RHEOLOGICAL AND MECHANICAL PROPERTIES OF BLENDED ASPHALTS CONTAINING RECYCLED ASPHALT PAVEMENT BINDERS

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ABSTRACT

This paper presents experimental investigation results on rheological and mechanical properties of asphalt binder containing recycled asphalt pavement (RAP). It also includes evaluation of resistance characteristics against permanent deformation, fatigue and low temperature cracking. In the present study, two base asphalt binders (AC-10 or PG 58-28 and AC-20 or PG 64-22) typically used in Rhode Island were blended with different amounts of RAP binders obtained from two sources, i.e., 0, 10, 20, 30, 40, 50, 75, and 100 percent based on the total weight of blended asphalt binder. The Dynamic Shear Rheometer (DSR) was used to evaluate the blended asphalt binders at high temperatures, i.e., 52, 58, 64, 70, and 76 C and intermediate temperatures, i.e., 19, 22, 25, 28, and 31 C. A good linear relationship between log-log rheological properties and the amount of RAP binders was obtained from this study. It was found that the addition of RAP binder generally increases the resistance against permanent deformation. It was observed that an increase in RAP binder content causes an increase in G*sinδ. Therefore, the Superpave criteria of G*sinδ may dictate the maximum amount of RAP addition not to have fatigue cracking.

The Bending Beam Rheometer (BBR) test was performed to evaluate the resistance characteristics of blended asphalt binder against thermal cracking at low temperatures, i.e., -6, -12, -18, and -24 C. It was observed that the creep stiffness increased and the m-value decreased for all temperatures as the amount of RAP content increased. It appears that the addition of RAP did not enhance the binder’s resistance characteristic against low temperature cracking. Furthermore, the Superpave criteria of s and m may dictate the maximum amount of RAP not to have thermal cracking.

For the quasi-static loading experiments, it was observed that the compressive strength and the stiffness were increased and the ductility was reduced as the amount of RAP binder increased. The fracture toughness was evaluated at different loading rates in order to study the rate dependency of the binder. It was observed that the fracture toughness values are increased gradually and then stabilized for the rates of loading over 152.4 mm per minute. Subsequently, a series of experiments was conducted at this rate to evaluate the fracture toughness of asphalt binder with different amounts of RAP binder, both at 22 and 0 C. Experiment results at 22 C indicated an increasing trend of fracture toughness as a function of RAP binder content. A brittle fast fracture occurred at 0 C, and interestingly RAP binder had no significant effect on the performance of the blended asphalt. The dynamic response of blended asphalt binder was evaluated using the Split Hopkinson Pressure Bar (SHPB) apparatus. The dynamic flow stress was found to increase with RAP binder content much like the variation in compressive strength under quasi-static conditions. The dynamic flow stress was not affected by the amount of RAP binder at low temperature.

Key Words: Asphalt Binder, Recycled Asphalt Pavement, Dynamic Shear Rheometer, Bending Beam Rheometer, Rheological Properties, Mechanical Properties, Compressive Strength, Fracture Toughness, Split Hopkinson Pressure Bar.
INTRODUCTION

For over a century, paved roadways have been constructed using asphalt concrete mixtures in Rhode Island as well as across the United States. However, a major problem still exists in asphalt pavement involving premature distresses and failures, e.g., permanent deformation, fatigue cracking, and low temperature cracking. Since the early 1970s, many highway agencies have recycled old pavements in the overlay or major reconstruction of highways. Recently, the use of Recycled Asphalt Pavement (RAP) has significantly increased due to the protection of the environment, economy of construction/rehabilitation procedures, and the conservation of materials. However, the evaluation of RAP performance has not been well established.

The Strategic Highway Research Program (SHRP) developed a performance-based specification for asphalt binder accompanied by a new system and testing procedures, as a component of "Superpave™," which stands for Superior Performing Asphalt Pavement (1). Six types of new binder testing equipment were recommended to measure the physical and/or rheological properties of modified as well as unmodified asphalt binders that can be related directly to field performance by engineering principles. Among the equipment, the Dynamic Shear Rheometer (DSR) was chosen to evaluate permanent deformation and fatigue cracking resistance characteristics by measuring the properties of asphalt binder at high and intermediate temperatures, respectively. The Bending Beam Rheometer (BBR) was chosen to evaluate the low temperature properties of the asphalt binders. Yet, Superpave did not include a comprehensive RAP mixture evaluation system. Therefore, an attempt was made to evaluate rheological properties of asphalt binder containing RAP utilizing the Superpave tool and to help engineers gain insight into the use of RAP in asphalt pavement.

Since SHRP performed a limited investigation related to fracture and crack propagation within asphalt mixtures, further mechanical characterization of asphalt binder and mixture is warranted. In the present study, single notched specimens were used for the fracture toughness testing. Besides, split Hopkinson pressure bar (SHPB) equipment was utilized to characterize the dynamic constitutive behavior.

The objective of the present study was to investigate the relationship between rheological and mechanical properties and RAP binder percentage in blended asphalt for evaluation of the permanent deformation, fatigue, and low temperature cracking resistance characteristics.

DYNAMIC SHEAR PROPERTIES OF ASPHALT CONTAINING RAP BINDERS FROM DIFFERENT SOURCES

Experimental Plan

To examine the source variation, the RAPs were procured from two hot-mix asphalt (HMA) plants (C and L) in Rhode Island. Two base asphalts, i.e., AC-10 (or PG 58-22) and AC-20 (or PG 64-22) typically used in Rhode Island, were blended with different amount of RAP.
binders, i.e., 0, 10, 20, 30, 40, 50, 75 and 100 percent based on the total weight of blended asphalt binders. DSR tests were performed in accordance with the procedure of American Association of State Highway and Transportation Officials (AASHTO) TP5 at Superpave high and intermediate temperatures, i.e., 52, 58, 64, 70, 76, 19, 22, 25, 28, 31°C, respectively (2). The overall experimental design is summarized in Table 1, and the experiment was conducted in the year of 1997 (3).

TABLE 1. Experimental Design to Evaluate Dynamic Shear Properties of Asphalt Containing RAP Binders from two Different Sources in 1997.

<table>
<thead>
<tr>
<th>Tests</th>
<th>RAP, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Absolute viscosity</td>
<td>4</td>
</tr>
<tr>
<td>DSR, Unaged</td>
<td>4</td>
</tr>
<tr>
<td>DSR, RTFO</td>
<td>4</td>
</tr>
<tr>
<td>DSR, PAV</td>
<td>4</td>
</tr>
</tbody>
</table>

Note: 1. 2 (RAP source) x 2 (base asphalt) = 4
2. DSR tests were performed at the Superpave high and intermediate temperatures, i.e., 52, 58, 64, 70, 76°C and 19, 22, 25, 28, 31°C, respectively.
3. Two replicate samples were tested.

Sample Preparation

RAP binders were recovered in accordance with the procedure of AASHTO T170-93 at the Rhode Island Department of Transportation (RIDOT) laboratory (4). It may be noted that detailed information on the two RAP sources was not available, e.g., virgin asphalt properties, age, etc. The base asphalts were procured from a Rhode Island distributor, H. Virgin and RAP asphalt binders were heated to 135 and 160°C, respectively. The base asphalt was blended with the required amount of RAP binder using a mechanical stirrer.

Laboratory Testing

After absolute and rotational viscosity testing, the base asphalt and blended asphalts with different amount of RAP binder were tested with the DSR. The DSR was used to characterize
the viscous and elastic behavior of asphalt binders. The test measures the complex shear modulus (G*) and phase angle (δ) of the asphalt binder at high and intermediate temperatures when the dynamic (oscillatory) shear is applied to the sample using parallel plate test geometry (1). G* is a measure of the total resistance of a material to deforming when repeatedly sheared. The δ is an indicator of the relative amounts of elastic (recoverable) and viscous (non-recoverable) deformation. The DSR tests were performed for unaged as well as aged binder aged with the Rolling Thin Film Oven (RTFO) and the Pressure Aging Vessel (PAV).

Initial DSR Test Results in the Year of 1997

A constant stress mode was used for the DSR tests in the present study. The rutting parameter, G*/sinδ and fatigue cracking parameter, G*sinδ were measured at 52, 58, 64, 70, 76 C and 19, 22, 25, 28, 31 C, respectively. The DSR test results of asphalt binder containing Plant C and L RAPs have been plotted for the comparative analysis purpose (Figures 1 through 3). It may be noted that the Y-axis is in a log-log scale. This is the scale recommended by the Asphalt Institute to find the viscosity of blends of new and recovered asphalt (5). Tables 2 and 3 present the linear regression models between log-log dynamic shear properties and amount of RAP binders from Plant C and L, respectively. It may be noted that the calculation gives 1,000 times the tabulated values for the dynamic shear.

It was observed that most R² values were above 0.90. This indicates a good correlation between log-log G*/sinδ values of the blended asphalt and RAP binder contents. Although, the linear regression models of the data on a log-log scale were not the best fit statistically, the log-log scale was used for the easier presentation and better interpretation. It may be noted that the y-intercept value is the log-log G*/sinδ value at 0 percent RAP binder, or base asphalt.

Analysis and Discussion

The Superpave binder specification requires the rutting factor, G*/sinδ to be a minimum of 1.00 kPa and 2.20 kPa for unaged and RTFO aged binders, respectively. The rutting factor reflects the total resistance of a binder to deform under repeated loading (G*), and the relative energy dissipated into non-recoverable deformation (sinδ) during the loading cycle. A higher value of G*/sinδ implies that the binder behaves more like an elastic material, which is desirable for rutting resistance. Since the tenderness factor, G*/sinδ for unaged binder was higher than 1.00 kPa at 58 and 64 C for AC-10 and AC-20 base binders, respectively; this value was not considered seriously in the present study. Rather, the rutting resistance was evaluated mainly by examining G*/sinδ values of RTFO aged binders, because the aging simulates a short-term aging, including the hardening at the asphalt plant.

As expected, the values of G*/sinδ for unaged and RTFO aged binders were increased as the content of RAP binder was increased at all temperatures (Figures 1 and 2). It was also observed that the binder with Plant L RAP exhibited higher G*/sinδ values than the one with Plant C RAP at all corresponding temperatures. It was also noted that the slope for binder with Plant L RAP is steeper than the one for binder with Plant C RAP. Since all values of G*/sinδ for RTFO aged binders were higher than the minimum 2.2 kPa at 58 and 64 C for AC-10 and AC-20
Table 2. The Linear Regression Models and $R^2$ Values for Blended Asphalt with Plant C RAP Binder in 1997.

<table>
<thead>
<tr>
<th>Dependent Parameter, $Y$</th>
<th>Temp, $(^\circ\text{C})$</th>
<th>G*/$\sin \delta$ determined by DSR test for unaged binder</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>RAP from Plant C</td>
<td></td>
</tr>
<tr>
<td><strong>AC-10 (PG 58-22)</strong></td>
<td></td>
<td>R<strong>2</strong> = 0.87</td>
<td></td>
</tr>
<tr>
<td><strong>AC-20 (PG 64-22)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>$Y = 0.5440 + 0.001412X$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.87$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>$Y = 0.5007 + 0.001127X$</td>
<td>$Y = 0.5241 + 0.001275X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.92$</td>
<td>$R^2 = 0.94$</td>
<td></td>
</tr>
<tr>
<td>64</td>
<td>$Y = 0.4612 + 0.001229X$</td>
<td>$Y = 0.4847 + 0.001172X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.97$</td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>$Y = 0.4188 + 0.001259X$</td>
<td>$Y = 0.4449 + 0.001172X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.94$</td>
<td>$R^2 = 0.99$</td>
<td></td>
</tr>
<tr>
<td>76</td>
<td>$Y = 0.3799 + 0.001234X$</td>
<td>$Y = 0.4037 + 0.001148X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.91$</td>
<td>$R^2 = 0.99$</td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>$Y = 0.5893 + 0.001272X$</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.87$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>$Y = 0.5514 + 0.001754X$</td>
<td>$Y = 0.5779 + 0.001401X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.98$</td>
<td></td>
</tr>
<tr>
<td>64</td>
<td>$Y = 0.5157 + 0.001467X$</td>
<td>$Y = 0.5257 + 0.001368X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.94$</td>
<td>$R^2 = 0.98$</td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>$Y = 0.4712 + 0.001480X$</td>
<td>$Y = 0.5007 + 0.001224X$</td>
<td></td>
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<tr>
<td>G*/$\sin \delta$ determined by DSR test for RTFO aged binder</td>
<td>$R^2 = 0.91$</td>
<td>$R^2 = 0.93$</td>
<td></td>
</tr>
<tr>
<td>76</td>
<td>$Y = 0.4370 + 0.001565X$</td>
<td>$Y = 0.4614 + 0.001394X$</td>
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</tr>
<tr>
<td></td>
<td>$R^2 = 0.91$</td>
<td>$R^2 = 0.99$</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>$Y = 0.8197 + 0.0002708X$</td>
<td>$Y = 0.8305 + 0.0001574X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.94$</td>
<td>$R^2 = 0.93$</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>$Y = 0.8086 + 0.0003143X$</td>
<td>$Y = 0.8199 + 0.0002087X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.95$</td>
<td>$R^2 = 0.90$</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>$Y = 0.7956 + 0.0003990X$</td>
<td>$Y = 0.8085 + 0.0002786X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.96$</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>$Y = 0.7844 + 0.0004263X$</td>
<td>$Y = 0.7958 + 0.0003060X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.95$</td>
<td>$R^2 = 0.99$</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>$Y = 0.7701 + 0.0004835X$</td>
<td>$Y = 0.7818 + 0.0003584X$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.94$</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. Independent variable, $X$ indicates the amount of RAP binder in blended asphalt.
2. Dependent variable, $Y$ is in log-log value.
3. '-' indicates that the data is not enough to develop the linear regression model.
Table 3. The Linear Regression Models and $R^2$ Values for Blended Asphalt with Plant L RAP Binder in 1997.

<table>
<thead>
<tr>
<th>Dependent Parameter, $Y$</th>
<th>Temp, $(^\circ C)$</th>
<th>AC-10 (PG 58-22)</th>
<th>AC-20 (PG 64-22)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$Y = 0.5440 + 0.002042X$</td>
<td>$Y = 0.5926 + 0.001383X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.98$</td>
</tr>
<tr>
<td>G*/sin$5$ determined by DSR test for unaged binder</td>
<td>52</td>
<td>$Y = 0.5007 + 0.001777X$</td>
<td>$Y = 0.5241 + 0.001995X$</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.90$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$Y = 0.4612 + 0.002244X$</td>
<td>$Y = 0.4847 + 0.002077X$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$R^2 = 1.00$</td>
<td>$R^2 = 0.95$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$Y = 0.4418 + 0.002302X$</td>
<td>$Y = 0.4449 + 0.002114X$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$R^2 = 0.98$</td>
<td>$R^2 = 0.97$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$Y = 0.3528 + 0.002737X$</td>
<td>$Y = 0.4193 + 0.001927X$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.98$</td>
</tr>
<tr>
<td>G*/sin$5$ determined by DSR test for RTFO aged binder</td>
<td>52</td>
<td>$Y = 0.5893 + 0.001910X$</td>
<td>$Y = 0.6287 + 0.001457X$</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>$R^2 = 0.94$</td>
<td>$R^2 = 0.93$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$Y = 0.5514 + 0.001996X$</td>
<td>$Y = 0.5779 + 0.001766X$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$R^2 = 0.95$</td>
<td>$R^2 = 0.93$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$Y = 0.5157 + 0.002063X$</td>
<td>$Y = 0.5257 + 0.001926X$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.93$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$Y = 0.4333 + 0.002228X$</td>
<td>$Y = 0.4649 + 0.002019X$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.93$</td>
</tr>
<tr>
<td>G*/sin$5$ determined by DSR test for PAV aged binder</td>
<td>19</td>
<td>$Y = 0.8197 + 0.0003111X$</td>
<td>$Y = 0.8305 + 0.0002013X$</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>$R^2 = 0.68$</td>
<td>$R^2 = 0.65$</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>$Y = 0.8086 + 0.0003699X$</td>
<td>$Y = 0.8199 + 0.0002491X$</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>$R^2 = 0.72$</td>
<td>$R^2 = 0.74$</td>
</tr>
<tr>
<td></td>
<td>25</td>
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<td>$Y = 0.8085 + 0.0003387X$</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>$R^2 = 0.81$</td>
<td>$R^2 = 0.90$</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>$Y = 0.7844 + 0.0005099X$</td>
<td>$Y = 0.7958 + 0.0003887X$</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>$R^2 = 0.76$</td>
<td>$R^2 = 0.76$</td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>$Y = 0.7701 + 0.0005913X$</td>
<td>$Y = 0.7818 + 0.0004596X$</td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>$R^2 = 0.79$</td>
<td>$R^2 = 0.83$</td>
</tr>
</tbody>
</table>

Note: 1. Independent variable, $X$ indicates the amount of RAP binder in blended asphalt.
2. Dependent variable, $Y$ is in log-log value.
base binders, respectively. It appeared that the addition of RAP binder would enhance the resistance against rutting.

The maximum limit for the fatigue resistance factor, $G\sin\delta$ has been set at 5,000 kPa on RTFO and PAV aged binders. The smaller the $G\sin\delta$ value, the more elastic the material and the better resistance to fatigue cracking. From Figure 3, it can be seen that $G\sin\delta$ was increased as the amount of RAP content increases. The increase in the $G\sin\delta$ value was mainly due to $G^*$. Values of $\delta$ decreased as the amount of RAP content increases, but this decrease was not sufficient to influence the values of $G\sin\delta$ significantly.

It may be noted that the AC-10 (PG 58-28) base binder containing 55 percent or higher of Plant C RAP binder and 45 percent or higher of Plant L RAP binder at 22 C did not meet the fatigue resistance criteria (Figure 3a and 3b). Similarly, it can be observed that AC-20 (PG 64-22) base binder containing 55 percent or higher of Plant C RAP did not meet the requirement at 25 C, and also the ones with 45 percent or higher of Plant L RAP did not meet the requirement at 25 C as shown in Figure 3c and 3d, respectively. Therefore, the Superpave criteria of $G\sin\delta$ may dictate the maximum amount of RAP addition not to have fatigue cracking.

**MECHANICAL PROPERTIES OF AC-20 (PG 64-22) BASE ASPHALT CONTAINING PLANT C RAP BINDER**

Quasi-Static Characterization Experiments with Uniaxial Compression Test

The experimental design to evaluate mechanical properties is summarized in Table 4 and the experiment was conducted in the year of 1997 (3).

<table>
<thead>
<tr>
<th>Tests</th>
<th>RAP,%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Comp. Strength</td>
<td>3</td>
</tr>
<tr>
<td>Fracture</td>
<td>5</td>
</tr>
<tr>
<td>SHPB</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: 1. The blended asphalt was prepared adding Plant C RAP into AC-20 (or PG64-22) base asphalt.

2. A Compressive strength test was performed at 22°C. Fracture and SHPB tests were performed at 22°C and 0°C.
The quasi-static experiments were conducted in accordance with the procedure of American Society for Testing and Materials (ASTM) D1074-93 to determine the compressive strength of the binders (6). The specimen was cylindrical in shape with 101.6 mm (4 inches) in diameter and 50.8 mm (2 inches) height.

The specimens were fabricated using the Marshall Compactor (5). The aggregates used for this study were only size #30+50 and were procured from two different contractors, i.e., contractors T and C. The aggregate size was so chosen that the aggregate would not contribute to the strength of the specimen and only binder characteristics could be studied (7). The virgin or base asphalt cement used was AC-20 (PG 64-22). The RAP was obtained from Plant C, and the RAP binder was extracted and recovered at RIDOT Lab. The compaction procedure was in accordance with Kennedy et al. (7). The heated mix of the binder and aggregates was compacted using the Instron testing machine under a uniaxial load of 2,722 Kg (6,000 lbs) for 15 minutes. The compacted specimen was later ejected and cured overnight before testing.

**Quasi-Static Behavior of Virgin Asphalt and RAP Binders**

The specimens were tested in axial compression without lateral support at a uniform rate of vertical displacement of 1.25 mm/min-mm (0.05 in./min-in), i.e., for the present study at 2.5 mm/min (0.1 in/min). The displacement as a function of the load was recorded using a two-channel data acquisition system, the NICOLET. As per the standards, three specimen each were tested at each increment in RAP binder content and the average of the results was taken as the compressive strength of the sample, at the corresponding RAP binder percentage.

The results from this study are presented in Table 5 and typical results from one of the experiments for each binder combination are provided in Figure 4. A perceivable increase of over 100 percent was noticed in both stiffness ($E$) and $\sigma_y$ for the 75 and 100 percent RAP specimen. The addition of RAP however, did not cause a noticeable variation in the yield strain of the virgin binder. An aspect to note is the steady increase in the linear range of the stress-strain response when RAP binder content is increased in the binders as shown in Figure 4.

**Table 5. Result of Static Characterization Study with Blended Asphalt of AC-20 (PG 64-22)Base Binder and Plant C RAP Binder.**

<table>
<thead>
<tr>
<th>RAP, %</th>
<th>Compressive Strength, kPa</th>
<th>Yield Strain, %</th>
<th>Stiffness, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Trial 1</td>
<td>Trial 2</td>
<td>Trial 3</td>
</tr>
<tr>
<td>0</td>
<td>52.6</td>
<td>52.1</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>70.2</td>
<td>69.2</td>
<td>61.1</td>
</tr>
<tr>
<td>50</td>
<td>87.8</td>
<td>87.5</td>
<td>87.6</td>
</tr>
<tr>
<td>75</td>
<td>123.8</td>
<td>109.9</td>
<td>115</td>
</tr>
<tr>
<td>100</td>
<td>123.9</td>
<td>124.5</td>
<td>124.5</td>
</tr>
</tbody>
</table>

Note: - indicated that the data is not available.
The mechanism of failure in all the specimens was noticed to be shear dominated, as shown in Figure 5. The side view is indicative of shear failure in the specimen, wherein the wedge shaped segments have been removed from the central core. The angle of shear happens to be approximately 45 degrees. This was observed in all the specimens, irrespective of the RAP binder content in them. The top view of 100 percent RAP specimen (Figure 6) shows the presence of circumferential cracks and also the formation of different segments along the periphery of the specimen.

Figures 7 and 8 show the presence of longitudinal cracks in 0 and 50 percent RAP specimens that were tested. These cracks presumably have been caused due to the circumferential stresses $\sigma_{\theta\theta}$, thus splitting the surface of the specimen lengthwise. These circumferential stresses may have been caused because of frictional effects, leading to a complex stress state along the periphery of the specimen. The cracks were observed to be much more predominant as the RAP binder content was increased, much unlike the presence of smaller cracks in the 0 percent RAP specimen. The cracks then proceeded inwards and due to maximum shear stress occurring along those planes, led to the formation of the wedge shaped segments. Also it appears that the number of these wedge shaped segments decreased as the RAP binder content increased. The low stiffness and the observation of crack propagation indicate a large amount of ductility in the virgin binder. However, it is apparent that the addition of RAP is causing the stiffness to increase, thus causing the binder to behave in a brittle manner.

Fracture Toughness Characterization

Fracture toughness is the value of the stress intensity at which the crack begins to propagate. Early studies on the behavior and performance of bituminous concrete go back to the work of Monismith et al. (8). The analysis of the experimental results indicated the variation of fracture toughness with asphalt content and consistency as well. The influence of asphalt content on the fracture toughness was found to be dependent upon the test temperature. The concepts of fracture mechanics and fatigue crack growth have considered the effect of various mixture constituents, such as asphalt cement, filler, polymeric and fibrous additives (9). The stress rate dependency of asphaltic overlays was investigated, and the fracture toughness was evaluated at various geometrical and loading conditions. The continuously changing stress distribution during the crack growth process is described by linear elastic fracture mechanics principles using Paris' law (10). Fatigue life found to be increased as the magnitude of load decreases. A fracture toughness test was used to measure the resistance of a material to crack growth in the present study.

Comprehensive research on the assessment of the performance of a series of asphalt mixes with varying recycled asphalt content was conducted by Sulaiman and Stock (11). The gradation based on Marshall mix design was implemented in the study. The fracture toughness testing was carried out at three different sub-zero temperatures using the three points beam specimen. Test results indicated the fracture toughness, $K_{IC}$ values at $-5$ C was greater than at $-15$ C but less than those obtained at $+5$ C. The fracture toughness of a mix containing 70 percent RAP binder was found to be marginally higher than values obtained for mixes containing pure binder. However, it was also found that $K_{IC}$ reached a maximum value between $+5$ C and $-5$ C for the mix tested. A factor known as the elastic-plastic region was used to study the
Lee et al.

elastic-plastic response of the mix. Curiously, the results indicated that the resistance to crack growth was greater at lower temperatures. Also, data showed that RAP binder contents did not have any significant effect on crack growth behavior for the range of mix tested.

ASTM Fracture Toughness Criteria

A fracture toughness test essentially measures the resistance of a material to crack extension. Fracture toughness is that value of the stress intensity at which the crack begins to propagate, catastrophically. The present study utilized the procedure of ASTM E399 “Standard Test Method for Plane-Strain Fracture Toughness of Metallic Materials” (6). The standard disk-shaped compact, DC(T), specimen geometry was adopted, mainly because of ease in fabrication. The standard proportions of this geometry are shown in Figure 9. The fracture toughness of a material having such a geometry is given by following relation:

\[ K_{IC} = \left( \frac{P_{max}}{BW^{1/3}} \right) f \left( \frac{a}{W} \right) \]  

(1)

Where,

- \( K_{IC} \) = Fracture Toughness, MPa√m
- \( P_{max} \) = Maximum load to failure, N
- \( B \) = Thickness of the specimen, m
- \( W \) = Width of the specimen, m, and
- \( f(a/W) \) = a factor dependent upon the geometry

\[ f(a/W) = \frac{\left(2 + a/W\right)\left(0.76 + 4.8a/W - 11.58(a/W)^2 + 11.43(a/W)^3 - 4.08(a/W)^4\right)}{(1 - a/W)^{5/2}} \]  

(1a)

The blank specimen had a diameter of 101.6 mm, and had a width of 76.2 mm (D = 1.35W). The thickness of the specimen was 38.1 mm, as per the relation W/B = 2. For an initial crack length of 34.29 mm, and an a/W ratio of 0.45, the f(a/W) factor was calculated as 8.71.

Fracture Toughness Characterization Experiments

The asphalt cement and Plant C RAP binder were heated to 160 C for an hour, till they started to flow. They were further mixed with 550 grams of heated aggregates. The weight of binder as a percentage of the total weight was established by performing preliminary tests, and the maximum value of fracture toughness was found to occur at 6 percent binder content. This mixture was then compacted with 55 blows on either face to account for proper compaction. Finally, the specimen was ejected, cured overnight and machined to the required dimensions. The edge crack was band-sawed and was sharpened up to 1 mm using a diamond saw to provide a sharp crack. The Instron testing machine was used to load the specimen. It was made sure that the loading pins were sufficiently lubricated so as to nullify frictional effects.
Rate Dependency of Virgin Asphalt Binder

In order to study the rate dependency of the virgin asphalt binder, experiments were carried out to evaluate $K_{IC}$ at different rates of loading from 25.4 mm per minute all the way up to 254 mm per minute. The results from this study are shown in Figure 10. As can be seen from the figure, a gradual increase in fracture toughness was noticed with an increasing rate. However, the values stabilized for rates over 152.4 mm per minute. Thus it was decided that further experimentation would be carried out at a displacement rate of 152.4 mm/minute. The maximum average fracture toughness obtained for the virgin binder was 78 MPa$\cdot$mm. The error generated in data was within a band of ±10 percent and well within tolerable experimental consistencies.

Room Temperature Experiments - $K_{IC}$ Evaluation

After determining the rate of loading to be used, further experimentation was carried out in a two phase manner. Firstly, the fracture behavior of binders with RAP was studied at room temperature, in order to evaluate fatigue cracking resistance. Secondly, the low temperature fracture characterization was conducted to examine the brittle fast failure, i.e., low-temperature cracking in the upper layers of the pavement.

Two specimens were tested at each increment of 25 percent RAP binder content and at the loading rate of 152.4 mm per minute. Also crack sharpening was consistently maintained in all the cases so as to obtain an accurate estimate of fracture toughness.

The trends from the experiments can be represented by a quadratic curve fit as shown in Figure 11. The percentage increase in the fracture toughness values for the 25 and 50 percent RAP binder cases were 5 and 6.3 percent, respectively when compared to the 0 percent RAP binder case. However, there was a significant increase of 71 percent in the toughness values when 75 percent RAP binder was added to the virgin binder. The maximum increase of 118 percent was noticed in the case of the 100 percent RAP specimen.

In order to understand this behavior better, the crack propagation in all the specimens was studied. The crack propagation for the 0, 50 and 100 percent RAP specimens are shown in Figure 12 through 14. A most common feature as shown by the photographs is the stable crack propagation leading to final arrest. Also the cracks seem to have propagated in an irregular manner. By a more careful observation of the 0 percent RAP specimen, it can be seen that the crack actually branched through a distance of 15.24 mm before finally arresting. The crack jump distance increased as the RAP content increased, and is perhaps indicative of reduced arrest toughness. Although stable crack growth was observed in all the experiments, the crack tended to propagate unstably in the 100 percent RAP specimen. Also the crack opening displacement seems to have decreased with RAP content. These visual observations are indications of plastic zones ahead of the crack front. In this vein it can be concluded that the addition of RAP is decreasing the ductility inherent in the binder and causing it become rather brittle.

Dynamic Characterization Using the Split Hopkinson Pressure Bar (SHPB)

The dynamic response of asphalt to compressive stress pulses induced due to traffic loads plays a significant role in the flexible pavement structures. The dynamic response of various asphalt mixtures and over range of frequencies were studied by Majidzadeh et al. (9). The
dynamic modulus, $E'$ was found to be dependent on the nature and percentage of additives used. The most effective additives were found to be sulfur, petroset emulsion and asbestos fiber. Also the dynamic modulus was found to decrease with increase in temperature indicating plasticity effects. Sousa and Monismith (12) idealized the pavement as a multi-layer elastic or viscoelastic system, i.e., the system response to dynamic loads were assumed to have internal damping and hence a phase lag 'd'. Test results indicated that the stiffness-modulus strongly depended on frequency and temperature. The internal damping was found to decrease with the increasing frequency and temperature. Strangely, the dynamic modulus was found to be independent of the stress level for the range of frequencies employed. Both the dynamic modulus and the shear modulus were found to be dependent on the density of the specimen. Also Poisson's ratio was found to decrease with increasing frequency of loading.

Sulaiman and Stock (11) conducted experiments to determine the dynamic stiffness of asphalt mixes with varying percentages of reclaimed asphalt. Testing was conducted over a range of temperatures and frequencies of sinusoidal loads. It was observed that the maximum stability was achieved with a mixture having 70 percent RAP binder by weight. However, it was also shown that incorporating RAP made a significant difference to the dynamic stiffness of the mix. At higher temperature, $E'$ was decreased.

**The SHPB for High Strain Rate Compression Testing**

The SHPB was used to study the dynamic behavior in the present study. The SHPB technique is a well established experimental technique used to study dynamic behavior in both ductile and brittle materials alike (13-16). A conventional SHPB or the Kolsky Bar consists of a striker bar, an incident bar and a transmitter bar, as illustrated in Figure 15 (17). The specimen under study is sandwiched between the incident and transmitter bar. The striker bar is launched at a predefined velocity towards the incident bar. This impact generates a compressive stress pulse, which travels towards the specimen. The amplitude of the stress pulse is a function of the velocity of the striker bar, and its period is approximately equal to twice the travel time of the wave in the striker bar. This wave, upon reaching the incident bar-specimen interface, gets partly reflected back and partly transmitted into the specimen depending on the impedance mismatch and the area mismatch between the specimen and the bar. From one-dimensional wave theory, it has been established that the amplitude of the transmitted pulse is a measure of the stress in the specimen and the amplitude of the reflected pulse is a measure of the strain rate in the specimen. Thus upon integrating the reflected pulse, the strain in the specimen can be determined. The specimen can be subjected to a wide range of strain rates by employing striker bars of various lengths.

**Governing Equations**

The fundamental relations stem from the classical D'Alembert-one dimensional wave equation given by

$$\mu(x,t) = f(x - c_f) + g(x + c_f)$$

(2)
where 'r' and 'g' represent propagating disturbances and are arbitrary functions of integration determined by the initial conditions of the forcing function of a given problem. Also, 'r' corresponds to a wave traveling in the positive x direction and 'g' corresponds to a wave traveling in the negative x-direction. A schematic of the incident, reflected and the transmitted strain pulses, \( e_i, e_r, \) and \( e_t \) are provided in Figure 16. From one dimensional rod theory, the displacements at the two specimen-bar interfaces are given by

\[
\mu_1 = c_0 \int_0^t (-\varepsilon_i + \varepsilon_r) dt \tag{3}
\]

\[
\mu_2 = -c_0 \int_0^t (\varepsilon_i) dt \tag{4}
\]

The average strain of the specimen, \( \varepsilon_s \), is then given by,

\[
\varepsilon_s = \frac{c_0}{l_s} \int_0^t (\varepsilon_i - \varepsilon_r - \varepsilon_t) dt \tag{5}
\]

where 'l_s' is the original length of the specimen. The loads at the two interfaces are given by,

\[
P_1 = A_b E_b (\varepsilon_i + \varepsilon_r) \tag{6}
\]

\[
P_2 = A_b E_b \varepsilon_t \tag{7}
\]

where, 'A_b' is the cross-sectional area of the bars. Now, an important assumption is made that wave propagation effects within short specimen may be neglected, thus \( P_1 = P_2 \). From this, it follows that \( \varepsilon_i + \varepsilon_r = \varepsilon_t \), and so, equation (5) simplifies to

\[
\varepsilon_s (t) = -\frac{2c_0}{l_s} \int_0^t (\varepsilon_i) dt \tag{8}
\]

The average stress in the specimen is given by,

\[
\sigma_s = E_b \left( \frac{A_b}{A_s} \right) \varepsilon_t \tag{9}
\]

where, 'E_b' is the modulus of elasticity of the pressure bars, 'A_s' is the instantaneous cross sectional area of the specimen and 'c_o', the wave speed in the bar, is known to be \( \sqrt{E/\rho} \), \( \rho \) being the mass density of the bar material.
There are following two fundamental assumptions in deriving the above equations. Firstly, wave propagation within the pressure bars must remain one-dimensional. Since the strain gages measure surface displacements, it is extremely important that this condition is met. This essentially means that the wave can be assumed to be one-dimensional, and surface displacements are accurate indicators of surface axial displacements in the bars. Secondly, the specimen must undergo homogenous deformation. Uniform deformation is generally hindered by radial and longitudinal inertia of the specimen and the frictional contact at the specimen-bar interfaces. Hence, it is customary to use oil-based molybdenum disulfide as a lubricant for experiments conducted at room temperature.

**Dynamic Characterization Experiments**

The dynamic characterization experiments were conducted by fabricating specimens having a diameter of 43.2 mm (1.729 in.) and a length of 0.125 mm (0.5 in.). The diameter was selected so as to provide a cross-sectional area mismatch between the pressure bars (diameter 50 mm) and the specimen of 25 percent, assuming a maximum strain of 25 percent in the specimen. This ensured that the specimen diameter did not exceed the diameter of the bars as it expanded radically.

The weight of aggregates needed for specimen fabrication was obtained from volume considerations, i.e., by linearly scaling down the volume needed for fabricating the DC(T) specimen for fracture toughness testing. Accordingly, the weight of aggregates came down to 35 grams. The specimen constituents were the same as the ones in fracture toughness testing and the specimen was compacted in accordance with the procedure of Kennedy et al. (7).

**Dynamic Response of Virgin Asphalt and RAP Binder at Room Temperature**

Experiments were conducted to determine the dynamic response of virgin and RAP binders. The dynamic flow stress was evaluated at every increment in RAP content and at room temperature (~22 C). The SHPB system in conjunction with the high speed data acquisition system, LECROY, was used for this study and a user-defined code was compiled to aid in data manipulation. All the experiments were carried out at a nominal strain rate of 450 /s in order to establish a base for performance comparison.

The trends in the true stress-strain response from 0 and 22 C experiments are provided in Figure 17. Also the variation of dynamic flow stress at room temperature as a function of RAP content is provided in Table 6, and the trends are presented in Figure 18. As can be perceived, the flow stress increased as the percentage of RAP was increased. There was a nominal increase of 6 percent in the values of flow stress obtained for the 25 and 50 percent RAP specimen when compared to flow stress of 0 percent RAP. An increase of 12 percent in the flow stress values was noticed for the 75 percent RAP specimen. The largest increment of 26 percent was noticed in the case of the 100 percent RAP specimen.

The specimen failure was noticed to be shear dominated, much like the one observed in the quasi-static experiments. An examination of the specimen face (facing the incident bar) revealed the presence of circumferential cracks, radial cracks and longitudinal cracks-in shown in Figure 19 (75 percent RAP specimen). A more predominant segment formation was noticed in these specimens as compared to specimens tested under static loading conditions. It seems quiet possible that cracking initiated because of circumferential stresses along the periphery of the
As previously mentioned, these stresses may have been caused due to inertial and frictional effects. These reasons may also have been a factor leading to the formation of larger wedge shapes. The side view of the 75 percent RAP specimen, Figure 20, indicates the shear dominant failure in the specimen. This type of failure was typical of all the specimens that failed at room temperature. These visual observations and trends from the dynamic true stress-strain plots are indicators of reduced ductility in the specimen as the RAP percentage increased. The observations from the quasi-static experiments accentuate this argument.

Table 6. Results of Dynamic Characterization of RAP Binder at Room Temperature.

<table>
<thead>
<tr>
<th>RAP Percentage</th>
<th>Flow Stress (MPa)</th>
<th>Flow Strain (%)</th>
<th>Strain Rate (450/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Run 1</td>
<td>Run 2</td>
<td>Avg.</td>
</tr>
<tr>
<td>0</td>
<td>26</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>25</td>
<td>27</td>
<td>27</td>
<td>27</td>
</tr>
<tr>
<td>50</td>
<td>28</td>
<td>27</td>
<td>27.5</td>
</tr>
<tr>
<td>75</td>
<td>30</td>
<td>29</td>
<td>29.5</td>
</tr>
<tr>
<td>100</td>
<td>32</td>
<td>34</td>
<td>33</td>
</tr>
</tbody>
</table>

Fracture Toughness Test at Low Temperature

The methodology adopted in conducting these set of experiments remained much the same as the previous case except the low testing temperature (0°C). Thus, the cured specimens were soaked in ice for a period of 24 hours, before testing. Three specimens each were tested at every increment of RAP content and an average of the results was reported. The results from the experiments are presented in Figure 11. Interestingly, an increase of 30 percent was noticed in the $K_{IC}$ values from these experiments as compared to that obtained for the 100 percent RAP specimen at room temperature. The data was fit by a flat trend line, which signifies that the addition of RAP does not have any noticeable effect on the performance of the binder at low temperatures.

A brittle fast fracture occurred in all the specimens tested at this temperature. The crack propagation in the 100 percent RAP specimen, shown in Figure 21 is indicative of this fact. Interestingly, stable crack propagation leading to arrest was observed for the 100 percent RAP specimen at room temperature, as shown in Figure 14. The crack opening displacement was also found lesser at low temperature than what was observed at room temperature.

In the light of visual observations of crack propagation in the specimens at both temperatures, the following can be concluded. The actual numerical values of fracture toughness may be rendered invalid due to the presence of plastic zones at room temperature. This is
because fracture toughness is estimated based on the principles of linear elastic fracture mechanics. However, the $K_{IC}$ values obtained at 0 C are definitely valid as the binder behaves as a true brittle material at this temperature. Hence, the increasing trend of $K_{IC}$ with RAP content can be considered to be one leading into brittle behavior, as shown in Figure 21. More importantly, it is plausible that there is a ductile to brittle transition in the behavior of the binder as the temperature is varied from room 22 C to 0 C.

**Dynamic Response of Virgin Asphalt and RAP Binders at Low Temperature**

Experiments were carried out at low temperature (0 C) in order to determine the low temperature performance of RAP binders under dynamic loading conditions. These sets of experiments were carried out in as much the same way as the room temperature experiments. The only difference was that the specimens were soaked in an ice bath for a period of 24 hours so as to lower their temperature to 0 C. These set of experiments were also conducted at a nominal strain rate of 450 /s in order to make a comparison of performance. The equipment used and the procedure for acquiring data was much the same as mentioned in the previous section.

The results from these experiments are provided in Table 7. There was a marginal increase of 8 percent in the dynamic flow stress values for the 100 percent RAP specimen when compared to that of the 0 percent RAP specimen. The maximum strain (true strain) was also observed to be the same in all the cases. The specimens could not be recovered for postmortem analysis as all of them had fragmented upon impact. This observation is definitely indicative of brittle failure in the specimen.

**Table 7. Results of Dynamic Characterization of RAP Binder at Low Temperature.**

<table>
<thead>
<tr>
<th>RAP Percentage</th>
<th>Flow Stress (MPa)</th>
<th>Flow Strain (%)</th>
<th>Strain Rate (450/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>31.5</td>
<td>7.2</td>
<td>456</td>
</tr>
<tr>
<td>25</td>
<td>34.0</td>
<td>7.0</td>
<td>429</td>
</tr>
<tr>
<td>50</td>
<td>32.0</td>
<td>7.0</td>
<td>433</td>
</tr>
<tr>
<td>75</td>
<td>32.0</td>
<td>7.0</td>
<td>446</td>
</tr>
<tr>
<td>100</td>
<td>34.0</td>
<td>7.0</td>
<td>448</td>
</tr>
</tbody>
</table>

**Note:** Testing was performed at 0 C.
To study the variation from the same asphalt plant, the other RAP was secured from a different stockpile of Plant C in 1998. Different amount of RAP binders were blended with base asphalts similar to those used in 1997 study, and DSR tests were performed with the same stress mode used in the previous year study (Table 8).

Table 8. Experimental Design to Evaluate Rheological Properties and Superpave Grading of Asphalt Containing a RAP Binder in 1998.

<table>
<thead>
<tr>
<th>Tests</th>
<th>RAP, %</th>
<th>0</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>75</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>DSR, Unaged</td>
<td></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>DSR, RTFO</td>
<td></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>DSR, PAV</td>
<td></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>BBR</td>
<td></td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: 1. (Plant C RAP) x 2 (PG 58-28 & PG 64-22) = 2
2. DSR tests were performed at the Superpave high and intermediate temperatures, i.e., 52, 58, 64, 70, 76°C and 19, 22, 25, 28, 31°C, respectively.
3. All BBR testing was performed at the Superpave low temperatures (-6, -12, -18, and -24°C).
4. Two replicate samples were tested.

Similar observations were obtained from these experiments that $G^*/\sin\delta$ and $G^*\sin\delta$ increased as the amount of RAP content increased as shown in Figures 22 through 24. The DSR test results for blended asphalt with Plant C RAP binder from 1997 were compared with the data from 1998. The dynamic shear values of the base binder from 1997 were slightly higher than the ones from 1998, and the dynamic shear of 100 percent RAP binder was lower than the ones from 1998. Both data sets have similar trends, but the slope of the trend from 1998 was steeper. It can be concluded that the properties of the RAP vary even though the RAP came from the same asphalt plant.

From the statistical analysis, it was observed that there was a good linear relationship between log-log dynamic shear properties and RAP binder content as can be seen from Figure 22 through 24. The RAP binder content and temperatures were observed to be significant, and it can be concluded that both factors had an effect on the dynamic shear values. Table 9 shows the regression models and $R^2$ values for blended asphalt with Plant C RAP binder in 1998. It was observed that the $R^2$ values were higher than the ones in 1997.

<table>
<thead>
<tr>
<th>Dependent Parameter, Y</th>
<th>Temp, (°C)</th>
<th>PG 58-28</th>
<th>PG 64-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>G*/$\sin \delta$ determined by DSR test for unaged binder</td>
<td>52</td>
<td>$Y = 0.5362 + 0.001559X$</td>
<td>$Y = 0.5738 + 0.001180X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>$Y = 0.4891 + 0.001698X$</td>
<td>$Y = 0.5321 + 0.001265X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$Y = 0.4450 + 0.001771X$</td>
<td>$Y = 0.4860 + 0.001441X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$Y = 0.3972 + 0.001888X$</td>
<td>$Y = 0.4421 + 0.001405X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 1.00$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$Y = 0.3491 + 0.001969X$</td>
<td>$Y = 0.3965 + 0.001505X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.99$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td>G*/$\sin \delta$ determined by DSR test for RTFO aged binder</td>
<td>52</td>
<td>$Y = 0.5839 + 0.001586X$</td>
<td>$Y = 0.6222 + 0.001193X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>58</td>
<td>$Y = 0.5428 + 0.001729X$</td>
<td>$Y = 0.5837 + 0.001299X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>64</td>
<td>$Y = 0.5090 + 0.001840X$</td>
<td>$Y = 0.5468 + 0.001343X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>$Y = 0.4560 + 0.001979X$</td>
<td>$Y = 0.5043 + 0.001457X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.97$</td>
<td>$R^2 = 1.00$</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>$Y = 0.4105 + 0.002071X$</td>
<td>$Y = 0.4610 + 0.001564X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.97$</td>
<td>$R^2 = 0.99$</td>
</tr>
<tr>
<td>G*/$\sin \delta$ determined by DSR test for PAV aged binder</td>
<td>19</td>
<td>$Y = 0.8172 + 0.0003814X$</td>
<td>$Y = 0.8315 + 0.0001962X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.95$</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>$Y = 0.8051 + 0.0004431X$</td>
<td>$Y = 0.8215 + 0.0002290X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.96$</td>
<td>$R^2 = 0.95$</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>$Y = 0.7924 + 0.0005010X$</td>
<td>$Y = 0.8107 + 0.0002649X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.97$</td>
<td>$R^2 = 0.96$</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>$Y = 0.7786 + 0.0005621X$</td>
<td>$Y = 0.7990 + 0.0003038X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.97$</td>
<td>$R^2 = 0.9605$</td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>$Y = 0.7629 + 0.0006390X$</td>
<td>$Y = 0.7859 + 0.0003490X$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R^2 = 0.97$</td>
<td>$R^2 = 0.96$</td>
</tr>
</tbody>
</table>

Note: 1. Independent variable, X indicates the amount of RAP binder in blended asphalt.
2. Dependent variable, Y is in log-log value.
BRR tests were added in the 1998 study to evaluate the thermal cracking resistance characteristics at low temperatures, i.e., -6, -12, -18, and -24 C in accordance with the procedure of American Association of State Highway and Transportation Officials (AASHTO) TPI (2). The overall experimental design is shown in Table 8.

The BBR is a tool to evaluate the properties of binders at low temperature in Superpave binder specification. A constant load of 100 grams is applied for 4 minutes at the center of the asphalt beam which is supported at both ends. The deflection of the beam is measured continuously during the loading and the stiffness is calculated. At very low temperatures, the binder gets brittle and this leads to cracking. To avoid this, the Superpave binder specification has set the limits for the creep stiffness not to exceed 300 MPa and the m-value to be at least 0.300 after 60 seconds of loading. A higher m-value indicates better relaxation characteristics; therefore the thermal stresses do not accumulate to produce cracks. The characterization of RAP binder with the BBR has shown an increase in creep stiffness and decrease in m-value for all temperatures as the amount of RAP binder increases. The effect of RAP binder content on creep stiffness and m values can be seen from Figures 25 and 26, respectively. It should be noticed that Y-axis is in log-log scale. Both the PG 58-28 and PG 64-22 blends have a similar trend, but the slope for the PG 58-28 blend appears to be steeper than the one for PG 64-22 blend. This indicates that the addition of RAP binder did not improve the thermal cracking resistance of the resulting binder over the base binders. Table 10 summarizes the linear regression models and R² values. A good linear relationship was observed from the analysis.

From the results of DSR and BBR testing, there appears to be a difference in how the different/same RAP sources react with base asphalt binders. As shown before, the addition of RAP has a significant effect on the $G^*/\sin\delta$ and $G^*\sin\delta$ for all temperatures. An addition of RAP may improve the rutting resistance properties. According to BBR testing, the creep stiffness increased and m-value decreased as the amount of RAP increased. The resistance against low temperature cracking was not improved with the increase in RAP binder amount. The SUPERPAVE performance based grading for asphalt containing Plant C RAP binder is reported in Table 11. A general trend is that the addition of RAP binder has increased the high temperature grade for all binders. The increase in the high temperature grade was due to the increase of stiffness at high temperatures and is evident from the measured values of $G^*/\sin\delta$. The low temperature grade for the PG 58-28 blend remained the same until it decreased at 30 percent RAP binder. Interestingly, an increase in RAP binder amount had little effect in the performance grade at low temperatures for PG 64-22 base binder. The low temperature grade remained the same until it decreased at 75 percent RAP binder. It may be noticed that the low temperature grade was controlled by the m-value.
Table 10. The Linear Regression Models and R² Values for Plant C RAP.

<table>
<thead>
<tr>
<th>Dependent Parameter, Y</th>
<th>Temp, (°C)</th>
<th>AC-10 (PG 58-28)</th>
<th>AC-20 (PG 64-22)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Y = 0.6618 + 0.0005170X</td>
<td>Y = 0.6792 + 0.0003134X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.97</td>
<td>R² = 0.93</td>
</tr>
<tr>
<td>Stiffness determined by BBR test</td>
<td>-6</td>
<td>Y = 0.6974 + 0.0003346X</td>
<td>Y = 0.7085 + 0.0002306X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.96</td>
<td>R² = 0.99</td>
</tr>
<tr>
<td></td>
<td>-12</td>
<td>Y = 0.7271 + 0.0002783X</td>
<td>Y = 0.7354 + 0.0001877X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.95</td>
<td>R² = 0.95</td>
</tr>
<tr>
<td></td>
<td>-18</td>
<td>Y = 0.7528 + 0.0001885X</td>
<td>Y = 0.7588 + 0.0001112X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.96</td>
<td>R² = 0.97</td>
</tr>
<tr>
<td></td>
<td>-24</td>
<td>Y = 0.7902 + 0.0002900X</td>
<td>Y = 0.8143 + 0.0002080X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.93</td>
<td>R² = 0.91</td>
</tr>
<tr>
<td>m-value determined by BBR test</td>
<td>-6</td>
<td>Y = 0.4115 + 0.0002619X</td>
<td>Y = 0.4036 + 0.0001629X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.95</td>
<td>R² = 0.88</td>
</tr>
<tr>
<td></td>
<td>-12</td>
<td>Y = 0.3996 + 0.0002311X</td>
<td>Y = 0.3928 + 0.0001627X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.87</td>
<td>R² = 0.97</td>
</tr>
<tr>
<td></td>
<td>-18</td>
<td>Y = 0.3870 + 0.0003093X</td>
<td>Y = 0.3788 + 0.0002433X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>R² = 0.91</td>
<td>R² = 0.91</td>
</tr>
</tbody>
</table>

Note: 1. Independent variable, X indicates the amount of RAP binder in blended asphalt.
2. Dependent variable, Y is in log-log value.
Table 11. SUPERPAVE Performance Grading of Asphalt Containing Plant C RAP Binder in 1998.

<table>
<thead>
<tr>
<th>RAP (%)</th>
<th>DSR Unaged</th>
<th>RTFO Aged</th>
<th>PAV Aged</th>
<th>BBR Stiffness</th>
<th>Slope</th>
<th>m-value</th>
<th>PG Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>G*/sinδ (KPa)</td>
<td>G*/sinδ (KPa)</td>
<td>G*/sinδ (KPa)</td>
<td>S (MPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>1.21@58°C</td>
<td>3.09@58°C</td>
<td>3666@19°C</td>
<td>216@-18°C</td>
<td>0.323</td>
<td></td>
<td>PG 58-28</td>
</tr>
<tr>
<td>10</td>
<td>1.67@58°C</td>
<td>4.77@58°C</td>
<td>4030@19°C</td>
<td>228@-18°C</td>
<td>0.319</td>
<td></td>
<td>PG 58-28</td>
</tr>
<tr>
<td>20</td>
<td>1.22@64°C</td>
<td>4.29@64°C</td>
<td>2872@22°C</td>
<td>217@-18°C</td>
<td>0.330</td>
<td></td>
<td>PG 64-28</td>
</tr>
<tr>
<td>30</td>
<td>1.40@64°C</td>
<td>4.67@64°C</td>
<td>2901@25°C</td>
<td>126@-12°C</td>
<td>0.334</td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td>40</td>
<td>2.03@64°C</td>
<td>6.26@64°C</td>
<td>3561@25°C</td>
<td>144@-12°C</td>
<td>0.332</td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td>50</td>
<td>1.34@70°C</td>
<td>4.42@70°C</td>
<td>2884@28°C</td>
<td>161@-12°C</td>
<td>0.316</td>
<td></td>
<td>PG 70-22</td>
</tr>
<tr>
<td>75</td>
<td>1.43@76°C</td>
<td>4.10@76°C</td>
<td>N/A</td>
<td>102@-6°C</td>
<td>0.315</td>
<td></td>
<td>PG 76-16</td>
</tr>
<tr>
<td>100</td>
<td>2.95@76°C</td>
<td>11.58@76°C</td>
<td>N/A</td>
<td>151@-6°C</td>
<td>0.285</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>PG 58-28</td>
</tr>
<tr>
<td>0</td>
<td>1.15@64°C</td>
<td>3.33@64°C</td>
<td>2932@25°C</td>
<td>129@-12°C</td>
<td>0.341</td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td>10</td>
<td>1.47@64°C</td>
<td>4.47@64°C</td>
<td>2896@25°C</td>
<td>136@-12°C</td>
<td>0.336</td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td>20</td>
<td>1.90@64°C</td>
<td>5.50@64°C</td>
<td>3203@25°C</td>
<td>140@-12°C</td>
<td>0.332</td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td>30</td>
<td>1.21@70°C</td>
<td>3.64@70°C</td>
<td>2501@28°C</td>
<td>156@-12°C</td>
<td>0.322</td>
<td></td>
<td>PG 70-22</td>
</tr>
<tr>
<td>40</td>
<td>1.53@70°C</td>
<td>4.15@70°C</td>
<td>2920@28°C</td>
<td>166@-12°C</td>
<td>0.318</td>
<td></td>
<td>PG 70-22</td>
</tr>
<tr>
<td>50</td>
<td>1.92@70°C</td>
<td>6.59@70°C</td>
<td>3499@28°C</td>
<td>183@-12°C</td>
<td>0.306</td>
<td></td>
<td>PG 70-22</td>
</tr>
<tr>
<td>75</td>
<td>1.79@76°C</td>
<td>6.41@76°C</td>
<td>N/A</td>
<td>203@-12°C</td>
<td>0.305</td>
<td></td>
<td>PG 76-22</td>
</tr>
<tr>
<td>100</td>
<td>2.95@76°C</td>
<td>11.58@76°C</td>
<td>N/A</td>
<td>151@-6°C</td>
<td>0.285</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTE: 1. N/A means that the data is not available.
2. - indicates that the value is insufficient to determine the PG Grading.
CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be drawn from the finding of the present investigation:

1. A good linear relationship was obtained between log-log rheological properties of blended asphalt and the amount of RAP binders.
2. The values of G*/sinδ of blended asphalt were increased as the RAP binder amount was increased. It was also observed that the blended asphalt containing Plant L RAP binder exhibited the higher G*/sinδ and steeper slope than the one containing Plant C RAP binder.
3. All values of G*/sinδ at 58°C for AC-10 (or PG 58-22) base binders met the Superpave minimum requirement, i.e., 1.00 and 2.20 kPa for unaged and RTFO aged binders, respectively. This was also true for the AC-20 (or PG 64-22) base binders at 64°C. Furthermore, the addition of RAP binders increased the Superpave high temperature grade, and could improve the resistance against rutting.
4. For the quasi-static loading experiments, it was observed that the compressive strength and stiffness were increased as the amount of RAP increased. The increase in both compressive strength and stiffness reduced the ductility in the specimens as the amount of RAP binder was increased.
5. An increase in RAP binder content caused an increase in G*/sinδ. Therefore, the Superpave criteria of G*/sinδ may dictate the maximum amount of RAP addition not to have fatigue cracking.
6. The dynamic shear parameters of asphalts containing Plant C RAP binder, i.e., G*/sinδ and G*/sinδ, were significantly different between the years of 1997 and 1998.
7. An increase in RAP binder content caused an increase in creep stiffness, and a decrease in m-value. It appears that the addition of RAP binders did not enhance the binder’s resistance characteristics against low-temperature cracking. Furthermore, the Superpave criteria of s and m may dictate the maximum amount of RAP not to have thermal cracking.
8. The fracture toughness was increased as the amount of RAP increased, at room temperature. However, it was observed that RAP contents had a significant effect on the crack propagation. The crack propagation tended towards instability as the amount of RAP increased, which increased the crack jump distance in the specimen. These indicated that RAP binder addition reduced ductility in a specimen.
9. The fracture toughness values of the binder at low temperature were higher than the ones at room temperature. Again, the crack propagation was seen as a brittle fast fracture in a specimen. However, the increase in RAP content did not have any significant effect on the fracture toughness of the binder at low temperature.
10. The room temperature dynamic flow stress values of 100 percent RAP specimen were increased by 25 percent when compared to the 0 percent RAP specimen. In the mean
time, the dynamic response of the base asphalt binder was not affected by the addition of RAP at low temperature.

11. A comparison of results from the quasi-static and the dynamic experiments indicated an increase in flow stress of many orders of magnitude. Thus, the binder was found to be highly susceptible to rate of loading.

12. It is recommended that further research should be performed with asphalt mixture to incorporate the finding from the binder study.

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REFERENCES

Figure 1. Comparison of DSR Test Results for Unaged Binder at Different Temperatures in 1997.
Figure 2. Comparison of DSR Test Results for RTFO Aged Binder at Different Temperatures in 1997.
Figure 3. Comparison of DSR Test Results for PAV Aged Binder at Different Temperatures in 1997.
Figure 4. Trends in the Quasi-Static Stress-Strain Behavior of RAP Binder.
Figure 5. Shear Dominance in the 100% RAP Specimen, Typical of all the Specimens Under Loading Condition.

Figure 6. Failure of the 100% RAP Specimen, View Shows Circumferential Cracking
Figure 7. Cracking Propagation in the 0% RAP Specimen - Quasi-Static Loading Condition.

Figure 8. Cracking Propagation in the 50% RAP Specimen - Quasi-Static Loading Condition.
Specimen thickness = 38.1
R = 6.35
R = 50.8
34.29
22.23

All dimensions are in ‘mm’

Figure 9. Standard Geometry of Disc Shaped Compact Specimen, DC(T)

Figure 10. Variation of Fracture Toughness with Rate of Loading.
Figure 11. Variation of Fracture Toughness with RAP Content.
Figure 12. Crack Propagation in the 0% RAP Specimen at 22°C.

Figure 13. Crack Propagation in the 50% RAP Specimen at 22°C.

Figure 14. Crack Propagation in the 100% RAP Specimen at 22°C.
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Figure 16. Strain Pulses at the Specimen-Bar Interface.
Figure 17. Typical Trends in the Dynamic Stress-Strain Response of Base Asphalt Binder at 0 and 22°C.

Figure 18. Variation of Dynamic flow Stress with RAP Content.
Figure 19. Crack Propagation in the 75% RAP Specimen under Dynamic Loading Condition.

Figure 20. Shear Dominance in the 75% RAP Specimen - Typical of all Specimen Tested Dynamically.
Figure 21. Crack Propagation in the 100% RAP Specimen at 0°C.
Figure 22. Comparison of DSR Test Results for Unaged Binder at Different Temperatures in 1998.
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Figure 24. Comparison of DSR Test Results for PAV Aged Binder at Different Temperatures in 1998.
Figure 25. Comparison of the Creep Stiffness of PAV Aged Binder at Different Temperatures in 1998.
Figure 26. Comparison of the Slope m-values of PAV Aged Binder at Different Temperatures in 1998.