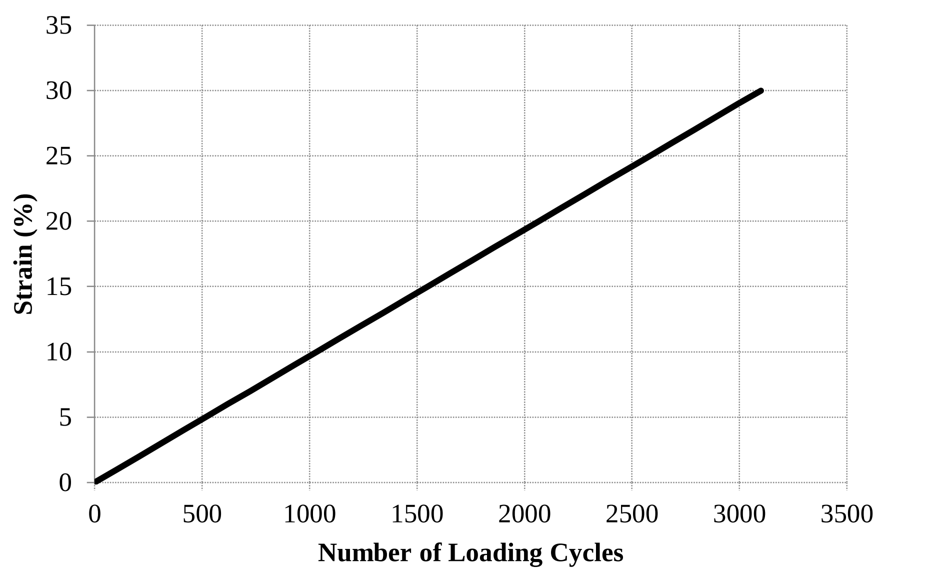

Figure 2– Loading scheme for amplitude sweep test

8. CALCULATION AND INTERPRETATION OF RESULTS

8.1 In order to determine the parameter α from frequency sweep test data, the following calculations are performed.

8.1.1. First, data for the dynamic modulus [|G*|(ω)] and phase angle [δ(ω)] for each frequency is converted to storage modulus, G'(ω):

$G'(\omega) = |G^*|(\omega) \times \cos\delta(\omega)$

8.1.2. A best-fit straight line is applied to a plot with log ω on the horizontal axis and log G'(ω) on the vertical axis using the form:

$\log G'(\omega) = m(\log\omega) + b$

8.1.3. The value obtained for m is recorded and the value of α is obtained by performing the following transformation:

$\alpha = 1 / m$

8.1.4. For the results of the amplitude sweep test, the data is analyzed as follows:

8.1.5. The damage accumulation in the specimen is calculated using the following summation (Kim et al, 2006):

$$D(t) \cong \sum_{i=1}^{N}[\pi\gamma_0^2(C_{i-1} - C_i)]^{\frac{\alpha}{1+\alpha}}(t_i - t_{i-1})^{\frac{1}{1+\alpha}}$$

where:

$C(t) = \frac{|G^*|\cdot sin\delta(t)}{|G^*|\cdot sin\delta_{initial}}$ which is |G*|·sinδ at time t divided by the initial "undamaged" value of |G*|·sinδ.

γ_0 = applied strain for a given data point, percent

|G*| = Complex shear modulus, MPa

α = value reported in Section 8.1.4

t = testing time, seconds

Note 2: The initial "undamaged" value of |G*|·sinδ is the second data point, as the first point after change of material condition from rest differs from the undamaged modulus of material at the target loading frequency.

8.1.6. Summation of damage accumulation begins with the first data point for the 1.0% applied strain interval. The incremental value of $D(t)$ at each subsequent point is added to the value of $D(t)$ from the previous point. This is performed up until the final data point from the test at 30 percent applied strain.

8.1.7. For each data point at a given time t, values of C(t) and D(t) are recorded (it is assumed that C at D(0) is equal to one, and D(0) = 0). The relationship between C(t) and D(t) can then be fitted to the following power law:

$C(t) = C_0 - C_1\,(D(t))^{C2}$

where:
C_0 = 1, the initial value of C,
C_1 and C_2 = curve-fit coefficients derived through linearization of the power law in the form shown below as suggested by Hintz et al. (2011):

$$\log(C_0 - C(t)) = \log(C_1) + C_2 \cdot \log(D(t))$$

Using the above equation, C_1 is calculated as the anti-log of the intercept and C_2 is calculated as the slope of the line formed as $\log(C_0 – C(t))$ versus $\log(D(t))$. For calculation of both C_1 and C_2, data corresponding to damages less than 10 are ignored.

8.1.8. The value of $D(t)$ at failure, D_f, is defined as the $D(t)$ which corresponds to a 35 percent reduction in initial $|G^*|\cdot\sin\delta$ (i.e., $C_0 - C(t) = 0.35$). The calculation is as follows:

$$D_f = \left(\frac{0.35}{C_1}\right)^{1/C_2}$$

8.1.9. The following parameters (A_{35} and B) for the binder fatigue performance model can now be calculated and recorded as follows:

$$A_{35} = \frac{f(D_f)^k}{k(\pi I_D C_1 C_2)^\alpha}$$

Where f = loading frequency (10 Hz).
$k = 1 + (1 - C_2)\alpha$
and
$B = 2\alpha$.

8.1.10 The binder fatigue performance parameter N_f can now be calculated as follows:

$$N_f = A_{35}(\gamma_{max})^{-B}$$

Where γ_{max} = the maximum expected binder strain for a given pavement structure, %.

9. REPORT

9.1. *Report the following, if known*:

9.1.1. Sample identification,

9.1.2. PG grade and test temperature, nearest 0.1°C

9.1.3. Fatigue model parameters A_{35} and B, four significant figures.

9.1.4. Binder fatigue performance parameter N_f, nearest whole number.

10. PRECISION AND BIAS

10.1. To be determined upon results of inter-laboratory testing.

11. KEYWORDS

11.1. Asphalt binder, viscoelastic continuum damage (VECD), fatigue, Performance Grading.

12. REFERENCES

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12.2. Hintz, C., Velasquez, R., Johnson, C., and H. Bahia. Modification and Validation of the Linear Amplitude Sweep Test for Binder Fatigue Specification, In *Transportation Research Record: Journal of the Transportation Research Board*, Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 99-106.

APPENDIX

(Non-mandatory Information)

X1. SAMPLE CALCULATIONS

X1.1. Example data from the amplitude sweep test is given in Table X1.1.

Table X1.1 – Example data output from amplitude sweep test

Testing Time	Shear Stress	Shear Strain	$\|G^*\|$	Phase Angle	$\|G^*\| \cdot \sin \delta$
[sec]	[MPa]	[%]	[MPa]	[°]	[MPa]
34	0.212	1.996	10.646	49.18	8.057
35	0.212	2.001	10.619	49.22	8.041
36	0.212	2.003	10.595	49.26	8.028
37	0.211	2.003	10.574	49.29	8.016
38	0.211	2.004	10.555	49.32	8.005
39	0.211	2.003	10.539	49.34	7.995
40	0.210	2.003	10.524	49.37	7.987

X1.2. The following values have already been assumed:

$D(33) = 10.77$

$\alpha = 2.58$

$I_D = 8.345$ MPa

$|G^*| \cdot \sin \delta_{t=33} = 8.075$ MPa

X1.3. *Sample calculations:*

X1.3.1. To calculate the accumulation of damage from $t = 33$ sec to $t = 34$ sec,

$$D(34) = D(33) + [\pi I_D \gamma_0^2 (|G^*| \sin \delta_{i-1} - |G^*| \sin \delta_i)]^{\frac{\alpha}{1+\alpha}} (t_i - t_{i-1})^{\frac{1}{1+\alpha}}$$

$$D(34) = D(33) + [\pi(8.345)(1.996)^2(8.075 - 8.057)]^{\frac{2.58}{1+2.58}} (34 - 33)^{\frac{1}{1+2.58}}$$

$$D(34) = 12.36$$

X1.3.2. This procedure is repeated, giving the following results shown in Table X1.2.

Table X1.2 – Example data output and damage calculation from amplitude sweep test

Testing Time	Shear Stress	Shear Strain	Complex Modulus	Phase Angle	$\|G^*\| \cdot \sin \delta$	$D(t)$
[sec]	[MPa]	[%]	[MPa]	[°]	[MPa]	
34	0.212	1.996	10.646	49.18	8.057	12.36
35	0.212	2.001	10.619	49.22	8.041	13.79
36	0.212	2.003	10.595	49.26	8.028	15.06
37	0.211	2.003	10.574	49.29	8.016	16.26
38	0.211	2.004	10.555	49.32	8.005	17.35
39	0.211	2.003	10.539	49.34	7.995	18.40
40	0.210	2.003	10.524	49.37	7.987	19.26

X2.1 The following example plots may be useful in visualizing the results:

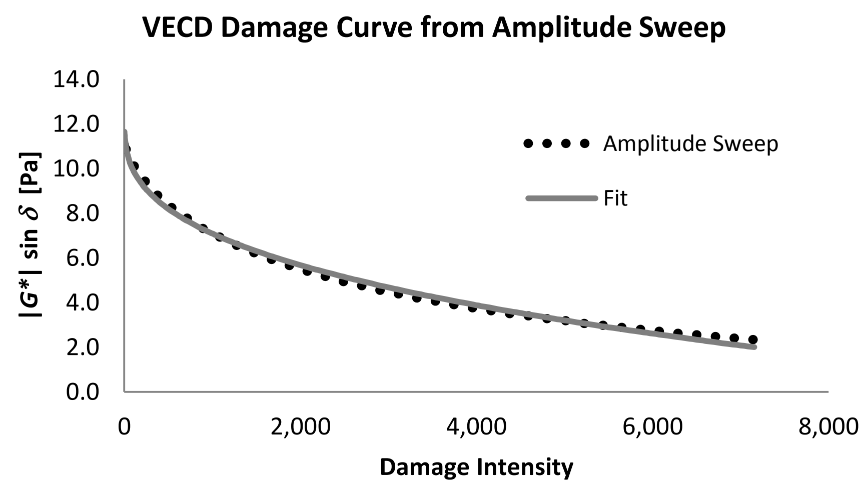

FIGURE X2.1 – Example $|G^*| \cdot \sin \delta$ versus damage plot with curve-fit from Section 7.2.

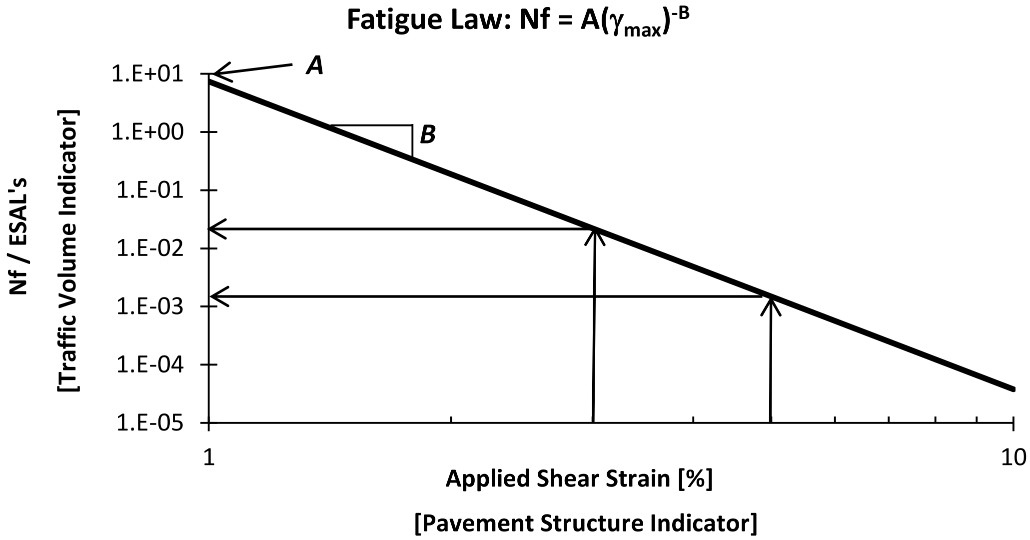

FIGURE X2.2 – Plot of fatigue parameter N_f (normalized to 1 million ESAL's) versus applied binder shear strain on a log-log scale. Allowable fatigue life can be determined for given strain amplitudes, as shown by the arrows.

Standard Method of Test for

Determining the Fracture Properties of Asphalt Binders Using the Single Edge Notched Bending Test

AASHTO Designation: T XXX-13

1. SCOPE

1.1. This test method covers how to determine the fracture resistance properties of asphalt binders at low temperatures by means of loading to failure a notched asphalt binder beam in three point bending using displacement controlled conditions. The Single Edge Notched Bending Test (i.e., SENB) is conducted using a simple modification of the loading mechanism of the Bending Beam Rheometer at pavement minimum service temperatures. The test method can be used with asphalt binder aged using AASHTO T 240 (RTFOT) and AASHTO R 28 (PAV) to simulate mixing and in-service aging conditions.

1.2. The values stated in SI units are to be regarded as the standard.

1.3. *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

M 320, Standard Specification for Performance-Graded Asphalt Binder

T 240, Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test)

R 28, Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)

T 313, Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)

PP 42: Determination of Low-Temperature Performance Grade (PG) of Asphalt Binders

2.2. *ASTM Standards*:

D 8, Standard Terminology Relating to Materials for Roads and Pavements

D 2872, Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)

D 6521, Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)

D 6648, Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)

D 6816, Standard Practice for Determining Low-Temperature Performance Grade (PG) of Asphalt Binders

E 399, Standard Test Method for Linear-Elastic Plane-Strain Fracture Toughness KIC of Metallic Materials

3. TERMINOLOGY

3.1 Definitions of terms used in this practice may be found in Terminology D 8, determined from common English usage, or combinations of both.

4. SUMMARY OF TEST METHOD

4.1 Asphalt binder is first aged using Test Method AASHTO T 240 (ASTM D 2872) (RTFOT) to represent short-term aging, or the material may be further aged using AASHTO R 28 (ASTM D 6521-08) prior to testing in order to simulate long-term aging of asphalt pavements. A sample is prepared consistent with Test Method AASHTO T 313 (ASTM D 6648) (BBR) with modification of the mold to allow the placement of a notch in the middle of the beam. The sample is tested to failure in three point bending configuration and displacement-controlled mode using a modified Bending Beam Rheometer. Failure energy (G_f) and fracture deflection (u_f) are calculated from the force-displacement response of the asphalt binder notched beam.

5. SIGNIFICANCE AND USE

5.1 This method is intended to evaluate the ability of an asphalt binder to resist low temperature cracking through a three point bending test of a notched beam. The fracture resistance properties of asphalt binders can be used to indicate thermal cracking performance and to compliment AASHTO M320.

6. APPARATUS

6.1 All equipment is as described in AASHTO T 313 with modifications to the loading system, beam support, mold assembly, and air supply.

6.1.1 *Mold Assembly* – The aluminum molds used in AASHTO T 313 should be modified to include notches on the side beam and a pin-hole assembly to keep the side beams and end pieces precisely aligned. This alignment is critical to insure proper notch location under the loading shaft. Figure 6.1 shows the modified mold assembly.

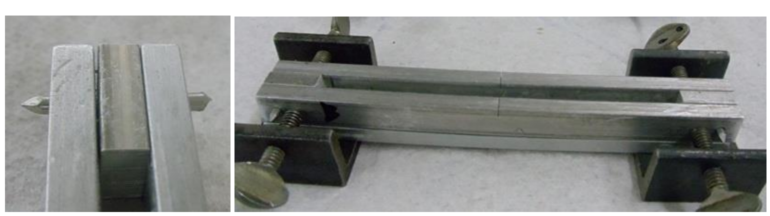

Figure 6.1. Photograph. Modified mold assembly for preparation of BBR-SENB beams.

6.1.2 *Loading Device* – The Bending Beam Rheometer loading setup should be equipped with a step motor enabling the movement of the loading shaft at a constant and adjustable rate. The motor should be capable of applying rates of displacement between 0.0025 to 0.1 mm/sec. Figure 6.2 shows the motor setup and design.

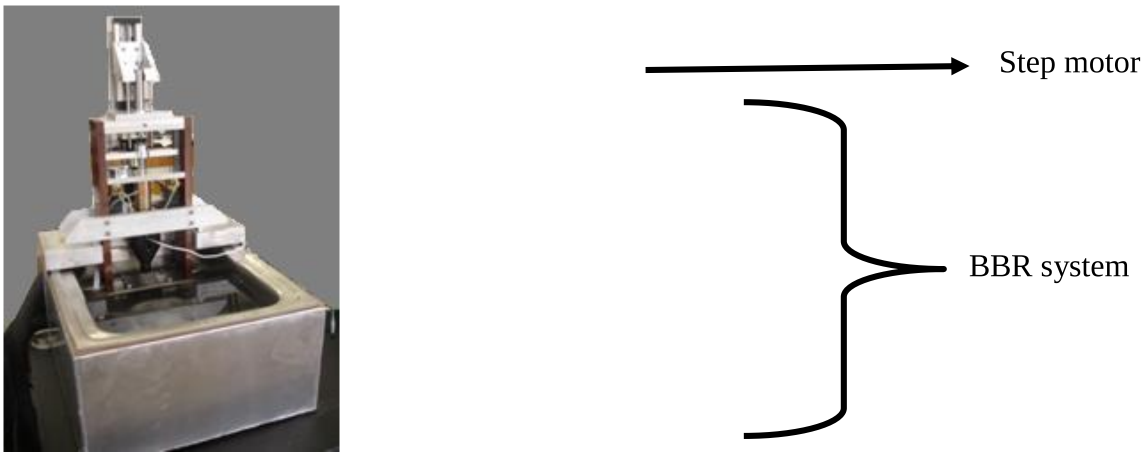

Figure 6.2. Photograph. Loading device in BBR-SENB

6.1.3 *Beam Supports* – To ensure the proper alignment of the notch under the loading shaft, the beam support is modified to hold a beam with a width of 6.3 mm and an alignment bracket. Figure 6.3 shows the support and bracket design.

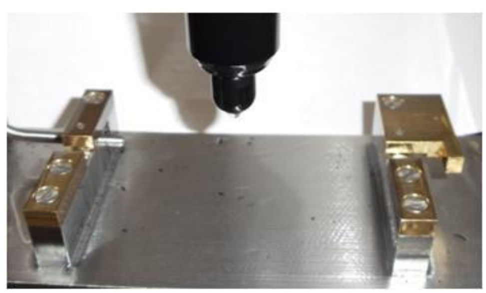

Figure 6.3. Photograph. Beam supports to ensure notch alignment

6.1.4 *Air Supply* – The hose supplying pressurized air to the loading shaft air bearing should be equipped with a pressure regulator to allow adjusting air pressure between 138 Pa and 310 Pa. Air bearing pressure is temporarily changed to 138 Pa only while zeroing the displacement gauge, as described in 7.3.2. Air bearing pressure should be set to 310 Pa at all other times.

7. PROCEDURE

7.1 *Condition the asphalt binder in accordance with T 240 (RTFOT) for short-term aging condition, or follow with R 28 (PAV) for long-term aging condition.*

7.2 *Sample preparation* – The sample for the Single Edge Notched Bending (BBR-SENB) test is prepared following AASHTO T 313 using the modified aluminum molds described in 6.1.1.

7.2.1 Assemble modified molds following AASHTO T 313.

7.2.2 Two rectangular strips, each 25 mm by 12.5 mm, are cut out of transparency paper using scissors. These rectangles are placed into the mold notches such that the 12.5 mm side is centered in the molds and the 25 mm side is vertical to the molds.

7.2.3 Place rectangular plastic strips in mold assembly before pouring the molten asphalt. Asphalt may be poured continuously from one end of the mold, in accordance to AASHTO T 313. The upper half of the strips may be cut off after the sample has cooled to room temperature. The clipping is done to ensure that the strips do not adhere to each other due to the excess of asphalt binder in between.

7.2.4 Mold assembly should be allowed to cool down before demolding. Cooling periods and conditions follow the T 313 requirements.

7.2.5 Trimming of excess asphalt on mold should be done in accordance to AASHTO T313 with the difference that trimming is done from each end of the mold toward the rectangle strips in the notches at the midpoint. Care should be taken to remove excess binder from around the strips.

7.2.6 Sample demolding should be performed with utmost care as to not damage or disturb the notch. Rotation of side beams should be minimized and all effort should be made to slide aluminum side-beams off sample perpendicular to the notch. Molds can be placed for up to 10 minutes in a freezer to facilitate demolding.

7.2.7 Samples should be conditioned in the device fluid bath at the test temperature. Conditioning should be performed in accordance to AASHTO T 313.

7.3. *Test protocol* – The Single Edged Notched Bending test is a three point bending test performed using modified BBR device as described in 6.1.2.

7.3.1 Calibration procedure for load cell and displacement gauge follows AASHTO T 313.

7.3.2 Position the thick calibration beam on its narrower side on the beam supports and in the alignment bracket. Determine the point of zero displacement by finding the point of contact between the loading shaft and the beam. The air bearing pressure is set to 138 Pa, thus letting the shaft fall under its own weight and make contact with the beam. The displacement is zeroed. After zeroing, the air pressure should be set back to 310 Pa.

7.3.3 After calibration of sensors, the asphalt beam should be carefully placed onto the supports using a pair of tongs. The beam should be positioned such that the notch is facing downward. The notch is centered under the load by ensuring that the beam is in contact with the alignment bracket.

7.3.4 Position loading shaft near the zero point above the beam. After loading tip is in contact with the beam, apply a constant shaft displacement rate of 0.01 mm/sec until fracture occurs and force values are close to zero.

7.3.4.1 A shaft displacement rate of 0.1 mm/sec or higher may be used to position loading shaft before and after testing.

7.2.4.2 Some material such as modified binders may demonstrate extensive post peak load bearing. When testing such samples loading should be continued after loading peak long enough to either reach ultimate failure or the maximum loading shaft stroke.

7.3.5 The fractured surfaces of the beams should be inspected after testing to insure that the crack surface is planar and crack propagation has been directly through the notch. Results from beams not conforming to the aforementioned criteria must be discarded.

8. CALCULATION AND INTERPRETATION OF RESULTS

8.1 In order to determine the failure energy, G_f, and deflection at fracture (u_f) from the test data, the following calculations are performed.

8.1.1 *Force-Displacement Curve* – The force and displacement readings from the test are used to generate a force-displacement curve as shown in Figure 8.1. Required precision is 50 mN for the force and 0.01 mm for the displacement. Higher precision in acquired data may cause excessive noise that may be eliminated using consistent moving average or other noise removal filters for both the force and displacement readings.

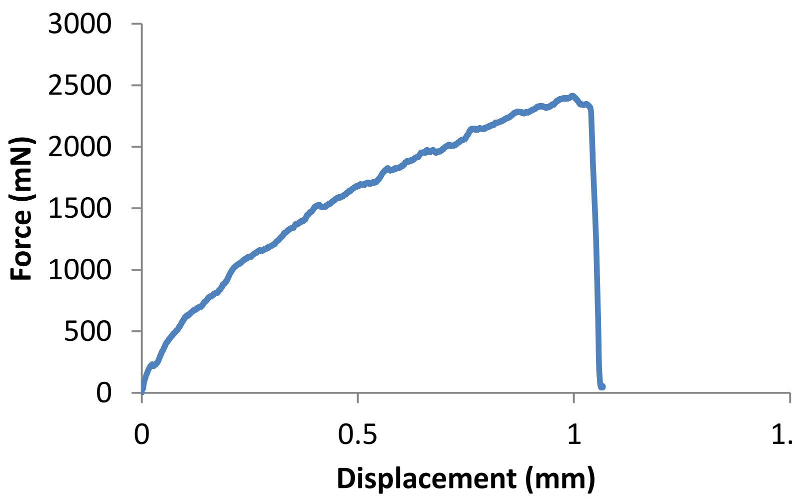

Figure 8.1. Graph. BBR-SENB typical force-displacement curve

8.1.2 *Calculation of G_f* – The failure energy parameter, G_f, is calculated from the division of the area under the force-displacement curve by the ligament area.

8.1.2.1 The ligament area or cracked surface is the effective rectangular area of the notched beam.

8.1.2.2 The area under the force-displacement curve can be numerically calculated with high accuracy using the trapezoidal method.

8.1.3 *Estimation of u_f* – The deflection at fracture, u_f, is estimated as the deflection corresponding to the maximum load.

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9. REPORT

9.1. *Report the following*:

9.1.1 Sample identification,

9.1.2 Performance grade of the asphalt binder, loading rate, and test temperature (nearest 0.1°C)

9.1.3 Fracture parameters G_f [N.m/m^2] and u_f [mm], to two significant figures.

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10. PRECISION AND BIAS

10.1 A minimum of 3 replicates should be tested for every asphalt binder and condition. If coefficient of variation for failure energy (G_f) and deflection at fracture (u_f) is greater than 15%, an extra replicate should be tested. The replicate with significantly different deflection at fracture may be discarded.

10.2 Test precision and bias is to be determined upon results of inter-laboratory testing.

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11. KEYWORDS

11.1 Asphalt binder, failure energy, deflection at fracture, low temperature cracking, three point bending, performance grading, SENB, BBR, notch.