Laboratory Evaluation of Asphalt-Rubber Gap Graded Mixtures Constructed on Stockholm Highway in Sweden

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ABSTRACT. *The objective of this study was to conduct an advanced laboratory experimental program to obtain typical engineering material properties for reference, asphalt-rubber (AR), and polymer-modified (PM) gap graded asphalt concrete mixtures placed in the Stockholm area of Sweden. The advanced material characterization tests included: Dynamic (Complex) Modulus for stiffness evaluation; triaxial shear strength test to evaluate shearing resistance; repeated load for permanent deformation characterization; beam fatigue for crack evaluation; Indirect Diametral Tensile test for thermal cracking mechanism evaluation; and C* Integral test to assess crack growth and propagation. Furthermore, conventional binder consistency tests were performed to complement other material mixture characteristics. The data was used to compare the performance of the AR gap graded mixture with respect to reference and PM gap graded mixtures. The results showed that the AR gap graded mix would provide better resistance to low temperature cracking and permanent deformation. The expected fatigue life for the AR gap graded mixture was higher than the reference and PM mixtures for the existing highway conditions. Furthermore, the crack propagation tests showed that the AR gap graded mixture had highest resistance to crack propagation than the other two mixtures.*

KEYWORDS: *Asphalt Rubber (AR) Gap Graded, Dynamic (Complex) Modulus, Flow Number, Fatigue, C* Integral, Indirect Tensile Creep and Strength*

1. Introduction

Arizona State University (ASU) in the United States of America is well known for its work on asphalt-rubber (AR) mixtures characterization studies, which includes a recently completed long-range AR pavement research program with the Arizona Department of Transportation (ADOT). The research programs have the ultimate goal in developing typical design input parameters and engineering properties specific for AR mixtures.

In 2008, a cooperative effort between ASU and the Swedish Transport Administration (STA) took place in testing a reference mix and an AR gap graded mixture placed on Malmö E6 External Ring Road in Sweden (Kaloush *et al*, 2009). The advanced material characterization tests were limited because of the mixture availability but included: Dynamic (Complex) Modulus for stiffness evaluation, Flexural Beam test for fatigue cracking evaluation, and C* Integral test to evaluate crack growth and propagation.

In 2009, SRA and ASU undertook another joint effort to test three types of gap graded mixtures: reference, polymer-modified and rubber-modified mixes, placed on E18 highway between the interchanges Järva Krog and Bergshamra in the Stockholm area of Sweden. Figure 1 shows map of E18 Highway near Stockholm-Sweden where the three different gap-graded mixtures were placed.

The AR gap graded mixtures contained approximately 20 percent ground tire rubber (crumb-rubber). The mixtures were sent to ASU laboratories for testing and evaluation (Kaloush *et al*, 2010). This paper documents the various mechanical tests conducted on these mixes to evaluate the pavement materials' performance characteristics in the laboratory at ASU facilities.

2. Objectives and Scope of the Work

The objective of this study was to conduct an advanced laboratory experimental program to obtain typical engineering material properties for "reference", "polymer-modified", and "rubber-modified" gap graded asphalt concrete mixtures placed in the Stockholm area of Sweden. The laboratory testing program utilized current laboratory tests adopted by the pavement community. The results were compared / ranked amongst each mixture with the other to evaluate the anticipated performance of these mixes.

At ASU, the mixtures were re-heated and compacted to cylindrical and beam specimen geometry. A Servopac gyratory compactor was used to compact the cylindrical specimens into 150 mm diameter and 170 mm in height gyratory plugs. One 100 mm diameter sample was cored from each gyratory plug. The sample ends were sawn to arrive at typical test specimens of 100 mm in diameter and 150 mm in height. Beam specimens were prepared according to the Strategic Highway Research Program (SHRP) and the American Association of State Highway and Transportation Officials (AASHTO): SHRP M-009 and AASHTO TP8-94 (SHRP M-009; AASHTO T321-03). Air voids, thickness and bulk specific gravities were measured for each test specimen and the samples were stored in plastic bags in preparation for the testing program.

Conventional binder consistency tests were performed on the three different binders, a virgin binder with no modification, and two modified binders with polymer and crumb-rubber additives. Furthermore, the advanced material characterization tests included: triaxial shear strength, E* dynamic (complex) modulus for stiffness evaluation; repeated load for permanent deformation characterization; indirect tensile creep and strength tests for thermal cracking characterization; flexural beam fatigue for cracking evaluation; and C* Integral test to evaluate crack propagation.

Figure 1. *Location of Stockholm E18 Highway – (A) Järva-Krog to (B) Bergshamra Interchanges*

3. Mixture Characteristics

The designated road section within the construction project had three asphalt gap graded mixtures: a “reference” gap graded mix (designation: ABS 16 70/100) used as a control, a “polymer modified mixture (designation: ABS 16 Nypol 50/100-75), and a “rubber-modified” mixture (designation: GAP 16) that contained approximately 20 percent ground tire rubber (crumb rubber). The Swedish Transport Administration provided information that the field compaction / air voids for the three mixtures were 3.0%. The original mix designs were done using the Marshall Mix design method. The in-situ mixture properties of the Swedish Highway E18 project are reported in Table 1. Table 2 shows the reported average aggregate gradations for the each mixture. The base bitumen used was Pen 70/100.

Table 1. *Mixture Characteristics, Stockholm E18 Highway*

Mix	Binder Content (%)	Air Voids (%)	G_{mm}
Reference ABS 16 70/100	5.9	2.6	2.4642
Polymer ABS 16 Nypol 50/100-75	5.9	2.6	2.4558
Rubber GAP 16	8.7	2.4	2.3588

Table 2. *Average Aggregate Gradations, Stockholm E18 Highway*

	Sieve Size (mm)	Reference	Polymer	Rubber
Gradation (% Passing by mass of each sieve)	22.4	100	100	100
	16	98	98	98
	11.2	65	65	68
	8	38	38	44
	4	23	23	24
	2	21	21	22
	0.063	10.5	10.5	7.5

4. Binder Characterization

The objective of binder testing was to compare the Swedish standard bitumen with Pen 70/100 and the effect of polymer modification (Nypol 50/100-75) normally 3 to 6 % mixed with bitumen, and rubber modification (crumb rubber content of asphalt mix was 1.5-2.0%). Conventional consistency tests, namely, penetration, softening point using ring & ball, and Brookfield viscosity tests were conducted on the three binders, one virgin and two modified at two aging conditions: tank and Rolling Thin Film Oven (RTFO). Also, consistency tests across a wide range of temperatures were conducted according to the accepted American Society for Testing and Materials (ASTM) International and/or AASHTO practices to determine whether there are any unique characteristics or difficulties in handling the material. Test results and analysis conducted in this task provided the viscosity-temperature susceptibility of the three different asphalt binders.

Figure 2 shows a comparison of the viscosity-temperature relationship for the three binders, including a virgin binder and two binders with modification (polymer and rubber additives) at tank and RTFO conditions. It was observed that the rubber modified binders (tank and RTFO) have flatter slopes than the polymer-modified binders and then followed by the virgin binders with increasing temperature, a behavior highly desirable for resistance to permanent deformation. At the same time, the rubber modified binder is expected to be less susceptible to thermal cracking than the polymer and virgin binders owing to lower viscosity than the other two binders at lower temperatures. The above results confirm some of the unique temperature susceptibility properties of the rubber as well as polymer-modified binders in contrast to the virgin binder.

All the three binders at RTFO aged condition had slopes of the viscosity-temperature curves similar to their respective tank conditions, which imply that the temperature susceptibility of the two aging conditions is the same. Despite the effect of aging on the binder, the conventional binder tests are still adequate in describing the viscosity-temperature susceptibility of the binders; and are indicated by the high degree of the coefficient of determination in both cases. Overall, it was observed that the rubber modified binder had the flattest slope amongst the three tested binders, indicating that rubber-modified binder would be least susceptible to viscosity changes across all ranges of low and high temperatures.

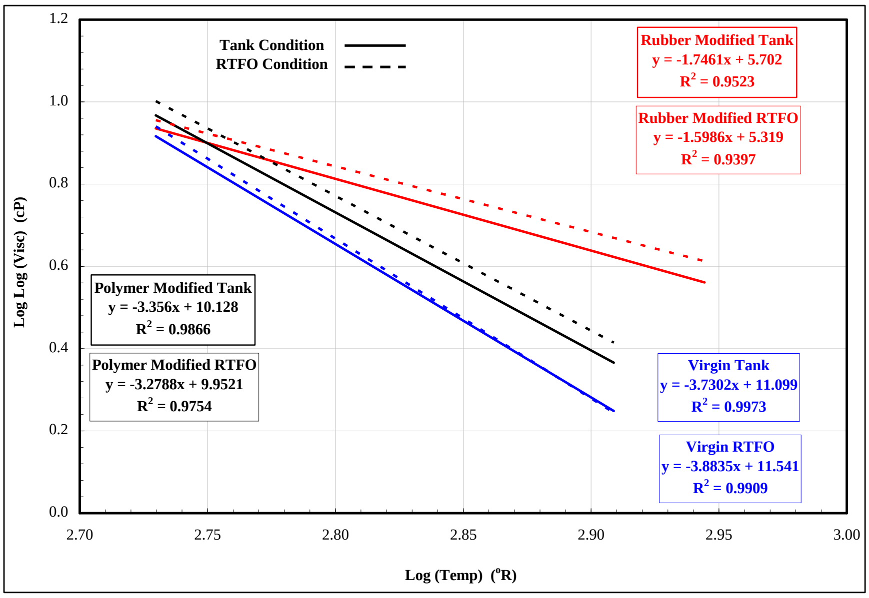

Figure 2. *Viscosity – Temperature Relationship of Stockholm Highway Binders*

5. Triaxial Shear Strength Test

The triaxial shear strength test has been recognized as the standard test for determining the strength of materials for over 50 years (Monismith *et al*, 1975). The results from these tests provide a fundamental basis, which can be employed in analyzing the stability of asphalt mixtures. The shear strength of an asphalt mixture is developed mainly from two sources: first, the cementing action of the binder, which is commonly referred to as "cohesion, *c*" from Mohr plots, and second, the strength developed by the aggregate matrix interlock from the applied loads, commonly referred to as "ϕ" or the angle of internal friction. The major role and interaction of both of these terms varies substantially with rate of loading, temperature, and the volumetric properties of the mixture. Triaxial tests are run at different confining pressures to obtain the Mohr-Coulomb failure envelope. The Mohr-Coulomb failure envelope is defined by:

$$\tau_{ff} = c + \sigma_{ff} tan\emptyset \tag{1}$$

Where,

τ_{ff} = shear stress at failure on failure plane

σ_{ff} = normal stress at failure on failure plane

c = intercept parameter, cohesion

$tan\emptyset$ = slope of the failure envelope (¤ is the angle of internal friction)

Typical "*c*" values for conventional asphalt mixtures are in the range of 5 to 35 psi (35 to 250 kPa); whereas typical "¤" values range between 35 and 48^o. According to the modified sample preparation protocols used in the NCHRP Report 465, a sample size of 100 mm in diameter and 150 mm in height was recommended (Witczak *et al*, 2002). In this study, three triaxial strength tests, one unconfined and two confined were conducted for each of the three gap graded mixtures: reference, polymer modified and rubber modified. These tests provided the "*c*" and "¤" parameters for each mixture. The tests were carried out on cylindrical specimens, 100 mm in diameter and 150 mm in height at 37.8 ^{o}C. In addition to the unconfined test, two additional confining pressures were used: 138 and 276 kPa. The specimens were loaded axially to failure, at the selected constant confining pressure, and at a strain rate of 1.27 mm/mm/min.

Figure 3 shows plots of the Mohr-Coulomb failure envelope represented by the cohesion "*c*" and angle of internal friction "¤" for the three tested mixtures. The larger the "c" value, the larger the mix resistance to shearing stresses. Also, the larger the value of "¤", the larger is the capacity of the mix to develop strength from the applied loads, and hence, the smaller the potential for permanent deformation. The ¤ values for the three mixes were similar to each other with not significant differences in their absolute values; however, the highest ¤ value was observed for the polymer mix (38.7^o) followed by reference (~37^o) and rubber

(~36^{o}) mixtures. The cohesion values for all the three mixtures were significantly different in that the polymer mix had the highest *c* value of about 250 kPa followed by the rubber mix with "*c*" 207 kPa, and then followed by the reference mixture of "*c*" around 160 kPa. The results of the cohesion parameter showed that the polymer mixture had the highest resistance to shearing stresses than the reference and rubber mixes. The angles of internal friction for the three mixtures were very similar, albeit the polymer mixture had the highest value, which is an indication of better resistance to permanent deformation.

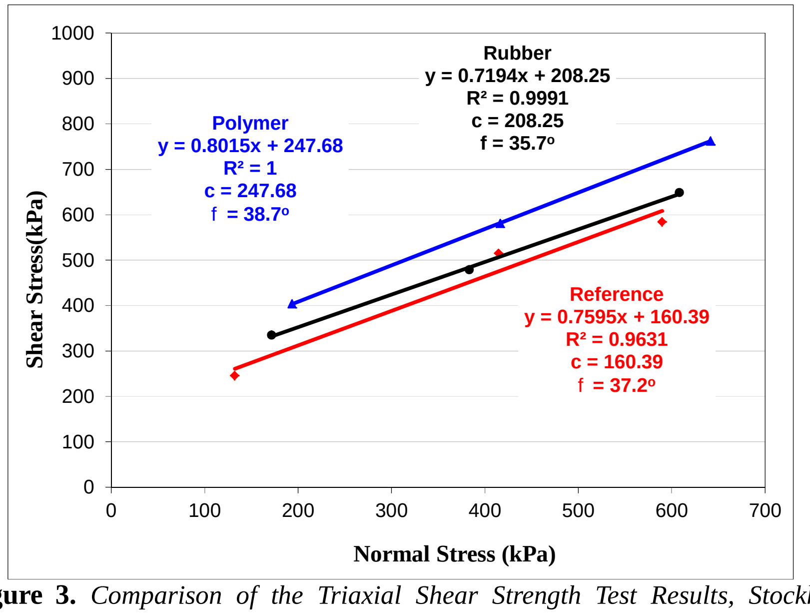

Figure 3. *Comparison of the Triaxial Shear Strength Test Results, Stockholm Swedish Gap Graded Mixtures*

6. E* Dynamic (Complex) Modulus Test

The AASHTO TP 62-07 was followed for E* testing (AASHTO TP62-07, 2007). For each mix, three replicates were used. For each specimen, E* tests were conducted at -10, 4.4, 21.1, 37.8 and 54.4 °C and 25, 10, 5, 1, 0.5 and 0.1 Hz loading frequencies. A 60 second rest period was used between each frequency to allow some specimen recovery before applying the new loading at a lower frequency. The E* tests were done using a controlled sinusoidal stress that produced strains smaller than 150 micro-strains. This ensured, to the best possible degree, that the response of the material was linear across the temperatures used. The dynamic stress levels were 69 to 690 kPa for colder temperatures (-10 to 21.1 °C) and 14 to 69 kPa for higher temperatures (37.8 to 54.4 °C). All E* tests were conducted in a temperature-controlled chamber capable of holding temperatures from –16 to 60 °C. Typical Swedish gap graded test specimen is shown in Figure 4.

Figure 4. *Typical Stockholm Swedish Gap Graded Laboratory Specimen; Sample Dimensions: 100 mm diameter and 150 mm height*

A master curve was constructed at a reference temperature of 21.1 °C using the principle of time-temperature superposition. Figure 5 shows the average E* master curves for the three gap graded mixtures: reference, polymer modified and rubber modified mixes. The figure can be used for general comparison of the mixtures, but specific temperature-frequency combination values need to be evaluated separately. That is, one cannot compare direct values on the vertical axis for a specific log reduced time values. As shown in the above figure, there is not any significant difference between the E* values for the three gap graded mixtures. However, reference mix shows higher moduli values than the two other mixtures at lower temperatures (-10 and 4.4 ^{o}C) while the trend is reversed with further increase in temperature from 21.1 to 54.4 ^{o}C. Lower moduli at cold temperatures are desirable for better resistance of thermal cracking. The increase in moduli values as the temperature increases is also desirable for better resistance to permanent deformation.

The evaluation of modular ratios of polymer and rubber gap graded mixture in contrast to the reference gap graded mix is described below. Modular Ratio (R) of a mix is represented by the following equation.

$$R = \frac{E^*_{mix}}{E^*_{reference}} \tag{2}$$

Where:

R = Modular ratio

E^*_{mix} = Dynamic modulus value for a given mixture

$E^*_{reference}$ = Dynamic modulus value for the reference gap graded mix

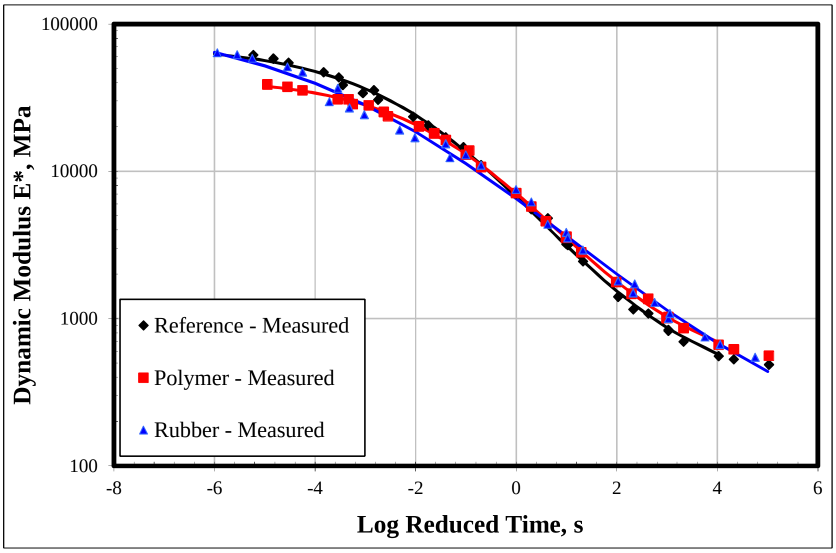

Figure 5. *E* Master Curves for the Stockholm Swedish Gap Graded Mixtures*

The temperature and frequency conditions used for the comparison were 4.4 °C for lower temperatures, and 37.8 and 54.4 °C for higher temperatures. The frequency selected were 10 Hz, representing vehicle speed typical for an arterial street, and 0.5 Hz, representing much slower vehicle speed such as in the case of parking lots or intersections. For E* values at 4.4 °C, the best performance will be that for the mix having lowest E* or R. Conversely, at high temperatures, the best mix performance would be for the highest E* or R. Table 3 shows ratios of dynamic modulus for polymer and rubber mixtures compared to the reference mix.

Table 3. *Comparison of Modular Ratios (R) for E18 Stockholm Swedish Highway*

Conditions	Temp. (°C)	Freq. (Hz)	R = E(Poly.)/(Ref.)	R = E(Rubber)/(Ref.)
High Temperatures at Moderate speed	54.4	10	1.24	1.56
	37.8	10	1.12	1.12
High Temperatures at Low Speed	54.4	0.5	1.17	1.26
	37.8	0.5	1.20	1.20
Low Temperatures at Moderate speed	4.4	10	0.83	0.80
	-10	10	0.64	1.07
Low Temperatures at Low Speed	4.4	0.5	0.88	0.82
	-10	0.5	0.66	1.09

As observed, the modular ratios of rubber gap graded mix with respect to the reference mix was greater than 1 at higher temperatures and the two test frequencies, a desirable characteristic especially for rutting resistance and for all types of loading conditions. A similar finding was observed for the polymer mix in comparison with the reference mix, although polymer mix had lower modular ratios than the rubber-reference combination. Likewise, at lower temperatures, the modular ratios of rubber and polymer mixtures with respect to the reference mix were lower than 1 or very close to 1, also an indication of the rubber-modified or polymer-modified mixtures' better resistance to low temperature cracking. Figure 6 presents comparison of moduli at 37.8 ^{o}C and two loading frequencies for the three gap graded mixtures.

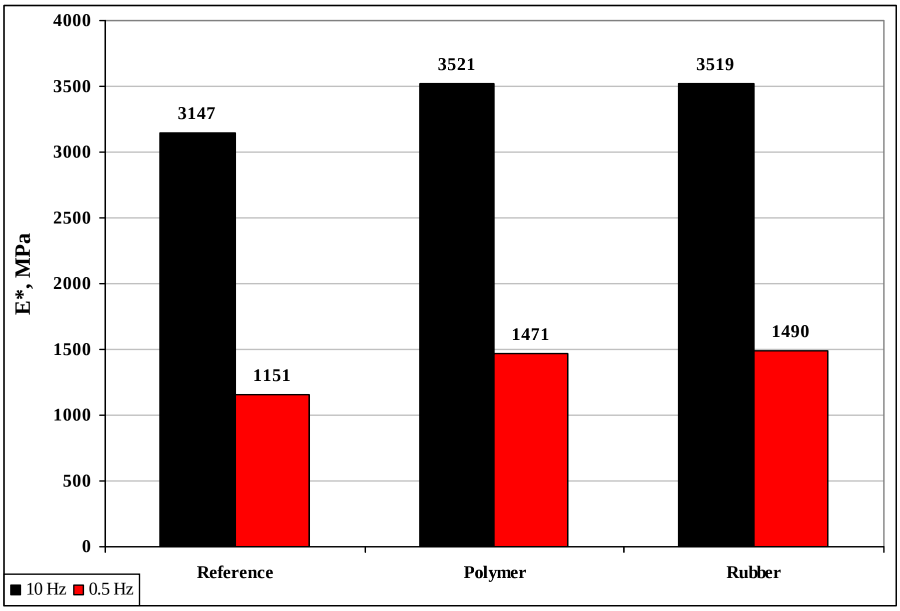

Figure 6. *Comparison of Measured Dynamic Modulus E* values at 37.8 ^{o}C for the Stockholm Swedish Gap Graded Mixtures at 10 and 0.5 Hz*

7. Repeated Load Permanent Deformation Test

The repeated load permanent deformation or Flow Number (FN) test is a dynamic creep test used to determine the permanent deformation characteristics of paving materials. It has been thoroughly documented in the NCHRP Report 465 study (NCHRP 465, 2002). In this test, a repeated dynamic load is applied for several thousand repetitions, and the cumulative permanent deformation, including the beginning of the tertiary stage (defined as FN) as a function of the number of loading cycles over the test period is recorded. FN Tests, confined and unconfined, were conducted using three replicate test specimens for the three mixes: reference, polymer, and rubber mixtures are carried out on cylindrical specimens, 100 mm in diameter and 150 mm in height. A haversine pulse load of 0.1 sec and 0.9 sec dwell (rest time) is applied. All tests were conducted within an environmentally controlled chamber throughout the testing sequence (i.e., temperature was held constant within the chamber to ±0.5 ^{o}C throughout the entire test). Figure 7 (a) and (b) show photographs of actual specimens' set-up for unconfined and confined tests. Repeated load / Flow Number (FN) tests were conducted at unconfined and confined test conditions for reference, polymer and rubber mixtures using at least two replicates per mixture, at 37.8 ^{o}C. Figure 8 presents the Flow Number results for the unconfined and confined tests performed on the three asphalt gap graded mixtures.

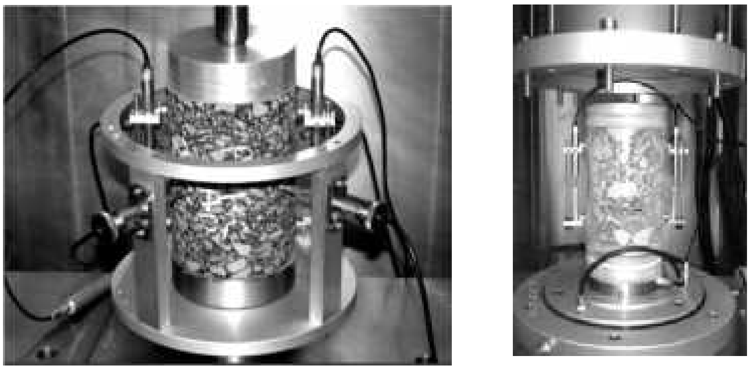

Figure 7. *Flow Number Test Setup (a) Unconfined (left) (b) Confined (right)*

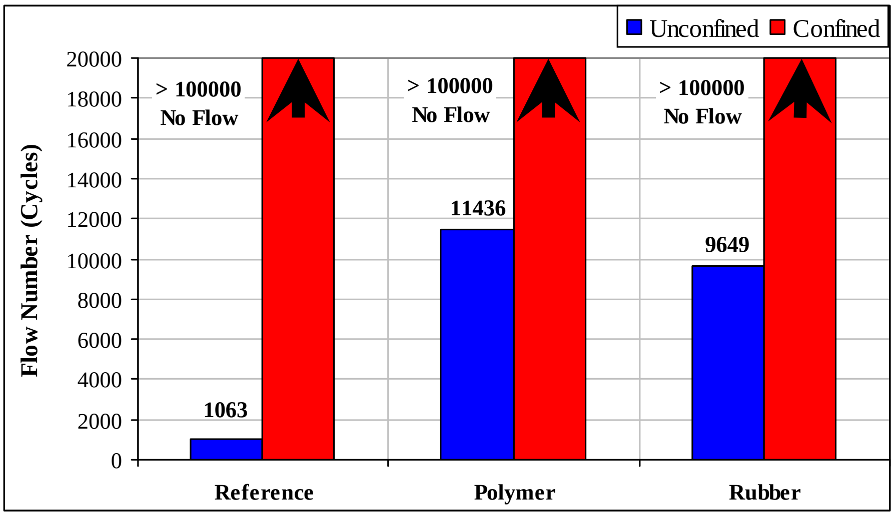

Figure 8. *Flow Number Test Results, Stockholm Swedish E18 Highway*

The results show that on average, polymer and rubber mixtures had higher flow number values than the reference mix. Since the average FN of the polymer and rubber mixtures were about 10 times higher than the reference mix in unconfined state, polymer and rubber mixtures are less susceptible to permanent deformation. It is noteworthy that in confined state, all the three mixtures tested at 138-kPa confinement stress condition had no tertiary flow indicating that these mixtures have highest resistance to permanent deformation. Rubber mixtures at both unconfined and confined stress conditions had 20-50% higher strains at failure than the reference and polymer mixtures.

8. Fatigue Cracking Test

The most common model form used to predict the number of load repetitions to fatigue cracking is a function of the tensile strain and mix stiffness (modulus) as follows (SHRP-A-404).

$$N_f = K_1 \left(\frac{1}{\varepsilon_t}\right)^{K_2} \left(\frac{1}{E}\right)^{K_3} = K_1 (\varepsilon_t)^{-K_2} (E)^{-K_3} \tag{3}$$

Where:

N_f = number of repetitions to fatigue cracking

ε_t = tensile strain at the critical location

E = stiffness of the material

K_1, K_2, K_3 = laboratory calibration parameters

Flexural fatigue tests were conducted according to the AASHTO T321and SHRP M-009 (AASHTO T321-03; SHRP M-009). The flexural fatigue test has been used by various researchers to evaluate the fatigue performance of pavements (Witczak *et al*, 2001; Harvey and Monismith, 1993; Tayebali *et al*, 1995). Figure 6 shows the flexural fatigue apparatus. The device is typically placed inside an environmental chamber to control the temperature during the test. The beams are saw-cut from compacted specimes to the required dimensions of 63.5 mm wide, 50.8 mm high, and 381 mm long.

The air voids for reference mixes were at 5%, and for polymer and rubber mixes the air voids level was 3%. The tests were conducted at 10 Hz and at a constant strain level loading conditions between 325 and 1300 μ strain (at least 5 levels of the strain range was used). The test temperature was 21.1 ^oC for reference and rubber mixes; and 4.4 and 21.1 ^oC for polymer mixes.Initial flexural stiffness was measured at the 50th load cycle. Fatigue life or failure under control strain was defined as the number of cycles corresponding to a 50% reduction in the initial stiffness. The loading was also extended to reach a final stiffness of 30%. The control and acquisition software load and deformation data were reported at predefined cycles spaced at logarithmic intervals.

Figure 9. *Flexible Fatigue Apparatus*

Figures 10 and 11 show comparisons of predicted number of cycles to failure, N_f for a range of applied microstrains using 50 and 30% of initial stiffness for the three mixtures at 21.1 °C. It is observed that the rubber mix has the greatest fatigue life trend, followed by the polymer mix and the reference mix has the least expected fatigue life amongst the three mixtures. Note that the initial stiffness values were not similar across all mix specimens and thus the relationships can be used to compare fatigue data as general trend lines.

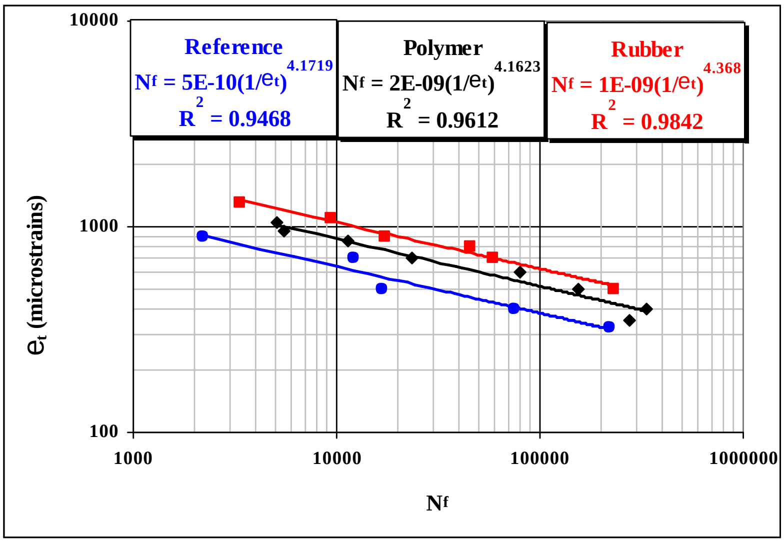

Figure 10. *Comparison of Fatigue Relationships for Three Mixtures at 50% of Initial Stiffness, 21.1 °C*

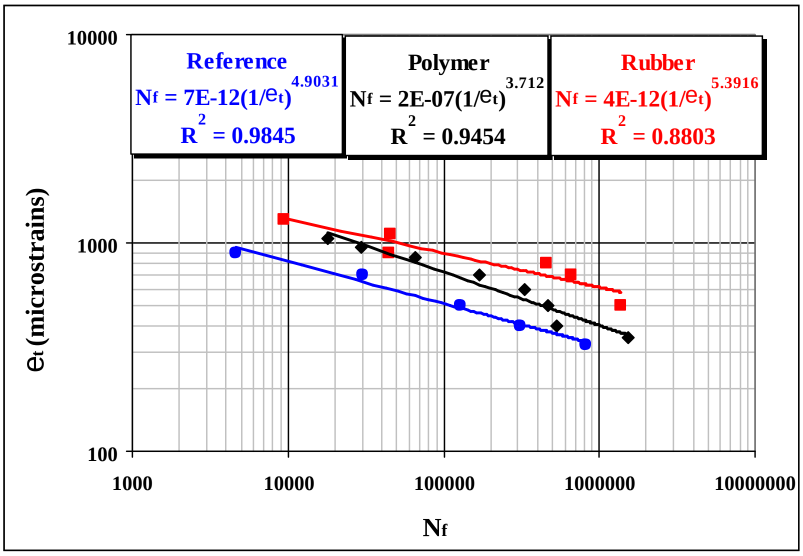

Figure 10. *Comparison of Fatigue Relationships for Three Mixtures at 30% of Initial Stiffness, 21.1 °C*

In another effort, fatigue characterization relationship was developed for the polymer mix since the mix was tested at two test temperatures, 4.4 and 21.1 °C. Equation 3 was used to estimate the regression coefficients K_1, K_2 *and* K_3. The relationships were developed at 50 and 30% of the initial stiffness. Table 4 summarizes the K_1, K_2 *and* K_3 coefficients of the generalized fatigue model for the polymer mixture at 50 and 30% reduction of initial stiffness. Note that the initial stiffness was measured at N = 50 cycles. As observed from the table, the analysis yielded excellent measures of model accuracies.

Table 4. *Comparison of Modular Ratios (R) for E18 Stockholm Swedish Highway*

Parameter	K_1	K_2	K_3	R^2
50% of Initial Stiffness, So @ N=50 Cycles	2.527E-17	6.87776	0.422151	0.9953
30% of Initial Stiffness, So @ N=50 Cycles	2.238E-08	4.96845	0.91901	0.9842

9. Crack Propagation Test – C* Line Integral

The concept of fracture mechanics was introduced to asphalt concrete by Majidzadeh (Majidzadeh, 1976). Abdulshafi applied the energy (C*-Line Integral) approach to predicting the pavement fatigue life using the crack initiation, crack propagation, and failure (Abdulshafi, 1983). Abdulshafi and Kaloush used notched disk specimens to apply J-integral concept to the fracture and fatigue of asphalt pavements (Abdulshafi and Kaloush, 1988). The relation between the J-integral and the C* parameters is a method for measuring it experimentally. J is an energy rate and C* is an energy rate or power integral. An energy rate interpretation of J has been discussed by Rice; and Begley and Landes (Rice, 1968; Begley and Landes, 1972). J can be interpreted as the energy difference between the two identically loaded bodies having incrementally differing crack lengths.

$$J = -\frac{dU}{da} \tag{4}$$

Where,

U = Potential Energy

a = Crack Length

C* can be calculated in a similar manner using a power rate interpretation. Using this approach C* is the power difference between two identically loaded buddies having incrementally differing crack lengths.

$$J = -\frac{\partial U^*}{\partial a} \tag{5}$$

C* can be calculated in a similar manner using a power rate interpretation. Using this approach C* is the power difference between two identically loaded buddies having incrementally differing crack lengths. Where U* is the power or energy rate defined for a load p and displacement u by:

$$U^* = \int_0^{u} \mathrm{pdu} \tag{6}$$

The test samples were prepared according to the Test Protocol UMD 9808, “Method for Preparation of Triaxial Specimens”. The specimens were reheated and compacted with a Servopac gyratory compactor into a 150-mm diameter gyratory mold to approximately 160-mm in height. Approximately 5-mm was sawed from each end of the compacted specimen, and 3 test specimens approximately 38-mm thick were cut from each compacted specimen.

A right-angle wedge was cut into the specimens to accommodate the loading device. A Universal Testing Machine electro-pneumatic system was used to load the specimens. The machine is equipped to apply 25 kN maximum vertical load. The test setup is shown in Figure 11. All tests were conducted at 21.1 °C.

The experimental testing involves collecting the data as load and crack length versus time for a constant displacement rate. The displacement rates used were 0.30, 0.45, 0.60, 0.75, and 0.90 mm/min for all the three gap graded mixtures. This information is used to determine load as a function of displacement rate for various crack lengths, and crack growth rate versus crack length. The power of energy rate input, U*, is measured as the area under the load displacement rate curve. The energy rate, U*, is then plotted versus crack length for different displacement rates and the slopes of these curves constitute the C*-integral. The C*-integral is plotted as a function of the displacement rate. Finally, the crack growth rate is plotted as a function of C* integral.

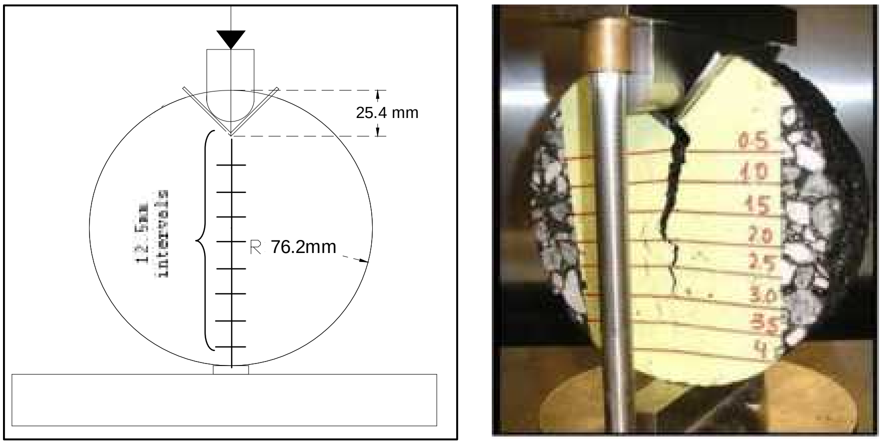

Figure 11. *Typical C* Test Setup*

Figure 12 shows relationships between crack growth rates and C* values for the three mixtures. Figure 13 shows relationships between slope values of C* versus crack growth rates for the three mixtures. It is observed that the slope value for the rubber mix is almost double that of the reference mix, and almost 6 times higher than that of the polymer mix. In other words, the energy difference required to bring the rubber modified mix from a low crack growth rate to a higher rate is much higher than the other two mixes. This is seen by the small values of crack growth rate obtained for the rubber mixes as opposed to the polymer mixes. From this comparison, it was seen that the rubber mix has the highest potential to resist cracking out of the three mixtures.

During testing, the polymer mix exhibited a higher force to initiate cracking, but once the initial crack had originated, the extent of the crack grew far more rapidly than the other two mixes. The total energy required to propagate the crack was analyzed for all the three mixes at different load displacement rates, and it was found that the rubber mixes required higher energy to form and propagate a crack of 60 mm. These results also confirmed the initial findings based on the C* versus crack growth rate values.

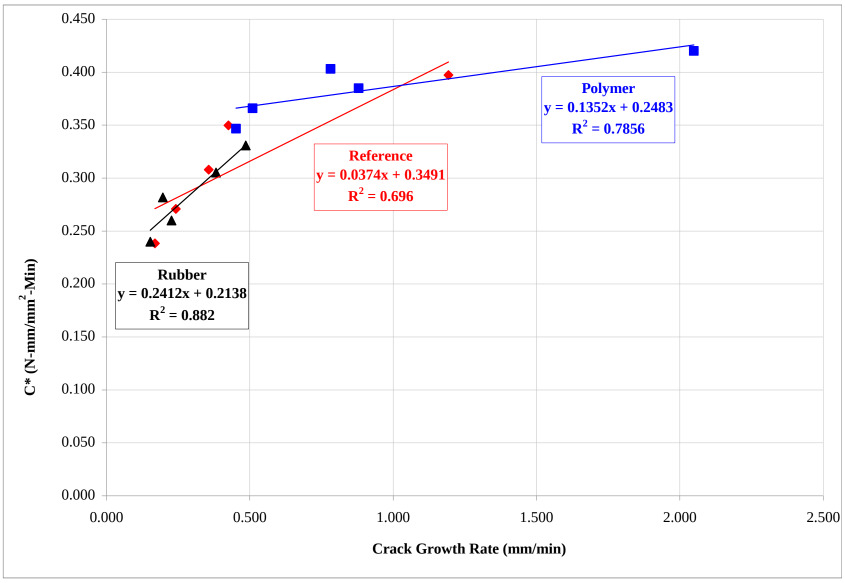

Figure 12. *Crack Growth Rate versus C* for the Three Mixes, Stockholm Highway*

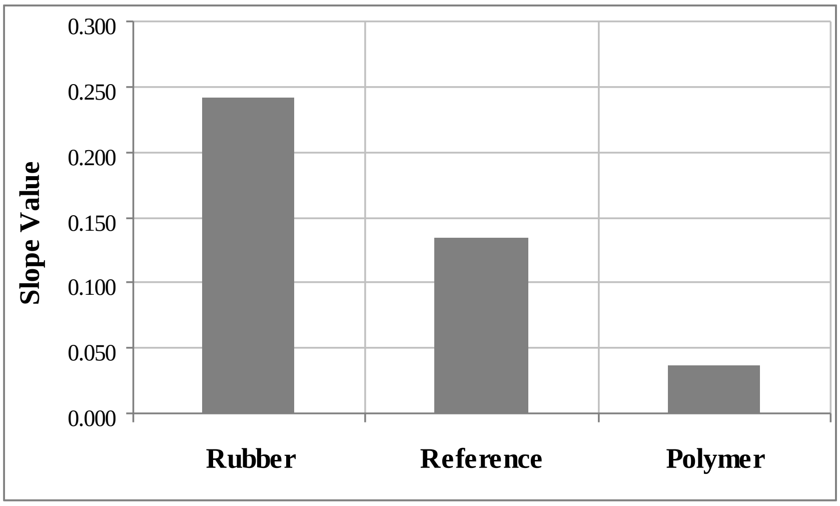

Figure 13. *Slope Values of C* versus Crack Growth Rates for the Three Mixes, Stockholm Highway*

10. Thermal Cracking

Tensile creep and strength test data are material inputs required for the Mechanistic Empirical Pavement Design Guide (MEPDG) Level 1 and 2, when a thermal fracture analysis is desired. Creep compliance data is used to predict field tensile stress development in the asphalt concrete layers as a result of temperature cycling. A fracture mechanics based crack tip model then estimates downward the thermal crack development as a function of time, which is in turn used to compute the amount of thermal cracking versus time based upon a probabilistic crack distribution model (Witczak *et al*, 2000; Witczak, 2003). The material inputs required for the fracture model are the tensile strength (at –10 ^{o}C) and the m-value (Roque *et al*, 2002). The tensile strength is directly obtained from the indirect tensile strength test. The m-value is related to the slope of the creep compliance master curve, and is computed in the MEPDG using compliance data obtained from the indirect tensile creep test.

Tests were conducted using three replicates at three temperatures: -15, -10, and 0 ^{o}C. The required nine replicates were obtained from three gyratory compacted plugs. Each group of replicates (according to temperature) contains one specimen from every gyratory compacted plug to ensure unbiased test results. Based on the results from the three test temperatures, data was extrapolated to obtain creep compliance parameters for temperature of -20 ^{o}C. Figures 14 present plots of the creep compliance master curves for the three mixtures. Higher creep compliance values were exhibited by the rubber mixtures followed by the reference and polymer mixtures. High creep compliance values are desirable from the thermal cracking point of view.

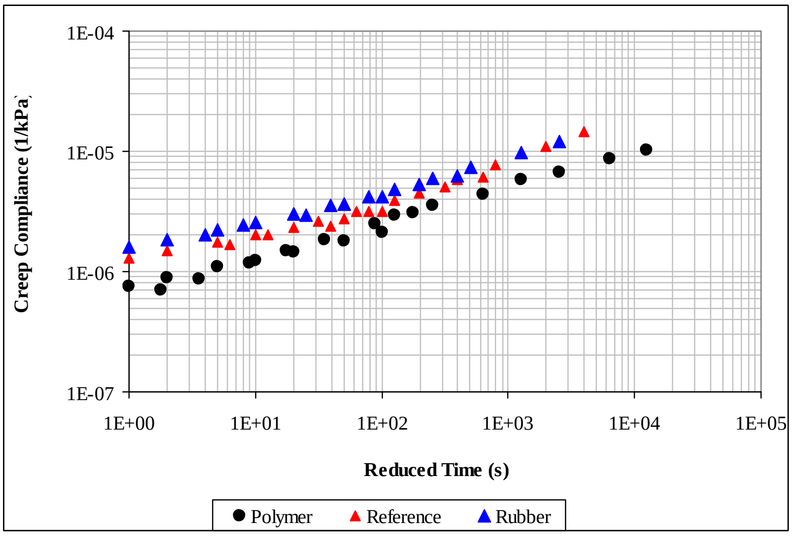

Figure 14. *Creep Compliance Master Curves for the Three Mixes.*

Figure 15 shows the fracture energy, which decreased with decreasing temperature for all the three mixtures. The rubber mixture had the highest total fracture energy than the other two mixtures at 0 °C (~1.5 to 1.7 times higher), and about 10% higher values at the other two lower temperatures. At the highest temperature (0 °C), the rubber mix exhibited the highest fracture energy; the difference being about 60-80% when compared to the other two mixtures. At the immediate lower temperature of -10 °C, a similar trend was observed with a difference of 25% of fracture energy between the mixtures. At -15 °C, the same trend was observed, the fracture energy difference was close to 5-10%. Lower thermal cracking should be expected as the energy at failure or fracture energy is increased.

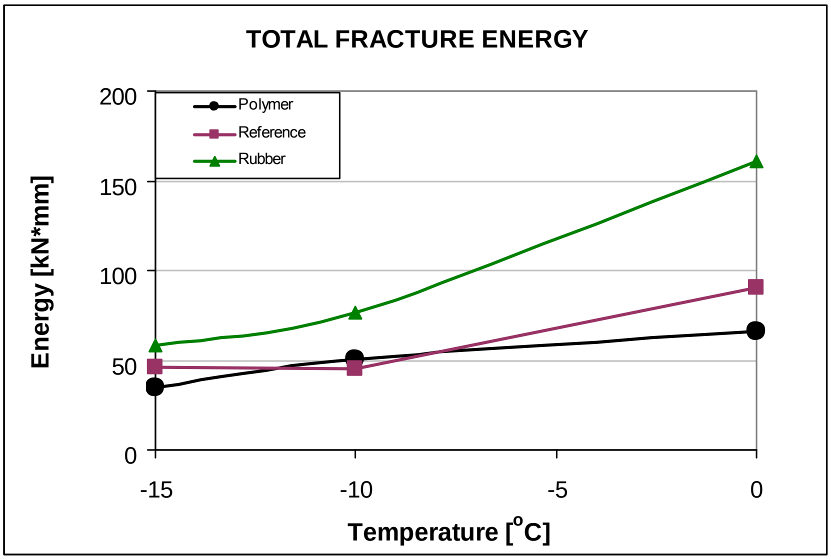

Figure 15. *Fracture Energy Comparison for the Three Mixes.*

11. Conclusions

The material characterization tests results in this study showed that the crumb rubber gap graded mix provided improved performance over the polymer modifed and reference gap graded mixtures in several unique ways. The binder consistency testing results revealed that crumb-rubber modified binder would be least susceptible to viscosity changes across all temperature ranges.

The results of the triaxial shear strength tests indicated that there was not a significant difference between the different mixtures' shearing parameters values, albeit the polymer mixture had the highest value in terms of magnitude of the shearing properties. The dynamic modulus tests indicated that the rubber modified gap graded mix would provide better resistance to low temperature cracking (softer modulus at lower temperatures) and to permanent deformation (stiffer modulus at higher temperatures). The flow number test showed that the rubber and polymer modified mixtures had 20-50% higher performance than the reference mix. In terms of fatigue life, the rubber modified mix exhibited better fatigue life than the polymer and reference mixes. The C* Integral tests revealed that the rubber modified mix had higher resistance to crack propagation than the reference and polymer modified mixes. Higher creep compliance and total fracture energy values were also exhibited by the rubber indicating better thermal cracking performacne. In conclusion, the labortory tests indicated that the rubber mixture will have the best field performance. Future follow up field evaluation should validate the finding of this laboratory study.

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