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Contents

Foreword

The papers in this volume, dealing with various facets of recycled tire rubber and other waste materials in asphalt mixtures, should be of interest to state and local construction, design, materials, and research engineers as well as contractors and material producers.

In the first papers, Rebala and Estakhri, Malpass and Khosla, and Baker and Connolly describe research related to crumb rubber modified mixtures that was done for the Texas, North Carolina, and New Jersey State Departments of Transportation. Ali et al. report on their research in Canada to determine the feasibility of using reclaimed roofing materials in hot mix asphalt pavement. Emery discusses the evaluation of 11 Ontario rubber modified demonstration projects in terms of pavement performance, environmental impacts, and recyclability. Lesueur et al. describe their work comparing carbon black derived from pyrolized tires to other filler as asphalt rheology modifiers. Morrison et al. discuss the modification of asphalt binders with crumb and devulcanized waste rubber. In the last paper, Fwa and Aziz report on their work in Singapore related to the use of incinerator residue in asphalt mixtures.

Laboratory Evaluation of Crumb Rubber Modified Mixtures Designed Using TxDOT Mixture Design Method

SEKHAR R. REBALA AND CINDY K. ESTAKHRI

The Texas Department of Transportation has developed a mixture design procedure for crumb rubber modified (CRM) asphalt concrete mixtures. Eight CRM mixtures were designed using this procedure. Four wet process mixtures and four dry process mixtures, in addition to one control mix, were considered for material characterization and performance evaluation using the asphalt aggregate mixture analysis system (AAMAS). The control mixture was designed using the conventional design method. It was determined that CRM has the potential to significantly improve the fatigue cracking performance of asphalt concrete pavements, but only when the wet method is used and the binder is properly designed. The dry process should produce mixtures with a reduced propensity for rutting, but may have an adverse effect on cracking. Fine and coarse rubber can be added dry to the dense graded mix (0.5 percent by weight of aggregate) without having any adverse effects on the performance. Although state transportation departments must comply with the existing legislative requirements, tire rubber, like any additive, should be used only to address a given mixture deficiency or expected deficiency in a given situation.

The use of crumb rubber modified asphalt concrete in paving applications has been a practice for many years. Legislation by federal and state governments required states to develop new procedures for effective utilization of crumb rubber in pavements. Several crumb rubber modified (CRM) pavements were constructed on an experimental basis in the state of Texas (*1*). Because there was a great deal of variation in the success and performance of these pavements, the Texas Department of Transportation (TxDOT) developed a new mixture design method for CRM hot-mix asphalt concrete based on the stone matrix concept. A research program was undertaken at Texas Transportation Institute (TTI) to evaluate the CRM mixtures for various distresses and to develop construction guidelines for their application.

RESEARCH PROGRAM

The research program undertaken at TTI consists of three phases. Only the results from Phase 2 of the research program are discussed here. The three phases of the program are

- Phase 1: laboratory evaluation of unmodified and modified CRM mixtures. The objectives included preparation of CRM binders to be used in hot-mix asphalt concrete; characterization of these binders and determination of appropriate test procedures for the binders.
- Phase 2: laboratory evaluation of CRM asphalt concrete. Asphalt Aggregate Mixture Analysis System (AAMAS) (*2*) was selected as a tool to evaluate CRM mixtures. Objectives include the determination of optimum rubber content that can be used in dense-graded mixtures and checking the adequacy of the CRM mixtures to be used in the surface layer of a flexible pavement.
- Phase 3: field evaluation of CRM mixtures and developing usage and construction guidelines.

Phase 1 of this project is completed and reported elsewhere (*3*). Phase 3 of this project is currently in progress at TTI. The scope of Phase 2 includes two gradations of crumb rubber (No. 10 size and No. 80 size), two aggregate gradations (dense and open), two rubber contents (10 percent and 18 percent), and two methods of incorporating rubber (wet and dry) in the mixture (a total of nine mixtures). All of the mixtures are shown in Table 1.

MATERIALS

Aggregates

The aggregates used for the mixture designs included a crushed limestone and field sand. Care was taken to eliminate the influence of aggregate properties in the evaluation process by obtaining aggregate from the same source. The crushed limestone was from Giffordhill in New Braunfels, Texas. Limestone screenings were from Georgetown, Texas, and the field sand was from a source near Hearne, Texas. Specific gravities of the aggregate are

- Bulk specific gravity of coarse limestone: 2.554;
- Bulk specific gravity of limestone screenings: 2.443; and
- Bulk specific gravity of field sand: 2.551.

Asphalt Cement

The asphalt cement used for this study was Texaco AC-10 from Port Neches, Texas. This asphalt was used for the design of all the mixtures, including the control mixture. All the binder properties are reported in Estakhri et al. (*3*). No additives were used in the blending process of asphalt cement with rubber.

Rubber

The rubber used in this study is from ground, whole tires. Two different sizes of CRM were used in this study: No. 10 mesh and No. 80 mesh. The source of rubber passing the No. 80 sieve size was

S. R. Rebala, Texas Department of Transportation, Wellington, Tex. 79095. C.K. Estakhri, Texas Transportation Institute, Texas A&M University System, College Station, Tex. 77843.

TABLE 1 Gradation, Binder Content, and Mixture Designation for Control and CRM Mixtures

Sieve Size	Dense-Graded Mixtures			Open-Graded Mixtures					
	0% CRM Control Mix	0.5%[1] Fine CRM (DGF)	0.5%[1] Coarse CRM (DGC)	10%[2] Fine CRM Wet Process (10FW)	10%[2] Coarse CRM Wet Process (10CW)	18%[2] Fine CRM Wet Process (18FW)	18%[2] Coarse CRM Wet Process (18CW)	18%[2] Fine CRM Dry Process (18FD)	18%[2] Coarse CRM Dry Process (18CD)
Optimum Binder Content (%)[1]	5	5	5.75	8.2	8.2	8.2	8.2	8.2	8.2
	% Passing								
1/2"	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3/8"	92.0	92.0	92.0	98.0	98.0	98.0	98.1	97.9	98.2
#4	60.0	60.0	60.0	42.5	43.7	41.1	46.1	42.2	49.0
#10	37.0	37.0	37.0	9.5	11.7	10.0	14.9	8.7	19.5
#40	21.0	21.0	21.0	7.5	9.2	8.5	11.6	6.8	15.1
#80	9.0	9.0	9.0	4.7	5.4	5.8	6.27	3.7	8.2
#200	4.0	4.0	4.0	3.0	3.3	4.5	3.75	2.1	4.6

1 - By weight of the aggregate
2 - By weight of asphalt cement

Rouse Rubber of Vicksburg, Mississippi. The source of rubber, passing the No. 10 sieve size, was from Granular Products of Mexia, Texas. Rubber passing No. 10 sieve size will hereafter be referred to as coarse rubber and rubber passing No. 80 sieve size as fine rubber.

MIXTURE DESIGN

Two methods were used for designing crumb rubber mixtures for laboratory characterization: TxDOT's standard method for mixture design (C-14) *(4)* and TxDOT's method for crumb rubber mixtures (Tex-232-F) *(4)*.

One of the simplest and most economical methods of incorporating CRM into asphalt mixtures is by using a generic dry process, that is, by adding the CRM to the mixture as a part of the aggregate rather than preblending it with the asphalt cement. This process was evaluated along with the dense-graded aggregates. It is believed that one of the major concerns with using CRM in dense-graded mixtures is the concentration of CRM. In the past, concentration of CRM has typically been too high for this type of gradation (18 percent CRM or more, by weight of the binder). Therefore, standard mixture design procedures *(4)* were used to determine how much CRM could be added to a Texas Type D mixture (dry) still maintaining all requirements associated with standard mixture design (acceptable air voids and Hveem stability values).

Dense-graded mixtures were designed using two different gradations of CRM (No. 10 mesh and No. 80 mesh) and varying the concentration of CRM. Three concentrations of CRM were evaluated: 0.2, 0.5, and 0.8 percent CRM by weight of the aggregate. Of these three concentrations, it was found that 0.5 percent was the optimum concentration of CRM for dense-graded mixtures. These mixtures were then characterized by using AAMAS *(2)*. Exact gradations, optimum binder contents, and mixture designations are given in Table 1.

Tex-232-F is a volumetric mixture design procedure for designing gap-graded crumb rubber mixtures. The mixture design philosophy is that crumb rubber particles fill the available voids and still maintain a stone-to-stone contact. The design criteria are a minimum of voids in the aggregate 20 percent; optimum laboratory density of 97 percent; and a minimum 17 percent volume of binder (asphalt cement + rubber). Preblending CRM and asphalt prior to incorporation into the hot-mix is known as the wet process. Two different percentages of rubber were added by the wet method, they are: 10 percent (coarse and fine rubber) and 18 percent (coarse and fine rubber) by weight of asphalt content. If rubber is added directly to the aggregate, it is called dry process. Two generic dry mixtures were also designed according to this procedure: 18 percent fine CRM and 18 percent coarse CRM (by weight of asphalt). For comparison purposes, the dry CRM concentrations are expressed herein as a percent of the asphalt content. Tex-232-F was originally developed to incorporate crumb rubber by the wet process, but these dry mixtures are also designed according to test method Tex-232-F and evaluated using AAMAS. Final gradations for all six mixtures mentioned are given in Table 1.

PREPARATION OF SAMPLES FOR MIXTURE EVALUATION

Using the gradations and binder contents given in Table 1, samples were prepared for the AAMAS testing program. Procedures described in *Standard Specifications for Construction of Highways, Streets and Bridges* *(4)* were strictly followed. Mixtures were kept in a forced draft oven at 135°C (275°F) for 3 hours to simulate the plant aging. All of the samples were molded using a California kneading compactor, the only modification in the fabrication procedure. This change was made because the AAMAS-recommended gyratory testing machine is not available at TTI. So it was decided by the research team that the traffic densified samples be molded with a California kneading compactor for creep testing. The samples to be tested for permanent deformation properties were molded to an air void level of 2 to 3 percent to maintain the uniformity. CRM samples were allowed to cool to room temperature before extracting from the mold.

Some of the samples molded for testing were conditioned to simulate loading and environmental conditions. Other samples were conditioned to simulate moisture damage, temperature conditioning, and traffic densification. Unconditioned samples were also tested for the material characterization.

TESTING PROGRAM

All nine mixtures were evaluated using AAMAS. The test matrix is shown in Table 2 for each mixture considered. A total of 243 samples were molded for the testing program. The tests performed were resilient modulus, indirect tensile strength, indirect tensile creep, and compressive static creep. At a later stage in the program, dynamic creep test was added. This test was also performed at the same load level as static creep test, 414 kPa (60psi). A Poisson ratio of 0.35 was assumed for all the mixtures in calculating resilient modulus, indirect tensile strain, and strength. Dynamic creep tests were performed for 10 000 cycles; each cycle consisted of 0.1 seconds loading and 0.9 seconds unloading. These material properties were used in various distress models to predict the performance of the mixtures.

DISCUSSION OF TEST RESULTS

Resilient Modulus Test

Diametral resilient modulus tests were performed at three temperatures: 5°C (41°F), 25°C (77°F), and 40°C (104°F). These data are shown in Figure 1. The addition of CRM in the dense-graded mixtures caused a decrease in the resilient modulus at 5°C (41°F) and 25°C (77°F) compared to the control. All of the gap-graded crumb rubber mixtures had lower stiffness than the dense-graded mixtures at all three test temperatures. It appears that CRM may have some propensity for decreasing the mixture's temperature susceptibility, particularly the mixture made with 18-percent fine CRM by the wet process (18 percent FW). This mixture had the lowest stiffness at 5°C (41°F) and yet a relatively high stiffness at 40°C (104°F). This trend was not observed, however, for the two gap-graded mixtures made with 10 percent CRM (10 percent CW and 10 percent FW), which exhibited the lowest stiffness of all the mixtures at 40°C (104°F). This may be due to higher binder content in 10 percent CRM than for 18 percent CRM. It should be noted here that the volume of the binder remained the same for both 10 and 18 percent CRM, so the net asphalt content for 10 percent CRM is 0.5 percent higher than for 18 percent CRM.

Figure 1 is the AAMAS chart for plotting the test results of total resilient modulus (unconditioned) versus temperature compared with the range of values that are appropriate for higher volume roadways. In general, the gap-graded crumb rubber mixtures have resilient modulus values that are considered to be too low based on this particular criterion. However, this criterion was developed for dense-graded asphaltic concrete mixtures and may not be applicable to gap-graded mixtures.

Indirect Tensile Strength Test

Indirect tensile strength tests were performed at 5°C (41°F,) 25°C (77°F), and 40°C (104°F). Figures 2 and 3, respectively, show the indirect tensile strengths and failure strains at 5°C (41°F). The tensile strength of the control mixture, which is a dense-graded (DG) mix, is 827 kPa (120 psi). The addition of dry fine and coarse rubber to a dense-graded mixture (DGF and DGC) did not cause a decrease in the tensile strength. Tensile strength and failure strain data at 5°C (41°F) for all three dense-graded mixtures (control, DGF, and DGC) were about the same. The remaining crumb rubber mixtures (which were gap-graded) exhibited a decrease in tensile

TABLE 2 Laboratory Test Plan for Control and CRM Mixtures

Test	Conditioning of Samples To Simulate Various Types of Distress					
	Unconditioned			Moisture Conditioning	Temperature Conditioning	Traffic Densification
Temperature, °C (°F)	5 (41)	25 (77)	40 (104)	25 (77)	5 (41)	40 (104)
Resilient Modulus	X	X	X	X	X	X
Indirect Tension Test	X	X	X	X	X	-
Indirect Tension Creep test	-	-	-	-	X	-
Compressive Static Creep Test	-	-	-	-	-	X
Compressive Dynamic Creep Test	-	-	-	-	-	X
Compressive Strength Test	-	-	-	-	-	X

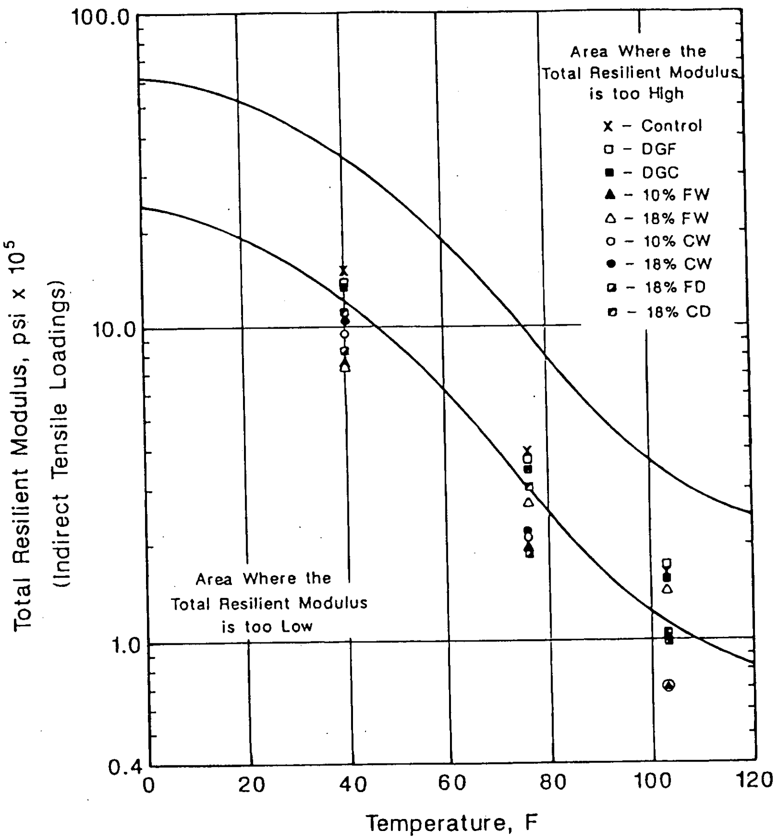

FIGURE 1 AAMAS plot showing resilient modulus versus temperature for control and CRM mixtures.

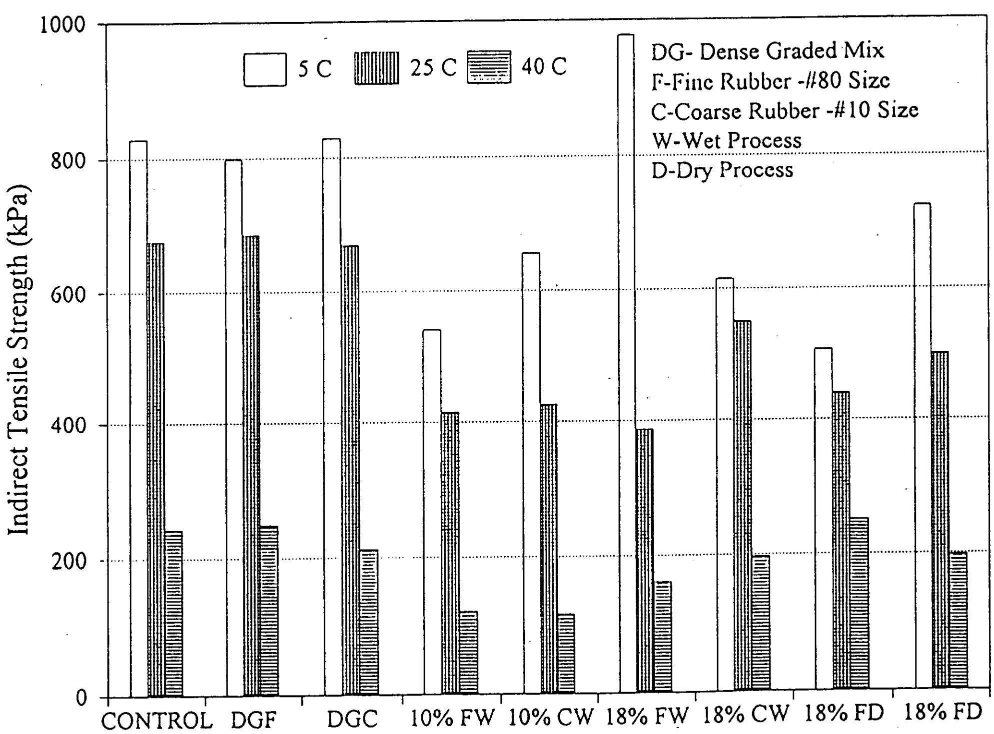

FIGURE 2 Indirect tensile strength for control and CRM mixtures at three different temperatures.

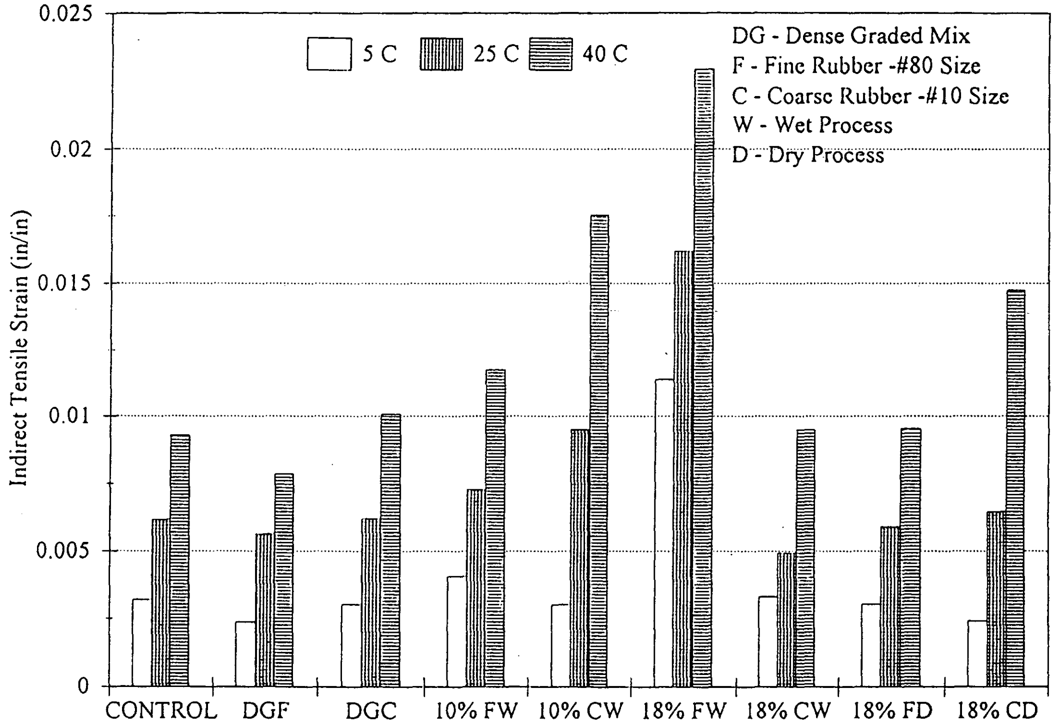

FIGURE 3 Indirect tensile strain for control and CRM mixtures at three different temperatures.

strength ranging from 15 to 35 percent except the 18 percent FW. This mixture displayed a significant increase in tensile strength over the control mixture. Tensile strain at failure was also much higher for 18 percent FW when compared to the other mixtures.

While the tensile strengths for 5°C (41°F) decreased for most of the gap-graded rubber mixtures as compared with the control, there was no decrease in the tensile strain at failure for these mixtures. In fact, the tensile strain at failure for the gap-graded rubber mixtures was as good or better than the control in most cases.

Tensile strengths and failure strains at 25°C (77°F) follow a similar trend in the data as observed for 5°C (41°F). As one would expect, tensile strength of bituminous mixtures greatly decreases at 40°C (104°F). The two gap-graded mixtures made with 10 percent FW and 10 percent CW had the lowest tensile strengths. This may be due to the higher net asphalt content available than any other mixture. Failure strains for the gap-graded crumb rubber mixtures were generally higher than the control and dense-graded crumb rubber mixtures. One mixture seemed to stand-out from all others in terms of having significantly higher failure strains at all three test temperatures: the gap-graded mixture 18 percent FW. The properties of the binder used in this mixture (18 percent FW) also support this conclusion and are presented in Estakhri et al. (*3*).

Creep Test

Static Compressive Creep Test

Static compressive creep tests were performed at 40°C (104°F) at 414 kPa (60 psi) with 1 hour loading period and 1 hour recovery period. Total compressive creep modulus was calculated at different times. Besides the creep modulus criteria, there are other parameters measured in this creep test worth discussion: total creep strain, log-log slope of the steady state creep curve, and strain recovery.

Creep strain, for a given stress level, is plotted versus time, and the creep strain is divided into three stages (*5*). In the first, or primary, stage, the rate of deformation increases rapidly. In the second, or "steady state," region, the deformation rate is constant as is the angle of slope (rate of deformation). The third is the failure stage (tertiary), in which the deformation again increases rapidly.

The AAMAS uniaxial creep curves for all of the mixtures are presented in Figure 4. None of the mixtures appeared to reach the tertiary creep region within the 1-hour loading period. Uniaxial creep data for all the mixtures are shown in Table 3. A log-log slope of the creep versus time of loading curve (as shown in Table 3) of less than 0.25 is indicative of a mixture that will not become unstable (reach tertiary creep) within the testing period of 3,600 sec (*6*). All of the mixtures shown here have a slope less than 0.25. When observing the curves in Figure 4 and the slope data in Table 3, it appears that some mixtures have significantly higher slopes than others. However, each of these slopes represents an average of three tests. A statistical analysis performed on these data revealed that none of these mixtures is significantly different from the other in terms of slope of the creep curve in the steady-state region.

After the one hour loading period in the creep test, there is a 1-hour recovery period. The percent strain recovered at the end of the 1-hour recovery is shown in Table 3. It appears from this data that the two dense-graded CRM mixtures (DGF and DGC), which have a similar gradation as the control, have much better recovery than the control. The dense-graded fine CRM mixture had a higher creep stiffness at 3,600 sec than the dense-graded coarse CRM mixture. However, the dense-graded coarse CRM mixture had a better recovery. Of the gap-graded rubber mixtures, 10 percent FW, 10

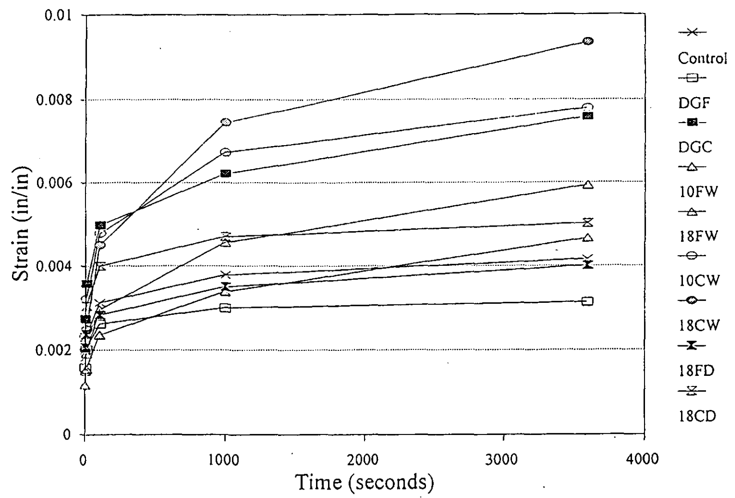

FIGURE 4 Plot showing strain versus time for control and CRM mixtures for uniaxial static creep test.

percent CW, and 18 percent CW had better recovery than the control mixture.

Based on the total strain at the end of the test, the mixtures were grouped into three categories in order to rank the mixtures from best to worst. The following criteria were arbitrarily selected for categorizing the mixtures using the total strain at the end of the static creep test.

- Category 1: total strain < 0.005 in./in.,
- Category 2: total strain between 0.005 and 0.0075 in./in., and
- Category 3: total strain between 0.0075 and 0.010 in./in.

Category 1 mixtures were the best in terms of total strain at 1 hour. It should again be emphasized that these criteria were arbitrarily selected for ranking the mixtures for comparison with each other (see Table 4).

Repeated Load Uniaxial Creep Testing

The repeated load uniaxial creep test was performed to more closely simulate wheel loading than the static creep test. The primary difference between a repeated load creep test and a static test is the plastic deformation that occurs between loading applications. Deformation is highly dependent on number of cycles. This permanent deformation or relative movement among particles is most effectively produced under dynamic loading conditions as the dynamic effect of each repetition produces some level of relative movement.

All of the creep tests performed in this study (both repeated and static load creep), were performed without confining pressure. The gap-graded rubber mixtures analyzed in this study were very similar in gradation to a stone mastic type of mixture. When creep tests are performed on these mixtures without confining pressure, the

TABLE 3 Uniaxial Static and Repeated Load Creep Data (σ_1 = 414 kPa (60 psi)) for Control and CRM Mixtures

Mixture Type	Static Creep Test			Repeated Load Creep Test		
	Log-Log Slope of Steady state Creep Curve	Strain @ End of 3600 Seconds, (in/in)	Strain Recovery, %	Log-Log Slope of Steady State Creep Curve	Strain @ End of 10000 Cycles, in/in	Strain Recovery, %
Control	0.078	0.004153	13	0.423	0.007079	1.4
DGF	0.030	0.003145	21	0.360	0.004149	3.0
DGC	0.067	0.005227	34	0.313	0.011609	3.0
10% FW	0.111	0.005033	9	0.463	0.016657	0.3
10% CW	0.098	0.007767	4	0.428	0.018085	0.7
18% FW	0.165	0.005931	20	0.324	0.014358	1.8
18% CW	0.180	0.009015	3	0.356	0.015528	2.8
18% FD	0.072	0.003383	29	0.365	0.013312	2.3
18% CD	0.049	0.005010	15	0.328	0.018772	4.9

TABLE 4 Ranking of Laboratory Mixtures (from Best to Worst) Based on Total Strain at End of Test Period for Both Static and Repeated Load Creep Tests

Ranking Category	Based on Static Creep Test	Based on Repeated Load Creep Test
Category 1	DGF Control 18%FD	DGF Control
Category 2	DGC 18%FW 18%CD 10%FW	18%FD DGC 18%FW
Category 3	10%CW 18%CW	18%CD 10%FW 10%CW 18%CW

mixtures may lack the lateral support that is present in the field. It is believed that because of the aggregate interlock that exists in a dense-graded mixture, unconfined uniaxial creep properties may be better for dense-graded mixtures than for stone mastic-type mixtures. However, field performance may be better for the latter. This important factor must be kept in mind when reviewing unconfined uniaxial creep properties for both dense- and gap-graded mixtures. It is appropriate to compare or rank gap-graded mixtures against each other, but it may not be appropriate to compare gap-graded mixtures to dense-graded mixtures.

The repeated load uniaxial creep curves for all the mixtures are presented in Figure 5. Tabulated data are shown in Table 3. The samples were subjected to 10 000 load cycles in the repeated load creep test. As shown in Figure 5, the dynamic loading causes the samples to deform at a much higher rate (as evidenced by the greater slope of the secondary portion of the curve) than in the static creep test. The slopes for these mixtures are also shown in Table 3. As in the static creep test, none of these mixtures is significantly different (statistically) from the other in terms of slope in the steady state portion. The percent strain recovery for all the mixtures (Table 3) is very low, which is probably to be expected after 10 000 load cycles.

As with the static creep data, the mixtures were categorized and ranked according to the total strain at the end of 10 000 load cycles. Based on the total strain at the end of the test, the mixtures were grouped into three categories. The criteria arbitrarily selected for categorizing the mixtures using the total strain at the end of the static creep test were

- Category 1: total strain < 0.010 in./in.,
- Category 2: total strain between 0.010 and 0.015 in./in., and
- Category 3: total strain between 0.015 and 0.020 in./in.

Based on these criteria the laboratory mixtures can be categorized as above in Table 3 (Category 1 mixtures being the best in terms of total strain) and compared with the rankings of the mixtures based on static creep tests. Because the repeated load creep test is more rigorous than the static creep test, some mixture rankings changed slightly. Both the DGF and control mixtures remained in Category 1 after repeated load creep testing; however, the 18 percent FD dropped to Category 2. The DGC and 18 percent FW remained in Category 2 while the 18 percent CD and 10 percent FW dropped to Category 3. Mixtures designated as 10 percent CW and 18 percent CW remained in Category 3.

PERFORMANCE EVALUATION OF MIXTURES

Check for AASHTO Structural Layer Coefficient

The only material property that is considered in designing flexible pavements is resilient modulus at 20°C (68°F) in the form of structural number. As layer coefficient decreases, the thickness required for a particular structural number increases. Layer coefficient is directly proportional to resilient modulus. From Figure 1, clearly, all CRM mixtures have resilient modulus values lower than dense-graded mixtures. So the thickness required for CRM mixtures will be much higher than required for dense-graded mixtures for a particular traffic level.

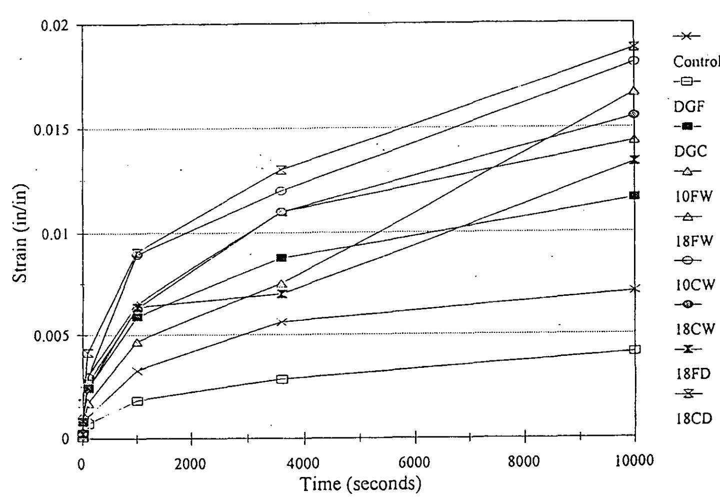

FIGURE 5 Plot showing strain versus time for control and CRM mixtures for uniaxial repeated load creep test.

Check for Resistance to Fatigue Cracking

Figure 6 presents the evaluation criteria by which fatigue potential is evaluated in AAMAS based on the mixture properties of indirect tensile strain at failure and diametral resilient modulus. If the total resilient modulus and indirect tensile strains at failure for a particular mixture plot above the standard mixture (FHWA fatigue curve is recommended), it is assumed that the mixture has better fatigue resistance than the standard mixture. From Figure 6, it appears that all of the mixtures, except one, have about the same fatigue potential and are inferior to the standard mix in terms of fatigue resistance potential as characterized by the FHWA relationship. This means that most of the CRM mixtures tested in this study are more fatigue susceptible than the AAMAS standard mixture but may not be anymore susceptible than conventional dense-graded Type D mixtures currently used in Texas. The mixture produced with 18 percent fine CRM by the wet method has significantly better fatigue resistance than the others.

Check for Resistance to Rutting

One method of evaluating rutting potential was recommended for use by AAMAS as a "rough" guideline for mixture evaluation. This method is a graphical solution by which uniaxial creep data can be compared to criteria for predicting rutting potential. The uniaxial creep test was performed, as described earlier, on samples that were 4 in. (10.2 cm) high × 4 in. (10.2 cm) in diameter and were molded to air void contents less than 3 percent to simulate traffic densification. The samples were loaded under static conditions at 60 psi (414 kPa) for 1 hour with a 1-hour recovery period. The creep modulus data are shown in Figure 7 along with AAMAS criteria.

According to Figure 7, the creep moduli of all the mixtures tested were considered to have low to moderate rutting potential. The dense-graded mixture with fine (dry) CRM seems to be most rut resistant, while the gap-graded, 18 percent coarse CRM (wet) mixture appears to be least rut resistant.

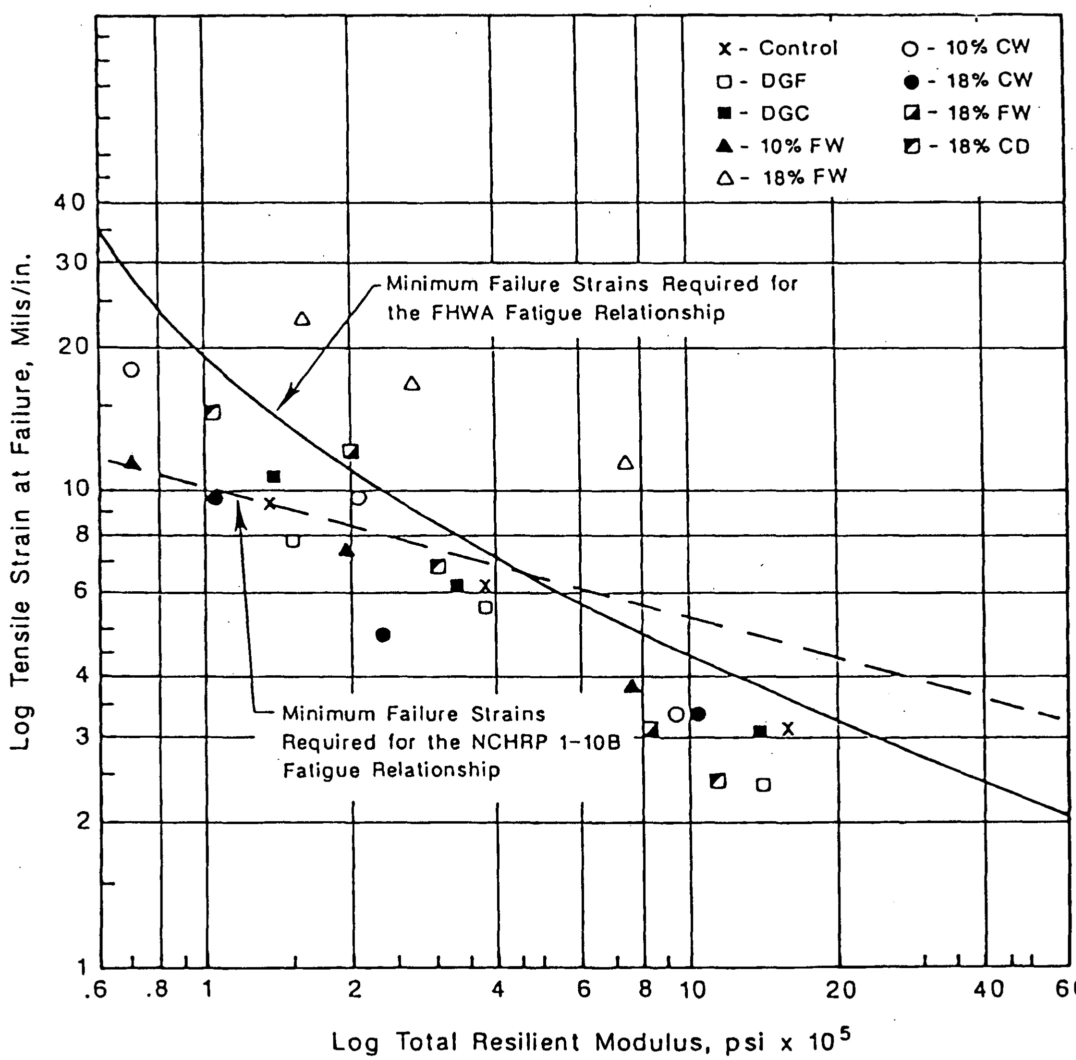

FIGURE 6 AAMAS chart showing resilient modulus versus tensile strain for control and CRM mixtures.

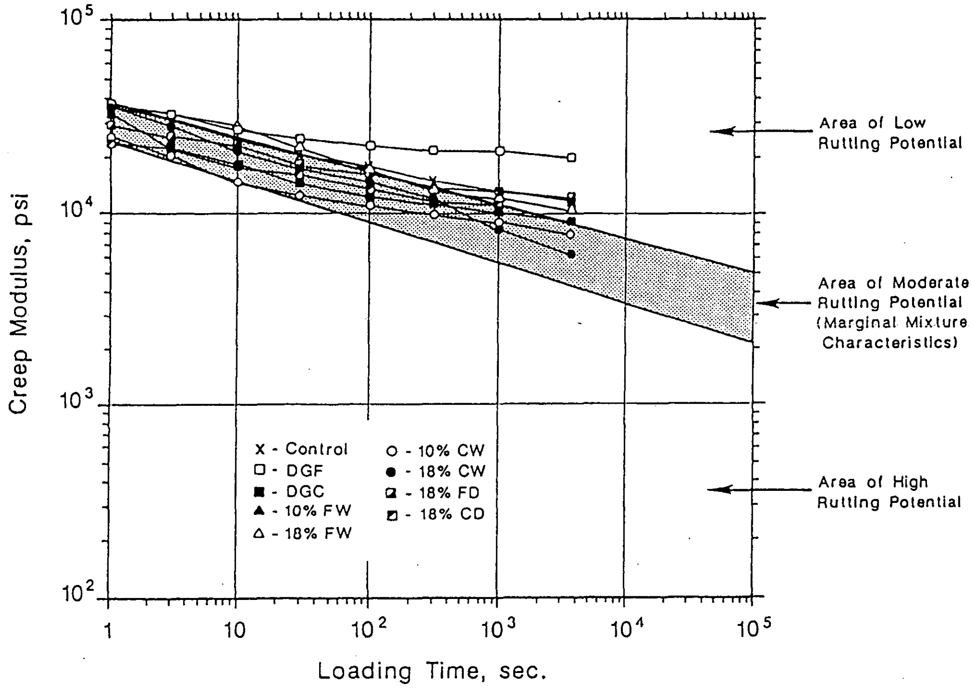

FIGURE 7 AAMAS chart for rutting potential for surface layers of asphalt concrete pavements.

Check for Resistance to Moisture Damage

The moisture damage evaluation (tensile strength and resilient moduli ratios) of AAMAS is simply used as a means of accepting or rejecting a mixture. Both values should exceed 0.80 for a dense-graded mixture. Tensile strength ratio, shown in Figure 8, is the tensile strength after moisture conditioning divided by the tensile strength of unconditioned specimens tested at 25°C (77°F). All of the mixtures exceeded the minimum requirement of 0.80 for tensile strength ratio.

Resilient modulus ratio is calculated as the modulus value after moisture conditioning divided by the modulus of unconditioned

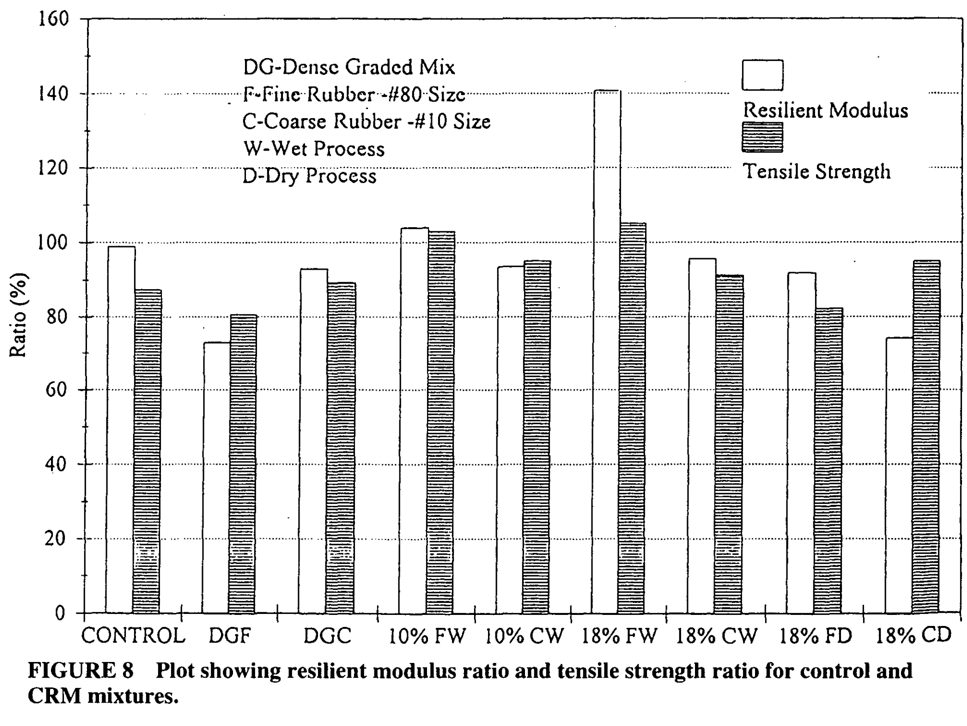

FIGURE 8 Plot showing resilient modulus ratio and tensile strength ratio for control and CRM mixtures.

specimens. This test is also performed at 25°C (77°F). All of the mixtures except two exhibited excellent resilient modulus ratios (see Figure 8). The dense-graded mixture produced with fine CRM and the gap-graded mixture produced with 18 percent coarse, dry CRM had resilient modulus ratios below the minimum recommended value of 0.80. These mixtures have one thing in common: both contain rubber that was added to the mixture as a dry process. However, the other mixtures that used a dry process (18 percent FD and DGC) had very good resilient modulus ratios. The mixture produced with 18 percent fine CRM by the wet process (18 percent FW) had resilient modulus values significantly larger after moisture conditioning. This may be attributed to this mixture's having a high degree of saturation (70 percent) and the development of pore water pressure during the testing procedure.

CONCLUSIONS

1. Results from this laboratory study indicate that acceptable performance may be obtained with CRM in dense-graded mixtures at lower concentrations of rubber (0.5 percent by weight of aggregate). The dense-graded laboratory mixtures evaluated in this study contained CRM, added dry, as part of the aggregate.
2. TxDOT's (volumetric) mixture design procedure for asphalt-rubber mixtures can be used to incorporate rubber of any size or process (wet or dry).
3. Mixtures designed with TxDOT's CRM mixture design procedure generally produced mixtures that are considered very rut resistant. This is particularly effective at higher concentrations of rubber (10 percent or more by weight of asphalt content).
4. CRM has the potential to significantly improve the fatigue and thermal cracking performance of asphalt concrete pavements, but only when the wet method is used and the binder is properly designed. A significant improvement in fatigue characteristics was observed with a particular mixture: 18 percent FW.
5. The wet process should produce asphalt mixtures (if binder is properly designed) that inhibit cracking and *may* inhibit rutting. The dry process, on the other hand, produces mixtures with reduced propensity for rutting but may have adverse effects on cracking. In the dry process, the rubber exists as discrete particles. Discrete particles in asphalt will normally intensify the propensity for cracking but may enhance rutting resistance.
6. The present laboratory testing suggests if acceptable performance can be obtained by CRM mixtures, AASHTO thickness design procedure may not be applicable to CRM mixtures.
7. Although state transportation departments must comply with the existing legislative requirements, crumb rubber, like any modifier, should be used only to address mixture deficiency or expected deficiency in a given situation.

ACKNOWLEDGMENTS

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Use of Ground Tire Rubber in Asphalt Concrete Pavements—A Design and Performance Evaluation

GLEN A. MALPASS AND N. PAUL KHOSLA

North Carolina State University, with support from the North Carolina Department of Transportation, has explored the design and performance of two types of rubberized pavements: ground rubber mixed with an asphalt binder at elevated temperatures (wet process) and rubber mixed with a gap-graded aggregate before the addition of asphalt cement (dry process). The wet process mixtures contained 11 percent ground tire rubber by weight of the binder. The dry process mixtures incorporated 2 percent ground tire rubber by weight of the aggregate. The Marshall and the Corps of Engineers gyratory testing machine (GTM) procedures were used to design conventional dry process and wet process surface course mixtures. These mixtures were tested with respect to resilient modulus, creep, and fatigue to obtain input parameters for a computerized performance prediction model. The addition of rubber was found to increase asphalt demand by 0.5 percent for the dry process and 1.5 percent for the wet process. The performance model estimated that the new rubberized pavement systems would have shorter service lives compared to a new conventional pavement system. When the wet process mixture was used to overlay a distressed conventional system, it performed as well as an equal thickness of a conventional overlay.

Federal mandates on the use of ground tire rubber in asphalt pavements have forced many state highway agencies to explore the feasibility of several types of asphalt-rubber combinations. North Carolina State University, with funding from the North Carolina Department of Transportation (NCDOT), has explored the design and performance of two types of asphalt rubber combinations: ground rubber introduced in the binder at elevated temperatures (wet process), and rubber granules mixed with a gap-graded aggregate before the addition of the binder (dry process). The optimum asphalt contents for the rubberized mixtures were determined using a modified Marshall procedure as well as the Corps of Engineers gyratory testing machine. After the optimum asphalt contents were determined, the rubberized mixtures were characterized in terms of resilient modulus, diametral fatigue, and incremental static (creep) testing. Owing to cost and time constraints, a computer performance prediction model was used instead of a full-scale field test to compare performance of the conventional and rubberized pavements. Performance of six different conventional and rubberized pavement systems was investigated, including new and rehabilitated pavements.

The use of ground tire rubber in asphalt pavements has shown promise in previous studies (*1–5*). The addition of rubber has been found to reduce temperature susceptibility and increase the ductility, resiliency, flexibility, and fatigue life of paving mixtures. The introduction of rubber to conventional mixtures has also been shown to increase asphalt demand in most cases. Field tests have shown that rubberized mixtures can exhibit less fatigue cracking and rutting than equal thicknesses of conventional pavements.

The specific objectives of this research were

1. To design the rubberized mixtures using the Marshall and GTM design procedures.
2. To characterize the rubberized mixtures in terms of resilient modulus, creep, and fatigue.
3. To compare the performance of the rubberized and conventional mixtures using a computerized performance prediction model.

MATERIALS

The aggregate used for this study was a 100 percent manufactured granite supplied by the Martin Marietta Company. Before the aggregate was used in the fabrication of test samples, it was dried overnight at 148.9°C, sieved into size fractions, and recombined using a standard or modified North Carolina surface course gradation. The gradation used for the wet process surface mixtures was a standard heavy duty surface (HDS) gradation, shown in Figure 1. The dry process gradation was gapped in the No. 8 to No. 50 sieve size range. Gapping the aggregate for the dry process mixtures was considered necessary in order to maintain sufficient air voids.

The rubber was supplied by BAS Engineering of Irvine, California. The rubber appeared to be angular with a smooth texture and was free from steel belts, cords, or other contaminants. It was easily sieved into size fractions using the same procedure as for the aggregate, and recombined using the gradations shown in Table 1. The rubber gradations in the wet and dry processes were suggested by TAK Engineering, a consulting firm with experience in asphalt-rubber applications. The amount of rubber used in the dry process mixture was 2 percent by weight of the aggregate.

The asphalt cement was graded as AC-20, and verified using the absolute and kinematic viscosity tests (ASTM 2170, 2171). The wet process binder was produced in the laboratory in 1-gal batches and contained 11 percent rubber by the total weight of the binder. The gradation of the rubber used in this process is shown in Table 1. An extender oil and reaction catalyst was also added at 7 percent and at 2 percent by total weight of the total binder, respectively. The procedure used for combining the ingredients of the wet process binder is as follows:

1. The base asphalt was heated to 176.7°C.
2. The extender oil was added to the asphalt and mixed for 2 min at medium shear (800 rpm).

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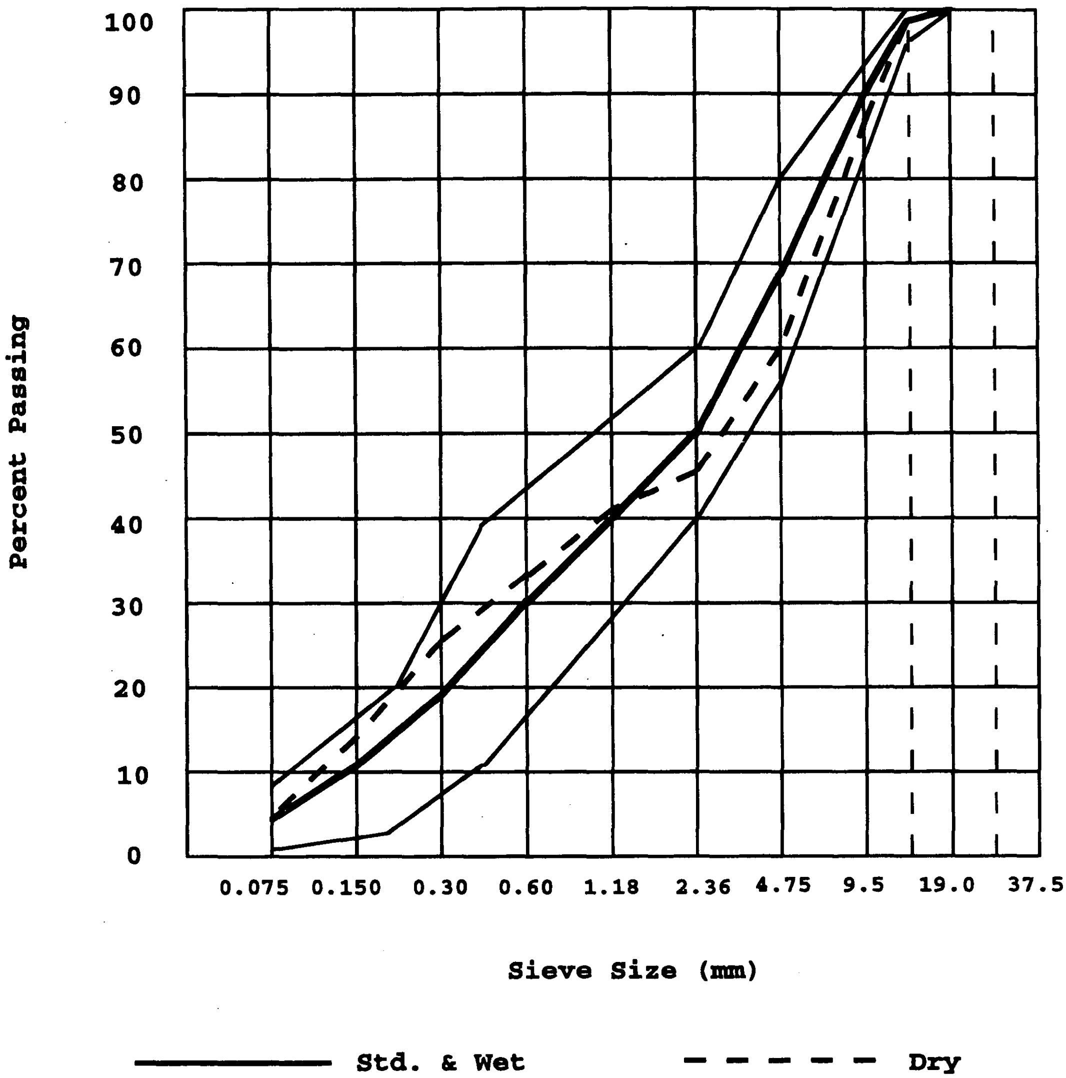

FIGURE 1 HDS specifications and gradations used.

3. The catalyst was added to the asphalt and mixed at medium shear for 10 min.
4. The rubber was added to the asphalt and mixed for 90 min at medium shear.

The wet process mixture had a texture that made it easily distinguishable from the conventional binder. The viscosity of the wet process binder seemed to be higher than the AC-20, but no formal viscosity test data are available. The coarse texture of the wet process binder made the validity of a conventional viscosity test doubtful. The rubber particles did not dissolve completely in the asphalt cement, causing blockages in the viscosity tubes and making the test results highly variable.

TABLE 1 Rubber Gradations

Sieve Size	Wet Process % Passing	Dry Process % Passing
#4	100	100
#8	100	90
#16	100	50
#30	75	40
#50	50	20
#80	10	-

SPECIMEN FABRICATION

Owing to the nature of the rubber materials, several modifications in the standard specimen fabrication procedures were made. The conventional samples were mixed at 148.9°C and compacted at a minimum of 140.5°C. The binder in the wet process samples was found to coat the aggregate better if mixed at 176.7°C. The densities of the wet process samples were also found to be adequate if compacted at 140.5°C. Mixing of the dry process specimens consisted of several steps to avoid excessive smoking and burning of the rubber particles. The rubber, which was at room temperature, was dry mixed with aggregate at 176.7°C for 30 sec. The aggregate and rubber were then placed in a 176.7°C oven for 5 min. The binder, which was heated to 148.9°C, was then added to the aggregate and rubber and mixed for 150 sec. After being cured for 1 hr at 148.9°C, the mixture was compacted at a minimum temperature of 140.5°C. The samples were either compacted with the Marshall hammer (75 blows) or gyratory testing machine (826.8 kPa, 1° angle, 60 revolutions), depending on the design method. Marshall size (6.35 cm × 10.16 cm) cylindrical samples were used for both design methods.

MIXTURE DESIGN

Marshall Method

The Marshall samples were tested for Marshall stability and flow following ASTM D1559, and the air voids were determined using ASTM D2726 and D2041. The procedure for the selection of the optimum asphalt content using the Marshall method was modified from that most widely used (*6*). If the requirements for stability, flow, and voids in mineral aggregate (VMA) were met, the asphalt content that yielded 5 percent air voids was selected as the optimum. This method for selecting the optimum asphalt content, currently used by NCDOT, produces samples with void contents similar to those in new pavements. The selection of a high initial air void content is thought to reduce wheel track bleeding due to traffic. The optimum asphalt contents, determined using this modified Marshall procedure, are shown in Table 2. The properties measured at the optimum asphalt content for the Marshall mixtures are shown in Table 3.

Gyratory Testing Machine (GTM)

The determination of the optimum asphalt content using the gyratory testing machine (GTM) was performed in accordance with ASTM D3387. The optimum was generally selected by averaging the asphalt contents that yielded the maximum unit weight, gyratory shear (Sg) or gyratory shear factor (GSF), and gyratory compactibility index (GCI). The optimum asphalt content was limited by the gyratory stability index (GSI), which must be less than or equal to 1.00. Mixtures with a GSI larger than 1.00 are considered unstable due to the binder overfilling the voids, resulting in loss of aggregate interlock and strength. The optimum asphalt contents for all mixtures, as determined by the GTM, are shown in Table 2. The properties at optimum asphalt content are shown in Table 4.

From Table 2 it can be seen that the asphalt demand for the dry process mixtures increased 0.5 to 0.8 percent over that required by the conventional mixture. The increase in asphalt content was larger for the wet process mixture. In this case the asphalt demand increased 1.5 percent. This large increase in binder demand for the wet process may be due to the fact that the wet process binder contains only 80 percent asphalt cement by weight. The actual amount of asphalt cement in the wet process mixtures is 5.25 percent, which is similar to the asphalt content of the conventional mixtures. The recommended optimum asphalt contents were based on the GTM design because the GTM's ability to measure the compaction stability of a mixture was considered important in the design of rubberized mixtures. The GTM was also used to fabricate the specimens for the mixture characterizations because this method of compaction was also thought to better represent field compaction than the Marshall method.

From Table 3, it may be noted that the rubberized mixtures generally had lower Marshall stability, tensile strength, and unit weight values than the conventional mixtures while the VMA and flow increased. The rather large decrease in unit weight values for the rubberized mixtures, compared to the conventional mixture, is most probably caused by the increases in asphalt content of the rubberized mixtures. In the dry process, the unit weight decrease could be due to the specific gravity of the rubber being lower than the aggregate it displaces. The increase in asphalt demand, and subsequent reduction in aggregate interlock, may explain the reductions in Marshall stability, tensile strength, and resilient modulus values found in the rubberized samples. The presence of rubber in the aggregate matrix may produce "weak links," which could also reduce the strength of the rubber mixtures. From Table 4, it may be noted that the dry process mixtures had larger Sg and GSF values compared to the conventional and wet process mixtures. In the GTM design, the unit weights of the rubber mixtures were also lower than those of the conventional mixture. The unit weights here could be thought

TABLE 2 Optimum Asphalt Contents

Design Method	% Asphalt Total Weight		
	Standard	Dry Process	Wet Process
GTM	5.0	5.5	6.5
Marshall	5.1	5.8	6.6
Recommended	**5.0**	**5.5**	**6.5**

TABLE 3 Marshall Mixture Design Properties at Optimum Asphalt Content

Property	Standard	Dry Process	Wet Process
Unit Weight (kg/m^3)	2298.6	2250.6	2221.8
Stability (kN)	15.5	13.7	12.3
Flow (0.25 mm)	10.8	12.5	11.2
Air (%)	5.0	5.0	5.0
Tensile Strength (MPa)	1.54	1.27	1.12

TABLE 4 Gyratory Mixture Design Properties at Optimum Asphalt Content

Property	Standard	Dry Process	Wet Process
Unit Weight (kg/m^3)	2378.7	2337.1	2338.7
GCI	0.981	0.983	0.987
Shear (kPa)	414.8	822.7	312.1
GSF	1.6	2.82	1.18
GSI	1.02	1.00	1.00
Tensile Strength (MPa)	1.70	1.55	1.10

of as representing those found at the end of a pavement's service, explaining the increase in unit weights here compared to those in the Marshall design.

Mixture Characterization

The mixture characterization was performed to obtain input for the VESYS—3AM performance prediction model and all tests were performed in accordance with the VESYS manual (7). The resilient modulus tests were performed in the indirect tensile mode on all mixtures at 4.4°, 21.1° and 37.8°C using a Retsina Mark IV pneumatic resilient modulus device. The deformations were measured using linear variable displacement transducers (LVDTs) and a strip chart recorder. The procedure for the test was similar to ASTM D4123 with a loading time of 0.1 sec and rest period of 2.9 sec. The sample sizes used for the tests were 10.16 cm in diameter and 6.35 cm in height. The resilient modulus was calculated using the following equation:

$$M_r = \frac{P(0.2734 + \mu)}{td}$$

where

M_r = resilient modulus (MPa),
P = applied load (N),
μ = Poisson's ratio (assumed here as 0.35),
t = sample thickness (mm), and
d = recoverable deformation (mm).

The results of the resilient modulus testing are shown in Figure 2. It can be seen from this figure that the conventional mixture had the highest resilient modulus of all the mixtures at all test temperatures. The difference between the resilient modulus for the conventional and rubberized mixtures decreases with increasing temperature. This may mean that the addition of rubber affects the temperature susceptibility of the mixtures.

The fatigue tests were performed at 21.1°C on the surface mixtures. These tests were performed even though the VESYS prediction model only requires the fatigue parameters for the lowest asphalt layer in a pavement system. The sample size used for the fatigue tests was the same as used in the resilient modulus tests. The fatigue tests were performed on an MTS model 810 in the indirect tensile mode and with computerized data acquisition. The total strain at the 200th cycle, called the initial total strain, and the cycles to failure were measured for all samples. Different stress levels were used to produce failures of the samples ranging from 1,000 to 100 000 loading cycles. Failure of the samples was defined as a total deformation exceeding 3.81 mm. The initial strain and cycles to failure were plotted for the mixtures, as shown in Figure 3. The following fatigue model was then developed for all the mixtures:

$$N_f = K1\left(\frac{1}{\varepsilon}\right)^{K2}$$

where

N_f = cycles to failure,
ε = initial total strain,
$K2$ = inverse of the absolute value of the slope of the regression line,
$K1$ = $100\ (I)^{K2}$, and
I = initial total strain causing failure at 100 cycles.

The parameters $K1$ and $K2$, shown in Table 5, were used only for fatigue model comparison because the VESYS model does not require the fatigue parameters of the surface course. From Figure 3 it can be seen that rubberized mixtures performed better than the conventional HDS mixture. The models for the rubber samples are similar, with the dry process performing slightly better than the wet process.

The incremental static loading (creep) tests were also performed at 4.4°, 21.1° and 37.8°C in accordance with the VESYS manual (7). The test consisted of axially loading a cylindrical sample for 0.1, 1.0, 10, 100, and 1,000 sec. The permanent deformation after each incremental loading interval was summed and denoted as the accumulated permanent strain for a given loading time. The accumulated permanent strains and loading time were plotted as shown in Figures 4, 5, and 6. A cyclic load of 0.1 sec duration and 2.9 sec

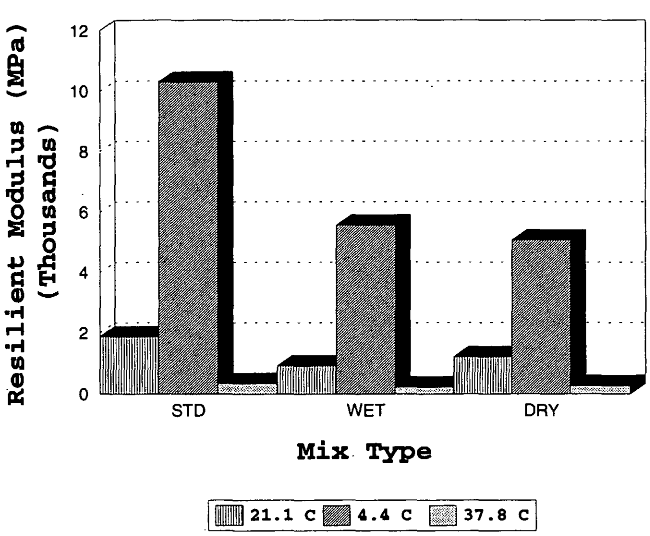

FIGURE 2 Resilient modulus test results.

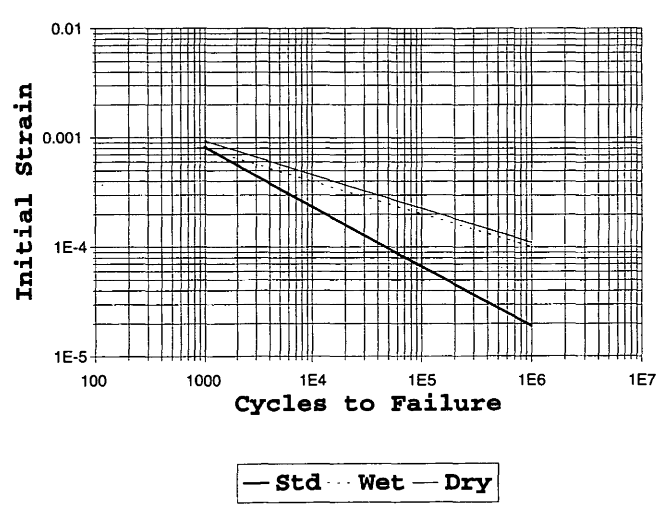

FIGURE 3 Fatigue test results.

TABLE 5 Fatigue Parameters for Surface Course Mixtures

Parameter	Standard	Dry Process	Wet Process
K1	2.15×10^{-3}	1.44×10^{-7}	3.63×10^{-8}
K2	1.84	3.25	3.36

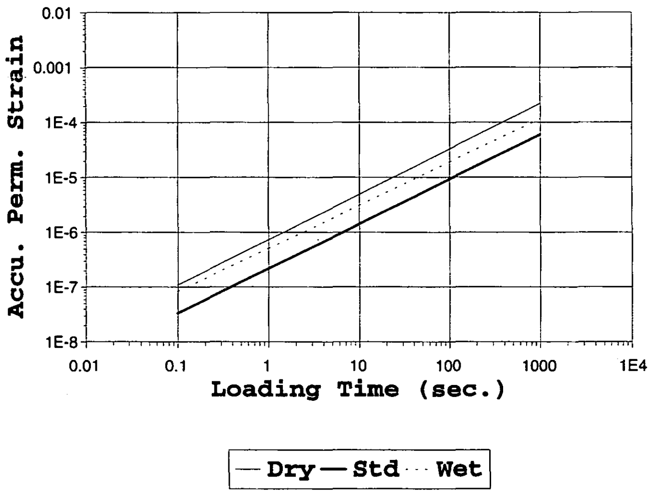

FIGURE 4 Creep test results—4.4°C

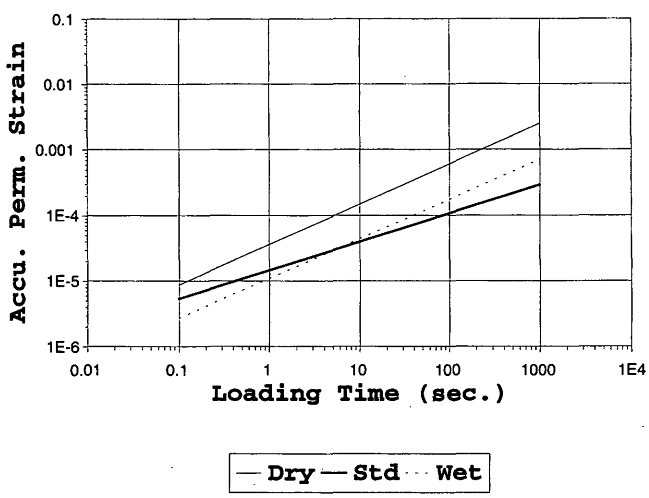

FIGURE 5 Creep test results—21.1°C.

rest was also applied after the last incremental loading of 1,000 sec to obtain the recoverable strain. From these plots the creep parameters Alpha and GNU were calculated as follows:

$$\text{Alpha} = 1 - S$$

$$\text{GNU} = \frac{IS}{\varepsilon_r}$$

where

I = permanent strain corresponding to a creep load of 0.1 seconds,
S = slope of the regression line, and
ε_r = recoverable strain due to a cyclic load.

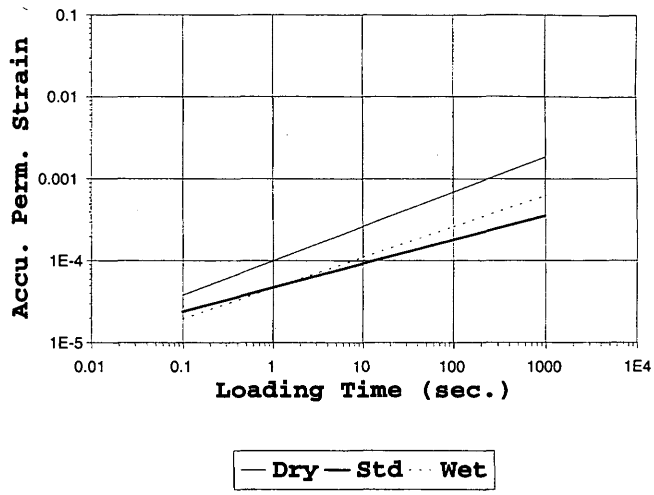

FIGURE 6 Creep test results—37.8°C.

A list of the Alpha and GNU parameters is shown in Table 6. From Figures 4 through 6, it can be seen that the dry process surface mixture had the highest permanent strain at all loading times and at all test temperatures. The conventional and wet process mixtures had similar creep lines. However, crossing of the creep lines suggests that the permanent strain for a given mixture depends on the temperature and the loading time.

Performance Predictions

The six pavement systems analyzed are shown in Figures 7 and 8. The new construction systems 1 through 3 contain an aggregate base course (ABC), a conventional binder layer, and a conventional wet process or dry process surface mixture. The mechanistic parameters for the conventional binder, aggregate base course, and clay subgrade were determined in a previous study (*8*) and are given in Table 7. The traffic was set at 300 ESALs per day with a 15.24 cm tire contact radius and 826.8 kPa tire pressure for all systems. The average seasonal pavement temperature was 4.4°C for winter, 37.8°C for summer and 21.1°C for spring and fall. The initial present serviceability index (PSI) was set at 4.6 and the terminal PSI at 2.5. As shown in Figure 8, the equivalent effective thicknesses of the distressed layers, used for the rehabilitated systems, were estimated to be ½ and ⅔ of the original thicknesses of the surface and binder course layers, respectively. An overlay thickness of 5.1 cm was used for the rehabilitated systems. The predicted service lives for all the new construction and rehabilitation systems are shown in Figure 9. The new conventional system had the longest service at 11.4 years. The new wet process system had a service life of

TABLE 6 Creep Parameters for Surface Course Mixtures

Parameter	Standard	Dry Process	Wet Process
Alpha (4.4°C)	0.0071	0.0031	0.0045
Alpha (21.1°C)	0.0358	0.0237	0.0193
Alpha (37.8°C)	0.0335	0.0282	0.0197
GNU (4.4°C)	0.1830	0.1730	0.2140
GNU (21.1°C)	0.5650	0.3960	0.3860
GNU (37.8°C)	0.6230	0.7050	0.5780

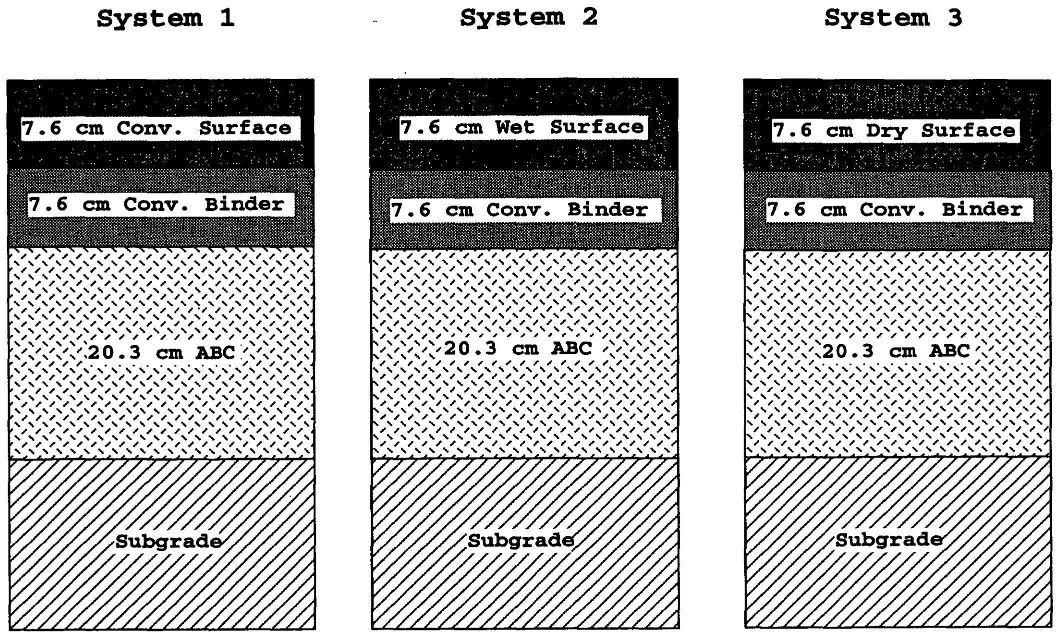

FIGURE 7 New construction systems 1, 2, and 3.

7.8 years, while the new dry process system had a service life of 5.0 years. The conventional and wet process overlays of conventional pavements had similar service lives at 8.2 and 8.0 years, respectively, while the dry process overlay had a service life of 6.2 years.

CONCLUSIONS

From the results of this investigation the following specific conclusions can be drawn:

- The addition of rubber to these asphalt concrete mixtures increased the asphalt demand. The gyratory testing machine predicted asphalt contents for the dry and wet process that were 0.5 and 1.5 percent higher, respectively, than those for the conventional mix. Since the GTM is able to predict the compaction stability of the mixtures, it was judged to be better suited for the design of rubber mixtures than the modified Marshall procedure.
- The fatigue models developed for the mixtures suggest that the rubber mixtures may be more resistant to fatigue cracking. However, the VESYS model predicted that the pavement systems containing rubberized asphalt surface layers failed in fatigue earlier than the conventional systems. Since a conventional binder course was used as the lowest asphalt layer for all of the systems in the VESYS analysis, these results indicate that the rubberized surface mixtures are less able to resist fatigue crack initiation and propagation than the conventional HDS mixture.
- The creep models developed for the mixtures suggest that the rutting performance of the wet and conventional mixtures is simi-

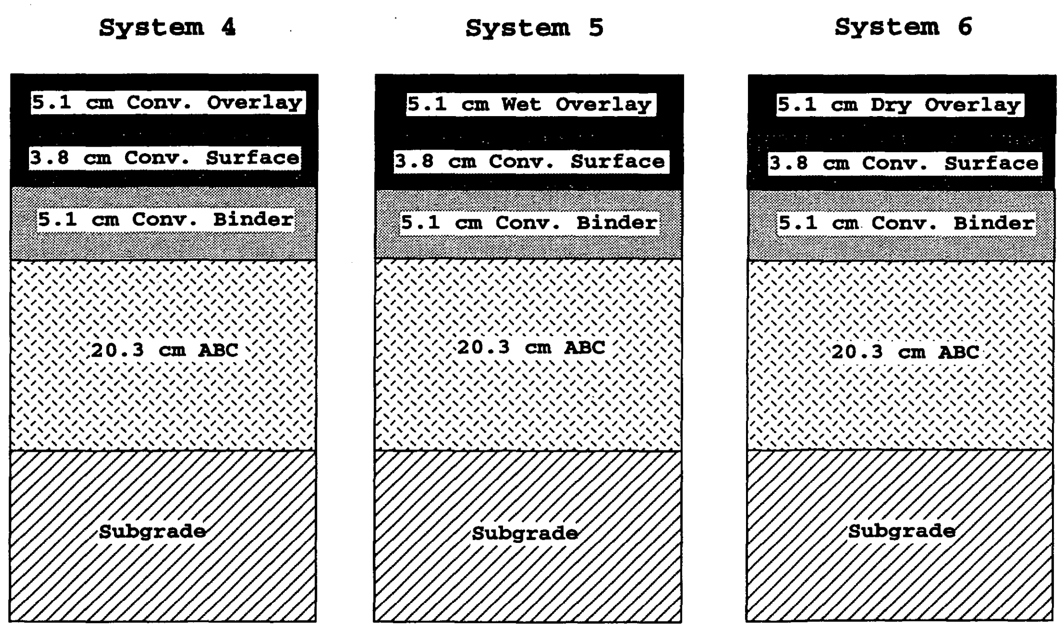

FIGURE 8 Rehabilitation systems 4, 5, and 6.

TABLE 7 Mechanistic Parameters for the Binder and Non-Asphaltic Layers

Parameter	Binder	Aggregate Base	Subgrade
Mr (MPa)			
4.4°C	12,953	186	27.6
21.1°C	2,225	179	20.7
37.8°C	407	200	62.0
Alpha			
4.4°C	0.410	0.810	0.850
21.1°C	0.620	0.840	0.850
37.8°C	0.690	0.870	0.720
GNU			
4.4°C	0.009	0.010	0.160
21.1°C	0.050	0.010	0.160
37.8°C	0.038	0.005	0.040
K1	1.95×10^{-7}		
K2	3.75		

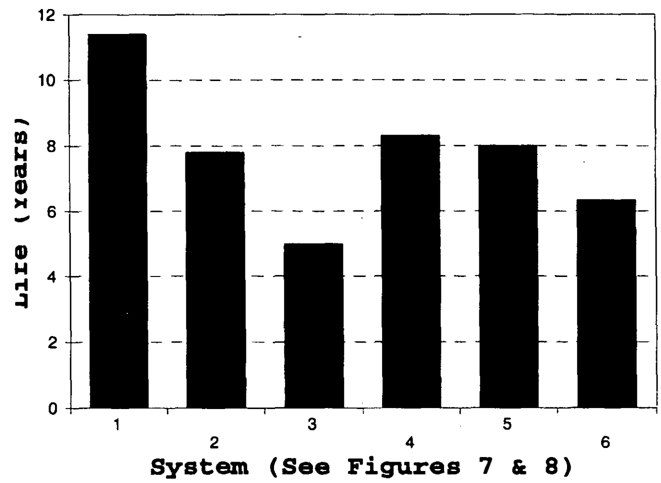

FIGURE 9 Predicted pavement lives.

lar. When the wet process is used as an overlay on a system containing an aggregate base, the performance is similar to a pavement system with an equal thickness of conventional overlay.

- The creep models also suggest that the wet process and conventional mixtures perform better in terms of creep than the dry process. The high gyratory shear strength that was obtained during the design of the dry process samples would appear to indicate these mixtures would be able to resist the shear deformation in late-stage rutting. However, these measurements were recorded with the vertical compaction pressure still acting on the specimen according to the ASTM procedure. After this vertical pressure is removed, all the dry process samples rebounded enough to exhibit cracking on the sides. For this reason, the GTM may not accurately predict the shear performance of dry process samples. It would appear that the amount of rebound after compaction, or the overall effect of the resiliency of the rubber, plays a major role in the parameters obtained in the GTM design of dry process mixtures. Because of the volume changes observed, it is believed that the samples tested to obtain the performance prediction parameters are at variance with those tested during their fabrication in the GTM.

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Mix Designs and Air Quality Emissions Tests of Crumb Rubber Modified Asphalt Concrete

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The mix design and air quality asphalt plant emissions testing of crumb rubber modified (CRM) bituminous concrete mixes are described. The wet process and the dry process were used to incorporate crumb rubber into standard bituminous concrete mixes, into an open graded friction course (OGFC) mix, into a recycled rubber reclaimed asphalt pavement (RAP) mix, and into a gap-graded propriety mix. The mix gradations are presented for six CRM bituminous concrete pavement-resurfacing and lane-widening projects. Asphalt plant emissions testing data are presented for the projects. Mix designs were developed successfully for three wet process CRM mixes using 10 percent, 15 percent and 18 percent crumb rubber by weight of an AC-20 or AC-10 asphalt cement. The emission tests indicated that total hydrocarbons, carbon monoxide, and particulate emissions for two of the mixes were within state of New Jersey emissions limits. The opacity and odor surveys indicated acceptable emissions. The third wet process project indicated some emissions values above the limit. The mix design was developed successfully for a project that used 20 percent rubber RAP in a fine aggregate surface course. The emissions tests indicate that total hydrocarbons and particulates were within emissions limits. One carbon monoxide reading was at the limit. Propriety gap-graded dry process CRM mixes design were constructed in the surface and base course of a resurfacing project. The propriety surface course mix raveled soon after construction and required replacement with the standard surface course. The air quality emissions tests indicated that the emissions data were higher than the standard mix and one total hydrocarbons reading was above the emissions limit. Three generic dry process mixes were constructed as a lane widening to develop CRM mix designs for surface course, base course, and base course with 10 percent crushed container glass. Air quality emissions values for the standard mix and CRM mix were above the emissions limits.

The objective of this study was to develop CRM bituminous concrete mixes, to compare those mixes to the standard mixes, and to compare the resulting air quality asphalt plant emissions during production of the mixes. With major emphasis on reducing the enormous numbers of scrap tires that were being landfilled and stockpiled, ground scrap tire rubber was incorporated into mixes for the potential benefit of both the long-term pavement properties and the environment. Furthermore, the Federal Intermodel Surface Transportation Efficiency Act (ISTEA) and the state recycling program has mandated the study and use of scrap tire rubber in highway applications.

BACKGROUND

The state of New Jersey discards annually approximately 10 million scrap tires. The nation discards annually some 285 million tires, of which 32 percent are retreaded, resold, or diverted to other uses such as tire-to-energy facilities. The remaining tires are stockpiled and landfilled, creating environmental concerns. The use of some scrap tires as ground tire rubber in bituminous concrete mixtures was proposed as a partial solution to mounting stockpiles of tires. This study of the ground tire rubber waste in bituminous concrete mixtures evaluates technical questions about the use of that material.

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RESEARCH METHODOLOGY

With the intent of allowing market forces to determine the most efficient and beneficial method of incorporating ground tire in bituminous concrete, the New Jersey Department of Transportation (NJDOT) initiated a study of both the wet and dry processes for the addition of crumb rubber modifier into bituminous concrete mixes for several major projects. Since CRM technologies were available to reasonably assure the success of experimental pavements, the mixes were constructed in large projects where sufficient materials could be produced to compare asphalt plant air quality emissions, mix designs, asphalt plant production, and construction procedures. To compare future long-term pavement performance of the CRM mixes, standard pavement courses were constructed adjacent to CRM mixes.

The NJDOT standard laboratory materials test procedures were conducted to determine the appropriate mix designs with the rubber. Using job mix formula specifications and the Marshall method, gradation tests were made to determine quantities of aggregates and asphalt cement, as well as the optimum quantity of rubber. Marshall mix design plugs were tested in accordance with the current specifications to evaluate performance parameters such as mix stability, flow, density, and voids. In one project, resilient modulus testing was performed on Marshall specimens to compare the CRM mixes with the standard mixes.

The field sections were surveyed for cracking, rutting, skid resistance, and roughness, and the initial field data are reported.

Asphalt plant air quality emissions protocols were developed to test both the standard and CRM mixes for carbon monoxide, total hydrocarbons, particulate emissions, odors, and visual emissions. These data were compared to the New Jersey Department of Environmental Protection (NJDEP) standards. Also, the air emissions data of the production of standard bituminous concrete mixes were compared to the emissions data of the CRM mixes under similar test conditions. The air quality test procedures used were

- Carbon monoxide: US EPA Reference Method 10, "Determination of Carbon Monoxide Emissions from Stationary Sources";

- Total hydrocarbon (THC) (as methane): NJ Air Test Method 3, Section 3.7, "Procedures for the Direct Measurement of Volatile Organic Substances Using a Flame Ionization Detector";
- Particulate emissions: NJ Air Test Method 1, "Sampling and Analytical Procedures for Determining Emissions of Particles from Manufacturing Processes and from Combustion of fuels"; and
- Opacity readings: NJ Air Test Method 2, "Procedures for the Visual Determination of the Opacity (%) and Shade or Appearance (Ringelmann Number) of Emissions from Sources."

CONSTRUCTION

Experimental Field Sections

From October 1991 to June 1994, experimental field sections of CRM bituminous concrete mixes were constructed as overlays and lane widening on six projects in urban and rural New Jersey. The projects were initiated to evaluate the use of crumb rubber in the NJDOT's standard surface course mix using the wet process, an open graded friction course (OGFC) mix using the wet process, gap-graded proprietary dry mix called Plus-Ride II using a dry process, a recycled CRM rubber-modified reclaimed asphalt pavement (RAP) surface course mix, and a generic dry process surface and base course mix. A summary of general information for the six demonstration projects is presented in Table 1. Summaries of the laboratory test properties and air quality emissions test results are presented in Table 2 and Table 3.

Wet Process CRM Mix (Rouse)

The first wet process CRM bituminous concrete surface course was constructed on a resurfacing project on Route I-95, Ewing Township, in October 1991. A standard bituminous concrete surface course was constructed on a contiguous section of Route I-295, Hopewell Township, as a control section for comparison with the CRM mix. The asphalt-rubber cement was blended at the plant by Rouse process.

The asphalt plant was a 2.7-tonnes McCarter (3-ton) batch type plant. The asphalt cement (AC-20) was blended with 10 percent of a No. 40 mesh recycled tire rubber in the on-site portable blending unit, heated to 177°C (350°F) and mixed with the aggregates and baghouse fines in the pugmill for 35 sec. The CRM mix was discharged from the plant between 143°C to 160°C (290°F to 320°F). As a reference, the standard mix was typically discharged at 138°C to 154°C (280°F to 310°F). The tire rubber was supplied in plastic bags. Dampness caused two pallets of these bags to solidify causing production delays.

The construction of the CRM surface course was preceded by milling 51 mm (2 in) of the existing bituminous concrete pavement to eliminate rutting. A bituminous stabilized base course replaced

TABLE 1 Summary of Crumb Rubber Modified Asphalt Concrete Experimental Field Projects

	Project #1	Project #2	Project #3	Project #4	Project #5	Project #6
Route	I-95&I-295	Ferry St.	I-195	US 130	I-95	I-287
Location	Ewing Twp.	City of Newark	Allentown	Logan Twp.	Ewing Twp.	
Bedminister						
	Hopewell Twp.		Westbound	Southbound	Hopewell Twp.	Northbound
	Northbound				Southbound	
Project	51 mm,	38 mm,	19 mm,	51 mm, surface	51 mm, surface	51 mm surface
	overlay	overlay	overlay	51 mm, base	76 mm, base	51 mm base
				overlay	overlay	lane widening
Mix Type	Surface Course	Surface Course	Open Graded	Plus-Ride II;	Surface Course	Surface Course
	Crumb Rubber	CRM-Recycled	Friction Course	Surface Course	Base Course	Base Course
	Modified	20% RAP	(OGFC)	Base Course		Base w/Glass
Mix Process	Wet	Recycled Dry	Wet	Propriety Dry	Wet	Generic Dry
AADT	43,700	N.A.	20,500	4,000	43,700	
54,700						
	6 lane, divided	2 lane,	4 lane, divided	4 lane, divided	6 lane, divided	6 lane divided
	Suburban	Urban	Rural	Rural	Suburban	Rural
Date	October 1991	July 1992	August 1992	October 1993	June 1994	May 1994
Total Material						
Surface Course	2086 t	1088 t	435 t	3592 t	2441 t	1886 t
Base Course				4280 t	2897 t	2231 t
Base/glass						2022 t
Length	1159 m	244 m	549 m	2653 m	1067 m	2670 m
Air Quality Testing	yes	yes	yes	yes	yes	yes

TABLE 2 Summary of Laboratory Test Properties

Laboratory Tests	Project #1 Wet Process		Project #2 Recycled Rubber RAP		Project #3 Wet Process		Project #4 Propriety Gap Graded Dry Process			
	CRM Surface	Standard Surface	CRM-RAP Surface	Standard Surface	CRM OGFC Surface	Standard Surface	Standard Surface	Standard Base	CRM Surface	CRM Base
Marshall										
Stability, lbs.	2252	2383	2080	2590	n.a.	n.a.	1675	1550	1310	950
Flow 0.01" (0.02cm)	14	14	17	14	n.a.	n.a.	10	11.7	38.5	41.5
Air voids, %	2.7	2.7	3.6	3.3	18	n.a.	3.4	4.0	2.8	2.3

Laboratory Tests	Project #5 Wet Process				Project #6 Generic Dry Process					New Jersey Requirements Standard Mixes	
	CRM Surface	Standard Surface	CRM Base	Standard Base	CRM Surface	Standard Surface	Standard Base	CRM/ Glass Base	Standard Base	Mix 19mm Surface	Mix 39.7mm Base
Marshall											
Stability, lbs.	2650	2100	2409	2700	2670	2375	2510	2277	2280	1200 min.	1200 min.
Flow 0.01" (0.02cm)	12	12	13	12	12	16	12	17	14	6 - 16	6 - 18
Air voids, %	3.5	3.5	3.5	3.5	3.7	3.4	3.9	3.5	3.2	2 - 5	2 - 5

the milling. The CRM mix was placed as a surface course on the inside shoulder and mainline pavement with a standard paver. A breakdown roller, an intermediate roller, and a finish roller were used on the CRM surface course. The laydown temperatures of the mix were above 143°C (290°F). As a reference, the lay down temperature of most standard NJDOT mixes was about 138°C (280°F).

Construction Data

Using the Marshall method, the standard mix and the CRM mix were developed using NJDOT standard gradations. The composition samples for the mixes are shown in Table 4. The standard surface course and the CRM mixes have similar gradations with the exception of a

TABLE 3 Summary of Emissions Tests Results

Test Number	Emissions Standards	Project #1 Wet Process Standard Mix	Project #1 Wet Process CRM Mix	Project #2 Recycled Rubber RAP Stand.Mix 1	Stand.Mix 2	CRM RAP 1	CRM RAP 2	Project #3 Wet Process CRM-OGFC 1	2	3	Project #4 Propriety Dry Process CRM Mix 1	2	3	Virgin Mix 1	2	3
Concentration Data																
Total Hydrocarbons (as methane)																
ppmV-dry @ 7% O_2	250	n.a.	n.a.	32	34	26	32	129	150	147	245	289	244	134	125	133
Carbon Monoxide																
ppmV-dry @ 7% O_2	500	n.a.	n.a.	284	328	396	500.6	222	217	265	134	122	160	96	111	87
Particulates																
grains/scf	0.02	n.a.	n.a.	0.006	0.006	0.006	0.006	0.004	0.003	0.004	0.004	0.002	0.002	0.006	0.001	0.003
Opacity (%)	10	10.4	9.0	n.a	n.a.	n.a.	n.a.	0	0	0	0	0	0	0	0	0
Odor Survey	n.a.	0	0	n.a.	n.a.	n.a.	n.a.	0	0	0	2	2	3	2	2	3
Intensity: 0-none, 5-very strong																

Test Number	Emissions Standards	Project #5 Wet Process CRM Mix 1	2	3	Standard Mix 1	2	3	Project #6 Generic Dry Process CRM Mix 1	2	3	Standard Mix 1	2	3
Concentration Data													
Total Hydrocarbons (as methane)													
ppmV-dry @ 7% O_2	250	417	362	307	78	70	66	1271	1301	1067	1042	324	517
Carbon Monoxide													
ppmV-dry @ 7% O_2	500	148	104	115	107	106	104	515	503	445	338	248	270
Particulates													
grains/scf	0.02	0.37	0.50	0.42	0.09	0.09	0.11	0.05	0.05	0.05	0.07	0.06	0.06
Opacity (%)	10	24	n.a	n.a.	19.2	n.a.	n.a.	3.4	4.3	4.7	4.4	4.2	4.9
Ordor Survey	n.a.	<1	<1	<1	<1	1.2	1.1	1.0	1.1	1.4	1.3	1.3	1.4
Intensity: 0-none, 5-very strong													

TABLE 4 Summary of Mix Gradations, Wet Process CRM Mix, Project No. 1, Route I-95

Sieve Size	Percent Passing-Job Mix Formula CRM Mix Surface Course	Percent Passing-Job Mix Formula Standard Mix Surface Course	NJDOT Specification Mix size (19mm)3/4inch Surface Course
25.4 mm (1 inch)	100	100	100
19 mm (3/4 inch)	99	99	98-100
12.7mm (1/2 inch)	95	95	88-98
9.5 mm (3/8 inch)	87	87	65-88
No. 4	61	62	35-65
No. 8	45.5	45.5	25-50
No. 16	33	33	10-40
No. 30	23	23	12-30
No. 50	17	17	10-25
No. 200	5.9	6.1	3-10
A.C.	6.5	5.6	4.5-9.5

Sieve Size	Ground Tire Rubber Gradation (% passing)
40	100
60	98-100
80	90-100
100	70-90
200	35-60

higher percentage of asphalt cement for the CRM mix. Marshall stability, flow, and air voids are similar for both mixes.

The asphalt-rubber cement tests are shown in Table 5. The viscosity of the asphalt-rubber cement is five to six times greater than the virgin AC-20.

After paving, thin lift nuclear gauge density measurements were made to survey the as-built pavement density. The measured percent air voids of the CRM mix were 4.4 percent for the inside lane, 5.4 percent for the center lane, and 5.6 percent for the outside lane. The NJDOT specifications require percent air voids to fall between 2 and 8 percent. A vibratory roller was required to compact the CRM mix.

Similar density measurements were made on the standard bituminous concrete surface course mix. The measured percent air voids of the standard surface course were 7.7 percent for the inside lane and 4.5 percent for the center lane. The percent air voids of the standard surface course were higher than the CRM section; the reason for this was unclear.

ARAN Ride Quality Index values were made in June 1993. The CRM mix pavement had an average value of 4.10, and the standard bituminous surface had an average value of 4.15. Both values indicate a good pavement. Average rut depths on both sections were 4.7 mm (3/16 in.). Some fine cracking was noted in the inside lane of the CRM section shortly after construction. To date, cracking has not significantly increased.

The resilient modulus data of Marshall plugs are shown in Figure 1. The modulus data of the CRM mix are somewhat higher than the standard surface course mix.

Asphalt Plant Air Emissions Tests

The visible and fugitive emissions and odor survey were conducted at the plant during the CRM and standard mix production. Since the plant is located in Penns Park, Pennsylvania, the air quality emission protocol was developed with the Pennsylvania State criteria. The plant is set up to produce material based on those criteria. An environmental consultant conducted the source emissions test for visible emissions compliance and the odor survey at various locations of the plant facility.

A total of 36 odor survey tests were conducted during the 2 days of testing. The odor survey consisted of monitoring the odors emitting from the plant for intervals of 5 to 10 min at several property

TABLE 5 Asphalt Cement Test Results, Wet Process CRM Mix, Project No. 1, Route I-95

Test	Virgin AC-20 Sample #1	Virgin AC-20 Sample #2	CRM AC-20 Sample #1	CRM AC-20 Sample #2
Absolute Viscosity @ 140 F, Poises	1967	1982	10,032	10,388
Kinematic Viscosity @ 275 F, Centistrokes	412	420	2806	3007
Penetration @77 F 100g, 5 sec.	82	83	50	50
Specific Gravity	1.036	1.036	1.045	1.045

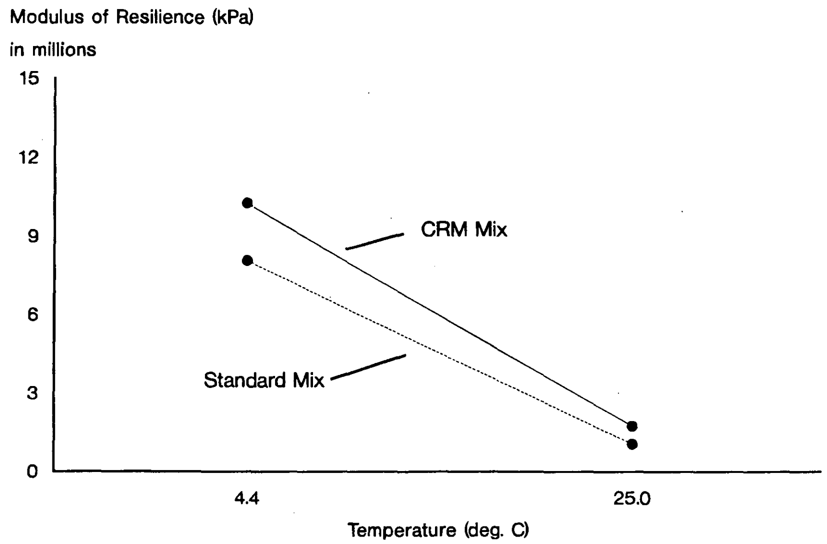

FIGURE 1 Modulus of Resilience Data, Project No. 1.

perimeter locations for each hour of production. The tests did not detect any odors emanating from the production of either mixes.

During the production of the CRM mix, six test runs of 30 min each were conducted both days with 6-min opacity readings every 15 sec at the outlet stack on the baghouse, with spot checks for fugitive emissions on the processing equipment downstream of the blender. During the production of the standard mix, 12 similar tests were conducted for 1 day's production.

The highest opacity reading of the CRM mix tests was 17.1 percent, which was taken during the start-up of the second day of the production. The high reading was attributed to the initial start-up problems of the plant and not the production of the CRM mix. The average opacity of the CRM mix tests was 10.4 percent and the standard mix was 9.0 percent. In Pennsylvania, the average opacity for the highest period should not exceed 20 percent. Although some CRM mix readings exceeded the New Jersey criteria, the test readings meet the Pennsylvania requirement where the plant was designed to operate. No statistical difference is noted between the opacity readings of the CRM and standard mix.

Recycled Rubber RAP (Dry Process)

In 1991, the City of Newark constructed a Plus-Ride dry process CRM pavement which raveled shortly after construction. The pavement was milled, and the rubber reclaimed asphalt pavement (RAP) was stockpiled for future use. The stockpile contained 226 tonnes (250 tons) of 3-percent rubber RAP.

The City of Newark proposed a Local Aid resurfacing project to use the stockpiled RAP. The proposed surface course mix used 20 percent rubber RAP in NJDOT's fine aggregate surface course mix. To complete the mix design, trial mixtures were prepared by varying the asphalt content for the virgin materials. This procedure permitted the asphalt cement (about 7 percent AC) in the rubber RAP to remain with the RAP in the mix. The mix composition is shown in Table 6. The surface course mix with 20 percent rubber RAP contained 6.4 percent asphalt cement and 0.46 percent crumb rubber (100 percent passing the No. 6-mm sieve size) from the RAP. In contrast, the surface course mix with RAP contained 5.55 percent asphalt cement. The Marshall tests for both mixes showed no sig-

TABLE 6 Summary of Mix Gradations Recycled Rubber RAP Project No. 2, City of Newark, Composition Analysis (Percent Passing)

Sieve Size	Surface Course Mix-RAP Composition Sample Fine Agg. Surface	Rubber RAP Mix Composition Sample Fine Agg. Surface	NJDOT Specification Mix size 9.5mm (3/8 inch) Fine Agg. Surface
12.7 mm	100	100	100
9.5 mm	99	98	80-100
No. 4	64	68	55-75
No.8	44.5	45.5	30-60
No. 16	32	32	20-45
No. 30	25	25	15-35
No. 50	17.0	17.0	10-30
No. 200	6.4	6.5	4-10
Asphalt Content	5.55	6.4	5-10
Rubber Content	--	0.46	--

nificant differences. However, the flow values of the rubber RAP samples were generally higher than the RAP mix samples.

The recycled rubber RAP mix was processed through a Bituma counter flow drum plant in Wharton, New Jersey, in July 1992. The mix was discharged from the plant and paved at normal temperatures. The mix was paved in a 38-mm resurfacing project on a city street in Newark. The nuclear density voids were 4.5 percent for the rubber RAP mix.

The air emissions tests were conducted at the asphalt plant stack outlet by an environmental consultant. The emissions tests included two 1-hour gaseous emissions tests for carbon monoxide, total hydrocarbons, and particulates during the asphalt plant production of the standard surface course material and the surface course with rubber RAP. The total hydrocarbons and particulates for both mixes were within emissions limits. Both carbon monoxide emissions test results for the rubber RAP mix were above the results for the standard mix without rubber. One carbon monoxide value of the rubber RAP mix test was at the emissions limit.

Wet Process CRM Mix Open Graded Friction Course

The wet process CRM mix open graded friction course was constructed on a resurfacing project on Route I-195, Allentown, in July 1992 to evaluate the use of CRM to stiffen the asphalt cement and reduce runoff during transportation. The Rouse process was used to blend 15 percent (No. 40 mesh size) crumb rubber by weight of the asphalt cement.

For most bituminous concrete mixes, the standard operating procedure for selecting the job mix formula is the Marshall method. However, the NJDOT determines the job mix formula for the open graded friction course by making trial batches having different asphalt cement contents and gradations consistent with the job mix specifications. The trial batches mixtures are spread evenly on heat resistant transparent pyrex dishes and placed in an oven for 1 hour. After the hour, the dishes are examined for a slight puddle at the points of contact between the aggregate and the glass dish and compared to photographs of desirable drainage conditions. The mixture meeting the "slight" puddle condition is selected for determination of the asphalt cement content. The job mix formula for the CRM open graded friction course is shown in Table 7.

A 2.7-tonnes (3-ton) Barber Green batch-type asphalt plant was used in the production of the mix. Similar to the above Route I-95, Ewing, project, the fine ground tire rubber was blended with the asphalt cement. The mix was discharged from the plant between 143°C and 160°C (290°F and 320°F). The laydown temperatures of the material were above 138°C (280°F) and ambient temperatures were above 16°C (60°F).

TABLE 7 Summary of Mix Gradations, Wet Process CRM, Open Graded Friction Course, Project No. 3, I-195 Job Mix Formula

Sieve Size	Percent Passing: NJDOT Specification OGFC	Percent Passing: CRM-OGFC Sample
12.7 mm	100	100
9.5 mm	80-100	94
No. 4	30-50	39
No. 8	5-15	10.5
No. 200	2-5	2.9
A. C./Rubber	5.7-7.0	6.6

Although some material cooled prematurely in the trucks causing clumping, no significant paving problems were noted on the project. Essentially, no asphalt cement runoff was noted in the trucks on the project.

Density measurements from cores indicated that the in-place air voids were above 15 percent as specified. The initial average skid measurements were good (SN = 52) for both lanes and the average Aran Ride Quality Index was good (ARQI = 4.23) for both lanes.

The air quality emissions testing was conducted according to the above methods and procedures at the asphalt plant. The concentration data were collected for three 1-hour tests. The data for total hydrocarbons, carbon monoxide, and particulates are below the standard. An odor survey was conducted upwind and downwind of the source at the property line; no odors were detected. The opacity (percent) of the plume from the baghouse exhaust was determined similar to the Route I-95 project. Observations were made at 60-sec intervals and were performed simultaneously with the emissions sampling. The opacity was 0.0 percent for all three runs.

Dry Process CRM Mix (Plus Ride)

The propriety gap-graded dry process CRM mix was constructed on a resurfacing project on Route 130, Logan Township in October/November 1993.

The gap-graded CRM mixes for the surface and base course contained 3 percent ground tire rubber by weight of the mix. The mix and rubber gradations are shown in Table 8. The Plus-Ride mixes are gap graded to allow space for the crumb rubber. For the Plus-Ride II Mix 12 and Mix 16, the gap grade was defined by restricting the amount of aggregate passing the No. 4 sieve and retained on the No. 8 sieve to be 8 percent. The gaping within this band of aggregate gradation is critical.

The crumb rubber was granulated reclaimed vulcanized rubber produced primarily from the processing of automobile and truck tires. The crumb rubber must be sufficiently dry so as to be free flowing and free of wire.

The mixes were produced in a 3.6-tonnes (4-ton) H & D batch type plant. For the CRM mix, the plant discharge temperatures were 151°C to 157°C (305°F to 315°F) and the laydown temperatures were 143°C to 151°C (290°F to 305°F).

Although paving was completed without significant material handling and compaction problems, the pavement began to ravel shortly after construction. Some of this serious deterioration is attributed to paving in the rain, but a significant portion of the deteriorated surface was not paved in the rain and cannot be explained without further testing. The deterioration appears to be isolated to the surface course, without affecting the base course material.

The air quality emissions testing parameters and methods were similar to the above tests. Three 1-hour tests were performed on the baghouse outlet during production of the standard surface course mix and the dry process CRM surface course mix. Previous emissions tests of this plant indicated typical carbon monoxide (CO) emissions under 150 ppm and total hydrocarbons (THC) under 200 ppm for virgin materials. On this project, the total hydrocarbons, carbon monoxide, and particulate emissions for the standard mix were below the emissions standard, and below the typical emissions for the plant. However, the THC for the CRM mix were above the

TABLE 8 Summary of Mix Gradations, Dry Process—Plus Ride II Mixes, Project No. 4, Route 130, Job Mix Formula

Sieve Size	Percent Passing: Plus Ride II - Mix 16 Base Course Sample	Plus Ride II - Mix 12 Surface Course Sample
19 mm	100	--
15.8 mm	---	100
9.5 mm	57	70
No. 4	34	34
No. 8	26	28
No. 30	19	19
No. 200	9.5	9.9
Asphalt Cement	7.7	7.7
Crumb Rubber	3.0	3.0

Sieve Size	Ground Tire Rubber Gradation (% passing)
6.3 mm	100
No. 4	76 - 100
No. 10	28 - 42
No. 20	16 - 24

Specific Gravity 1.15+/-0.05

typical emissions level of the plant, and higher than the THC for the standard mix. One value of THC exceeded the emissions standards. The CO and particulate emissions for the CRM mix were below the emissions standard.

Wet Process CRM Mix (McDonald)

For the third wet process project, a CRM bituminous concrete surface course and CRM bituminous stabilized base course were constructed on the southbound lanes of a resurfacing project on Route I-95, Ewing Township. The asphalt-rubber cement was blended at the plant by Asphalt Rubber Systems.

The asphalt plant was a 2.7-tonnes (3-ton) Barber Green batch type plant. The asphalt cement (AC-10) with an extender oil was blended with 18 percent Baker Rubber TR-24 and TBS-20 crumb rubber in the on-site portable blending unit, heated to 177°C (350°F) and mixed with the aggregates and baghouse fines in the mixer for 35 sec. The tire rubber was supplied in paper bags. The mix was discharged from the plant between 143°C and 160°C (290°F and 320°F).

The mix gradations are shown in Table 9. All mixes used the NJDOT standard specifications for gradations. The standard mixes contained 10 percent to 20 percent RAP. Laboratory test properties indicated that the Marshall stability, flow, and voids were similar for all mixes. The physical properties of the CRM asphalt cement are shown in Table 10.

The construction of the CRM surface course and base course was preceded by milling 50 mm (two in.) of the existing bituminous concrete pavement to eliminate rutting. A 75-mm (three-in.) bituminous stabilized base was placed on the milled surface. The 50-mm (two-in.) CRM surface course was placed on the base course. The CRM mix was placed as a surface course on the shoulders and mainline pavement with a standard paver. A breakdown roller, intermediate roller and a finish roller were used on the CRM surface course. The laydown temperatures were above 143°C (290°F). All in-place density measurements were within specifications. No problems were noted during paving.

The air quality emissions testing parameters and methods were similar to the above tests. Three 1-hour tests were performed on the baghouse outlet during production of the standard surface course mix and the wet process CRM surface course mix. For the CRM mix, the total hydrocarbons, opacity, and particulate emissions were above the limit. The total hydrocarbons were significantly higher than the standard mix. For the standard mix with RAP, the particulate emissions and opacity were also above the limit. Production problems were not apparent during the tests.

Generic Dry Process CRM Mix

The generic dry process was constructed on a lane reconstruction project on Route I-287, Bedminister, in November 1993 and June 1994.

The experimental design of the project consisted of (1) a test section of the generic dry process CRM bituminous concrete surface with 1 percent crumb rubber by weight of the mix on the standard bituminous stabilized base course, (2) a test section of the 1 percent CRM bituminous concrete surface on the CRM bituminous stabilized base course with 2 percent crumb rubber, (3) a test section of the 1 percent CRM surface course on the CRM/crushed container glass base course with 10 percent crushed container glass and 2 percent crumb rubber, and (4) a control section of the standard surface course and stabilized base course. The test sections were placed in the newly constructed northbound outside lane.

TABLE 9 Summary of Mix Gradations, Wet Process CRM Mix, Project No. 5, Route I-95

	Percent Passing-Job Mix Formula				NJDOT Specification
Sieve Size	Standard Surface Cr.	CRM Mix Surface Cr.	Standard Base Course	CRM Mix Base Course	Mix Size 37.9mm Base Course
50.8 mm	--	--	100	100	100
37.9 mm	--	--	100	100	90-100
25.4 mm	100	100	100	100	80-100
19 mm	99	99	--	--	--
12.7 mm	93	90	76	73	50-85
9.5 mm	--	80	--	--	--
No. 4	58	51	52	47	25-60
No. 8	45	34.5	41	35.0	20-50
No. 16	35	27	--	--	--
No. 30	26	21	--	--	--
No. 50	17	14	16	14.0	8-30
No. 200	5.8	4.4	5.5	5.4	4-12
A.C.	4.8	5.90	4.5	5.2	3.5-8

Ground Tire Rubber Gradation

Sieve Size	(% passing)
No.10	100
No.16	75-100
No.30	25-100
No.80	0-20
No.200	0-5

The CRM job mix formulas were performed by TAK Engineering using the Marshall method and are shown in Table 11. All mixes used the NJDOT standard specifications for mix gradations. The laboratory test properties were similar for all mixes. The standard mixes contained 10 percent to 20 percent RAP.

The mixes were processed through a 4.5-tonne (5-ton) McCarter batch type plant. The crumb rubber was introduced into the mixer from preweighed bags and mixed thoroughly for a minimum of 20 sec prior to introducing the asphalt cement. The wet mixing time was not less than 40 sec. The mix was discharged from the plant at between 143°C and 160°C (290°F and 320°F). The laydown temperatures were above 143°C (290°F). All in-place density measurements were within specifications. No problems were noted during paving.

The air quality emissions testing parameters and methods were similar to the above tests. Three 1-hour tests were performed on the baghouse outlet during production of the standard surface course mix and the generic dry process CRM base course mix. For the CRM mix, the total hydrocarbons, particulate, and one CO emissions values were higher than the standards. For the standard mix with RAP, the total hydrocarbons and particulate emissions were above the limit. The high emissions values for the CRM mix were attributed to intermittent production during a rainy period. The same emissions values for the standard mix were taken inadvertently during production of the plant's driveway mix which contained an unknown amount of RAP.

ANALYSIS AND DISCUSSION

This report describes the mix design and air quality asphalt plant emissions testing of crumb rubber modified bituminous concrete mixes. The wet process and the dry process were used to incorpo-

TABLE 10 Physical Properties of CRM Asphalt Cement, Wet Process CRM Mix, Project No. 5, Route I-95

Tests performed at 135 minutes reaction time	
Brookfield Viscosity @350°F, cp	3100
Viscosity, Haake@350°F, cp	1800
Penetration, needle@77°F (1/10mm, 100g, 5 sec.)	100
Penetration, needle@39.2°F (1/10mm,200g., 60 sec.)	61
Resilience@77°F % rebound	24
Ductility @39.2°F 1cm per min.	20.2
Softening Point °F	135.0
Cone Penetration @77°F 1/10mm	86

TABLE 11 Summary of Mix Gradations, Dry Process CRM Mix, Project No. 6, Route I-287

Sieve Size	Percent Passing-Job Mix Formula				
	Standard Surface Cr.	CRM Mix Surface Cr.	Standard Base Course	CRM Mix Base Course	CRM-Glass Base Course
50.8 mm	--	--	100	100	100
37.9 mm	--	--	100	100	100
25.4 mm	100	100	98	98	98
19 mm	100	100	--	--	--
12.7 mm	93	96	66	66	67
9.5 mm	77	77	--	--	--
No. 4	54	48	38	43	44
No. 8	42	32	30	30.0	30.0
No. 16	29	24	--	--	--
No. 30	20	18	--	--	--
No. 50	16	14	14	13.0	11.0
No. 200	6.6	6.0	5.8	7.6	5.9
A.C.	5.4	5.90	4.0	5.0	5.70
Rubber	--	1.0	--	2.0	2.0

Sieve Size	Ground Tire Rubber Gradation (% passing)	
	CRM Surface Course	CRM Base Course
No.4	--	100
No.8	100	63-77
No.16	63-77	45-55
No.30	45-55	27-33
No.50	22-28	8-12
No.80	8-12	--

rate crumb rubber into standard bituminous concrete mixes, an open graded friction course, a recycled rubber RAP, and a propriety gap-graded mix. The mix gradations are presented for six CRM bituminous concrete pavement resurfacing and lane widening projects. Asphalt plant emissions testing data are presented for the projects.

Using NJDOT standard mix gradations for bituminous concrete surface and base course, the mix designs were developed successfully for three wet process CRM mixes using 10 percent, 15 percent and 18 percent crumb rubber by weight of an AC-20 or AC-10 asphalt cement. The wet process used an on-site blending unit that heated the asphalt cement and crumb rubber to 176°C (350°F) before addition to the aggregate. In the asphalt plant, the CRM mixtures were heated about 6°C (10°F) higher than standard mixtures. Normal paving and compaction procedures were used for the projects. The paving temperatures were about 6°C higher than standard paving mixes. The wet process projects are performing initially similar to the standard paving mixes. The air quality emissions tests at the asphalt plants indicated mixed results with one project having test results above allowable standards.

Laboratory and modulus of resilience data indicate that the wet process mixes with AC-20 asphalt cement are somewhat stiffer than standard mixes. It is assumed that the stiffer CRM binder may resist rutting better than the virgin asphalt cement. However, the stiffer binder may be more susceptible to cracking.

For the open graded friction course mix, the CRM asphalt cement did not strip from the aggregate during transportation. As consistent with any open graded friction course, plant discharge and laydown temperatures are critical for handling and compaction. Previous experience has indicated that if the material cools, it has a tendency to clump and not provide desired compaction. Furthermore, if the plant mix temperatures are too high, the asphalt cement has a tendency to strip. From observations on this project, the CRM mix that was heated higher in the plant seemed to provide a better coating of the aggregate than standard OGFC mixes.

The mix design was developed successfully for a rubber reclaimed asphalt pavement (RAP) project which used 20 percent rubber RAP in a fine aggregate surface course. The rubber RAP was milled from a raveled gap-graded pavement. The construction data indicated that the density measurements are good. The air quality emissions tests indicated that the total hydrocarbons, and particulate emissions were within limits. However, one carbon monoxide reading was at the limit.

The propriety gap-graded dry process CRM mix design was constructed in the surface and base course of a resurfacing project. The 3 percent CRM mix required more asphalt cement, and the mix required the usual higher production temperatures. The CRM surface course mix raveled soon after construction and required replacement with the standard surface course. There were no problems with the CRM base course. The air quality emissions tests of the CRM mix indicated that the carbon monoxide readings were higher than the standard mix, and one reading was over the emissions limit.

Using NJDOT standard mix gradations, three generic dry process mixes were constructed as a lane widening. Containing 1 percent and 2 percent CRM, the mixes were developed for surface course, base course, and base course with 10 percent crushed container glass. Without blending equipment or additional processing, the generic dry process adds ground tire rubber directly into the mixer with the aggregate. Constructed in November 1993 and June 1994, the pavement surface course and base course look good. Some air quality emissions data for both mixes were above the limits. Intermittent production of the CRM mix may have contributed to high emissions values.

CONCLUSIONS

1. Using the NJDOT standard mix gradations and test properties, crumb rubber modified bituminous concrete mixes can be successfully developed with the NJDOT standard job mix formula methods, produced using the wet process and the generic dry process and constructed on overlays and new construction.
2. The wet process and generic dry process CRM mixes have similar Marshall stability, flow, and void test parameters compared with the NJDOT standard mixes. The wet process and generic dry process CRM mixes meet present test properties specifications and mix gradations.
3. From two projects, the initial field data such as rutting, cracking, smoothness, nuclear density, and skid resistance indicate that the CRM mixes produced by the wet process were similar to the standard mixes.
4. Using an NJDOT standard bituminous concrete mix gradation, rubber RAP from a gap-graded CRM mix can be successfully recycled in a bituminous concrete surface course. Additional rubber RAP projects should be conducted for wet and dry process RAP and constructed on the state highway system.
5. Although air quality plant emissions tests from two CRM mix projects indicated acceptable emissions levels, the emissions tests of the remaining four CRM mix projects indicated some unacceptable emissions levels. In addition, the emissions levels from the production of CRM mixes were generally higher than standard mixes.
6. The CRM asphalt cement in the OGFC permitted higher mix temperatures, provided better aggregate coating, and essentially eliminated asphalt cement runoff during transportation.
7. For the CRM bituminous concrete mixes, the plant discharge and the laydown temperatures were higher than standard bituminous concrete mixes.
8. The CRM bituminous concrete mixes required higher percentages of asphalt cement than the standard mixes.

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Mechanistic Evaluation of Asphalt Concrete Mixtures Containing Reclaimed Roofing Materials

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Recycling of waste construction materials has gained popularity owing to increasing demands on landfill sites. This is evident by the use of ground rubber tire, glass, fly ash, and slag in asphalt pavements by various highway agencies in North America and around the world. Waste roofing materials also pose a heavy burden on landfill sites. Using reclaimed roofing materials (RRM) in hot mix asphalt (HMA) concrete pavements can lessen the demand on landfills. A study was carried out to determine the feasibility of using RRM in HMA pavement. This paper presents the results of a mechanistic evaluation of three asphalt concrete mixes containing 0, 15, and 25 percent of RRM. By using laboratory prepared specimens of RRM mixes, mechanical properties such as resilient modulus, creep and permanent deformation, fatigue, and moisture sensitivity of these RRM mixes were determined. Performance of representative RRM pavements were modeled using the VESYS performance prediction model. Performance parameters, such as rut depths, cracking index, and the present serviceability index, were used to assess potential improvements of asphalt concrete mixes using RRM. The results indicated that the mix containing 25 percent of RRM exhibited significant improvements in greater pavement rutting resistance, longer fatigue life, and better overall pavement performance compared with a conventional asphalt mix.

Recycling has become increasingly popular due to a heightened awareness of the environmental impact of waste disposal. The disposal of waste roofing material is problematic because of its high asphalt content. Roofing material does not break down naturally; the degree of disintegration is insignificant even over long periods of time. The needs for new housing and replacement of aged asphalt roofing increase as society grows. The amount of waste roofing material produced will increase annually. Currently, it is estimated that the United States produces 12 million tons of waste asphalt roofing material each year. In Canada, more than 100,000 tons of waste roofing material is deposited into landfills annually.

Many landfills have imposed a fee for asphalt roofing waste or have banned it altogether. An alternative for disposing of waste asphalt roofing is needed. The high asphalt content and crushed stone aggregates in RRM suggest that it would be compatible with HMA. Studies have shown that it is feasible to recycle waste roofing materials in HMA pavements (*1–3*). In practice, however, the use of RRM in HMA pavements has been limited to test sections and laboratory evaluation and has yet to be used in large-scale production.

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The objectives of this paper are

- To provide an overview of the availability and feasibility of using common residential and commercial RRM in HMA;
- To present the results of a mechanistic evaluation of three different asphalt concrete mixes containing 0, 15, and 25 percent of RRM, respectively;
- To evaluate potential improvements in mechanical properties of asphalt concrete mixes with the use of RRM; and
- To evaluate the effect of RRM to mitigate pavement distress and improve pavement performance.

BACKGROUND

Although a relatively new concept, recycling waste roofing materials for use in asphalt paving is a growing and promising practice. One roofing material recycler used the motto "Recycling your roof to repair your road" (*4*). A number of highway authorities have made extensive use of cold patch compounds containing RRM to repair potholes. These hybrid compounds, compared to conventional cold patch compounds, can be applied more quickly and easily, are less expensive, and stay in place longer (*4*).

The properties of the various components in RRM make it a satisfactory substitute for many commercial additives presently used in HMA. Table 1 lists components commonly found in recycled roofing asphalt and their commercial equivalents. Significant economic savings can be achieved if a single additive composed primarily of RRM replaces the numerous and more costly additives currently used.

IKO Industries Ltd., in conjunction with the City of Brampton, and DBA Engineering Ltd. of Markham, Ontario, initiated a test project in 1994 which uses waste roofing materials in HMA pavements.

ReACT's HMA™, a commercially available recycled roofing material, is produced by ReClaim Inc. of Tampa, Florida. Currently, this is the only commercially available RRM material on the market which is specifically produced for use in HMA pavements. Grzybowski, et al. (*1*) found that up to 50 percent net asphalt savings could be achieved by using ReACT's HMA™ as an additive in HMA pavements. They also recorded improvements in terms of high temperature susceptibility and rutting resistance.

Paulsen, et al. (*2*) carried out laboratory testing to determine the feasibility of incorporating waste roofing materials in HMA pavements. The scope of their work included carrying out material com-

TABLE 1 Composition of Roofing Asphalt Recycled

Component	Commercial/Functional Equivalent
Fibres	Minerals, Cellulose in SMA's, Polypropylene
Fillers	Carbon Black, Limestone, Hydrated, Lime, Diatomaceous Earth
Hard Asphalt	Gilsonite, Trinidad Lake Asphalt, Propane Precipitated Asphalts

* after (1)

position analysis of waste roofing materials and determining an optimum quantity that can be added to a hot mix design. Their results indicated that up to 20 percent of recycled roofing material by volume (10 to 12 percent by weight) could be added to HMA while still providing adequate performance properties. They also noted that current methods of extracting asphalt from recycled roofing materials produced unsatisfactory binder samples with inconsistent penetration and viscosity. They further suggested that the properties of shingle asphalt and the gradation of shingle aggregates should be considered when formulating HMA with RRM.

Newcomb, et al. (*3*) examined the use of roofing shingles in densely graded and stone mastic asphalt (SMA) mixes. They found that up to 7.5 percent of asphalt roofing waste can be incorporated in densely graded mixes, and up to 10 percent can be added to SMA mixes. They also found that the addition of roofing materials lowers the resilient modulus at low as well as at high temperatures and that, in general, the roofing waste mixtures exhibited less temperature susceptibility. The resilient modulus of SMA mixes tended to remain relatively constant despite variations in the amount of waste roofing incorporated. However, the tensile strength of SMA mixes examined were 10 percent lower than those of the control mix.

MATERIALS

Common RRM discarded by contractors at residential sites was used in this research project. Table 2 shows compositions for both residential and commercial RRM. The commercial RRM sample acquired contained numerous layers of felt, tar paper, organic fibre, wood pieces, nail, and metal flashing not found in residential RRM. The presence of a number of foreign materials in commercial RRM would make it difficult to maintain uniform composition in shredded commercial RRM. Furthermore, commercial RRM only constitutes up to 10 percent of total roofing waste deposited in landfills. Therefore, commercial RRM was not included in this study.

Abson Recovery Method (ASTM 2172) was used to extract asphalt cement from samples of RRM. A discussion of methods for extraction and separation of asphalt cement can be found in ASTM D1856. Table 3 shows physical properties of recovered asphalt cement from both commercial and residential RRM.

The viscosity of both residential and commercial RRM asphalt was much higher than that of a typical paving grade asphalt. This is due to the oxidation of RRM during the service life of roofing mate-

TABLE 2 Composition of Reclaimed Roofing Materials

Composition Breakdown (% by wt.)	Residential	Commercial
Asphalt Shingles	96	89
Metal Flashing	0.14	2.5
Nails	0.5	3.4
Plastic Strips	0.06	0.35
Felt Underlayment	3.3	4.75
Total	**100**	**100**

TABLE 3 Physical Properties of Recovered Asphalt Cement

	Commercial RRM	Residential RRM
Asphalt Cement Content (%)	83.9*	38.7*
Pene. at 25°C in 0.1 mm (ASTM D-5)	15.0	17.0
Kinematic Viscosity at 135°C in Cst. (ASTM D-2170)	28,200	54,000
Specific Gravity (ASTM D-3124)	1.032	1.000

* Following the removal of debris.

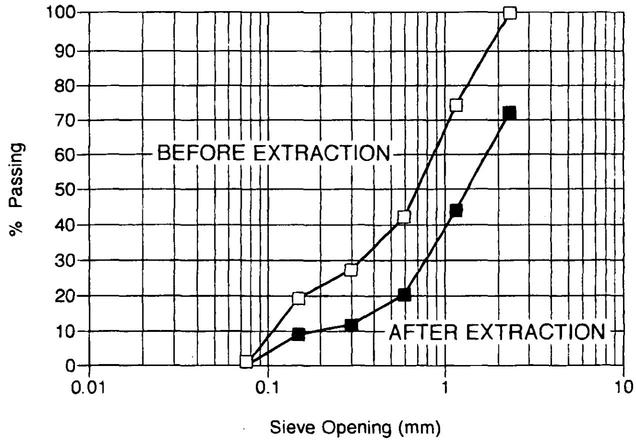

FIGURE 1 Typical grading curves of roofing waste before and after extraction.

rials. A comparison of asphalt cements extracted from residential and commercial RRM showed that while penetration values were comparable, the viscosity of residential RRM asphalt cement was almost twice that of commercial RRM asphalt cement. The high viscosity of asphalt cement from residential RRM compared to that of commercial RRM is likely due to the thick and dense consistency of commercial roofing materials. Figure 1 shows the gradation of RRM before and after the extraction process. Extraction of asphalt cement reduced the top size particles by approximately one sieve size. This is consistent with findings of Paulsen, et al. (*2*).

Materials used in the preparation of laboratory specimens were

- Crushed aggregates meeting the Nova Scotia Department of Transportation and Communication (NSDOT&C) standards for Type C aggregate. Aggregates were obtained from a quarry located in Bedford, Nova Scotia. Table 4 lists physical properties of aggregates used.
- ASTM designated 200 to 300 penetration grade asphalt cement supplied by the ESSO refinery in Dartmouth, Nova Scotia.
- Shredded residential RRM as described in previous sections.

MIX DESIGN

RRM Shredding

It was found that the best way to add RRM to HMA was in the form of a fine aggregate. Raw RRM must be shredded by some means. The most successful method for shredding RRM in the laboratory involved freezing raw RRM to approximately $-10°C$ and then shredding it with a 10-in. circular carbide tipped blade on a chop saw. The particles produced passed No. 4 sieve size. Other methods tried, including a commercial tire shredder, produced enough heat to melt the asphalt cement in the RRM, resulting in "gumming up" the shredder, and producing a poor quality RRM additive.

Preliminary Investigation

In a preliminary investigation, the Marshall Method of Mix Design and the guidelines set forth by NSDOT&C for a Type C mix were used. The purpose of this phase of investigation was to establish satisfactory procedures for adding RRM to an HMA and to identify potential problems that may arise and affect the quality of Marshall briquette specimens. Further, percentages of RRM that can be added to the HMA had to be predetermined to avoid the needless testing of unlikely ratios.

TABLE 4 Physical Properties of Aggregates

Size Fraction	Bulk S. G.	Apparent S.G.	% Absorption
Coarse Aggregate (ASTM C127)	2.549	2.616	1.41
Fine Aggregate (ASTM C128)	2.579	2.660	1.21
Filler (ASTM D854)		2.651	
RRM (ASTM C128)		2.10	

Sieve Size	% Passing			Specification
	Mix A	Mix B	Mix C	
20.0 mm	100	100	100	100
14.0 mm	95	96	95	95-100
4.75 mm	50	47	55	45-70
2.36 mm	30	26	29	25-55
300 μm	13	12	13	5-20
75 μm	5	2	5	2-9

TABLE 5 Marshall Mix Criteria

	Mix Designations		
	A	B	C
% of RRM	25%	15%	0
(%) Opt. Asphalt Content	1.85	3.4	5.25
Bulk Specific Gravity	2.350	2.340	2.410
Max. Theo. Specific Gravity	2.420	2.440	2.490
(%) Void	3.25	3.95	3.5
Stability, kN	18	14	9.3
Flow, 0.25 mm	8.5	10.0	8.0
VMA (%)	14.0	14.0	15.0

Preliminary investigation showed that the addition of cold RRM to virgin aggregates followed by mixing and heating was not feasible. If heating was done after the addition of RRM, asphalt cement in the RRM would separate and bond the entire mix together. Once bonded, mixing was difficult.

The only satisfactory results achieved were accomplished by heating the virgin aggregates alone to the maximum allowed temperature (150°C) and then adding the RRM. When room temperature RRM was added to hot aggregates, the mixture could be blended thoroughly since the RRM did not have time to melt and consequently form clumps in the aggregate. Once the RRM was added to the aggregates, virgin asphalt cement could then be added.

During the preliminary investigation, neither the requisite amount of virgin asphalt cement nor the contribution made by the RRM was known. Consequently, virgin asphalt cement was added until a coating and mix consistency resembling that of the control mix, at optimum asphalt cement content, was achieved. A more accurate optimum virgin asphalt cement content was determined from a series of HMA test mixes containing varying percentages of RRM.

RRM was added at increments of 5 percent and ranged from 5 percent to 50 percent by weight of HMA mixes. The addition of 5 percent and 10 percent of RRM had little effect, in terms of Marshall flow and stability, on HMA briquettes produced. At more than 25 percent of RRM, briquettes produced were unsatisfactory and crumbled easily. Not surprisingly then, stability and flow values from these samples were unacceptable.

These results suggested that the optimum amount of RRM to be added ranged from 15 percent to 25 percent. Potentially, the addition of less than 15 percent RRM could produce satisfactory results, but because the addition of 10 percent or less of RRM had limited effects, the minimum RRM content was fixed at 15 percent.

Mix Design

Three mixes were prepared for engineering evaluation and performance analysis:

- Mix A: 25 percent RRM;
- Mix B: 15 percent RRM; and
- Mix C: Control mix (0 percent RRM).

Room temperature RRM was added to and mixed with aggregates, which were preheated to approximately 150°C. Virgin asphalt cement was then added to the mix and again blended thoroughly. Although the mixing time was increased, special attention was given to the mix temperature to prevent burning of the binder in the RRM. The hot mix was then compacted following the 75-blow Marshall procedure. Table 5 shows the Marshall mix parameters for the three mixes prepared. All three mixes satisfy the Marshall mix design criteria and the virgin asphalt cement content has been reduced from 5.25 percent for Mix C (control mix) to

TABLE 6 Resilient Modulus MPa (Ksi)

	Temperature					
	0°C		20°C		40°C	
	F1	F2	F1	F2	F1	F2
Mix A	18550	14800	7270	5400	3150	2350
Mix B	13400	11600	6350	3400	1200	780
Mix C	14700	12650	2300	1400	300	220

F1 = 1 Hz; F2 = 0.33 Hz

1.870 percent for Mix A (25 percent RRM), a reduction of almost 3 percent.

LABORATORY TESTING AND RESULTS

Resilient Modulus

The diametral resilient modulus (M_r) test method detailed in ASTM D4123 was used in this study. Repeated haversine loading was used in all resilient modulus testing to avoid impact loading to specimens. Three levels of temperature, 0°C, 20°C, and 40°C, and two load frequencies, 1 Hz (0.1 sec loading and 0.9 sec unloading) and 0.33 Hz (0.25 sec loading and 2.75 sec unloading) were used. Results of resilient modulus testing are summarized in Table 6.

Resilient modulus has a well-defined negative correlation with test temperature. M_r decreased from about 18,000 MPa at 0°C to about 3,000 MPa at 40°C for Mix A and from about 15,000 MPa to about 300 MPa for Mix C. The results indicate that the addition of RRM in mixes increased the stiffness of mixes. This is consistent with findings of previous research conducted on HMA mixtures, which indicated that M_r reflected the stiffness of the binder used.

Results in Table 6 also show that the addition of RRM improves M_r characteristics of a conventional mix, especially at an elevated temperature (40°C). M_r of 25-percent RRM mix (Mix A) was about 10 times that of the conventional mix (Mix C). It was evident that rutting resistance could be improved by the addition of RRM. On the other hand, at a low temperature (0°C) the M_r value of the 25-percent RRM mix was about 1.5 times that of the conventional mix. Therefore, low temperature cracking potential should not be adversely affected by the addition of RRM.

Creep and Permanent Deformation

Indirect tensile loading was used to determine the effect of RRM on the viscoelastic behavior of paving mixtures. This behavior is usually measured by the creep and the permanent deformation parameters. Creep and permanent deformation tests were conducted in accordance with procedures outlined in the VESYS manual (*5*). The objective of this test series was to obtain modeling parameters that would be used to predict the rutting performance of a pavement.

Specimens were tested under a constant stress of 20 psi at 0°C, 20°C, and 40°C. The permanent deformation characteristics of the three mixes are in Figure 2. As expected, permanent deformation increased exponentially to loading times in all temperatures ranges. Figure 2 also shows that the 25-percent RRM mix exhibited the lowest permanent deformation at all test temperatures owing to the stiffening effects of RRM. This inferred that the 25-percent RRM mix (Mix A) had the least rutting potential. The conventional mix, Mix C, did not survive testing at 40°C. Test results were used to calculate the permanent deformation modeling parameters, ALPHA and GNU, for the VESYS structural subsystem to evaluate rutting potential of representative pavement structures.

Data from the 1,000-sec loading creep test were used to generate creep characteristic curves (Figure 3). As expected, the creep moduli decreased with increased loading time and/or temperature. The results in Figure 3 indicated that the addition of RRM to the mixes increased the creep modulus values. Creep modulus values increased as RRM content increased. Results also showed that Mix A had the highest creep modulus values. This was consistent with permanent deformation test results.

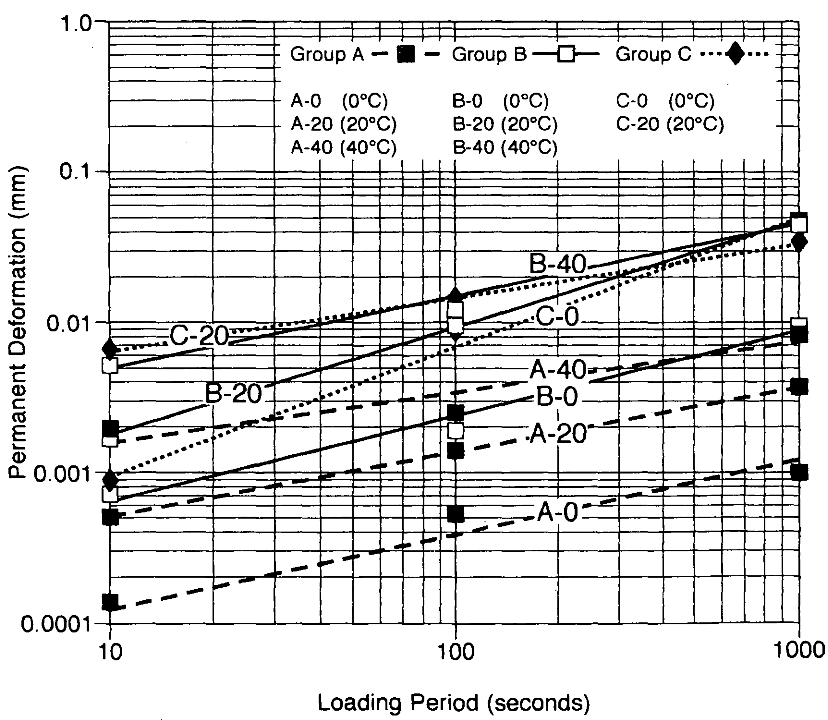

FIGURE 2 Permanent strain from incremental static loading test at 0°C, 20°C, and 40°C.

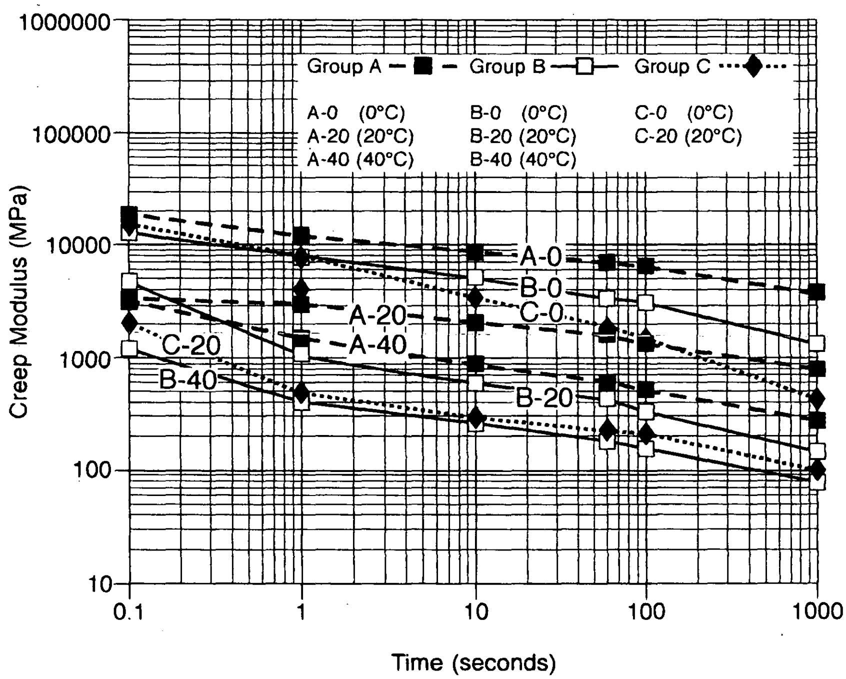

FIGURE 3 Creep modulus versus loading time at 0°C, 20°C, and 40°C.

Fatigue

Fatigue characteristics of the three mixes were measured using diametral indirect tensile tests in controlled stress mode. All tests were conducted at a temperature of 21°C. A load frequency of 1 Hz with 0.1 sec loading and 0.9 sec unloading was used. Indirect tensile stresses in the range of 4 to 50 kPa were used.

Fatigue analysis required an evaluation of both the induced tensile strain in the paving mixture and relation of this tensile strain to the allowable number of load applications. This analysis was performed by using the following equation:

$$N_f = K_1 \left(\frac{1}{\epsilon}\right)^{K_2} \qquad (1)$$

where

N_f = the number of load applications to failure,
ϵ = the initial tensile strain, and
K_1, K_2 = the material constants which can be determined through regression.

Strain increased continually throughout the duration of the controlled stress test. The initial strains reported were indirect tensile strains obtained at mid-height of specimens after 200 load applications.

Fatigue characteristic curves for all mixes are shown in Figure 4. The fatigue parameters, K_1 and K_2, can be found in Table 7. Values of K_1 and K_2 can be used as indicators of how RRM content affects

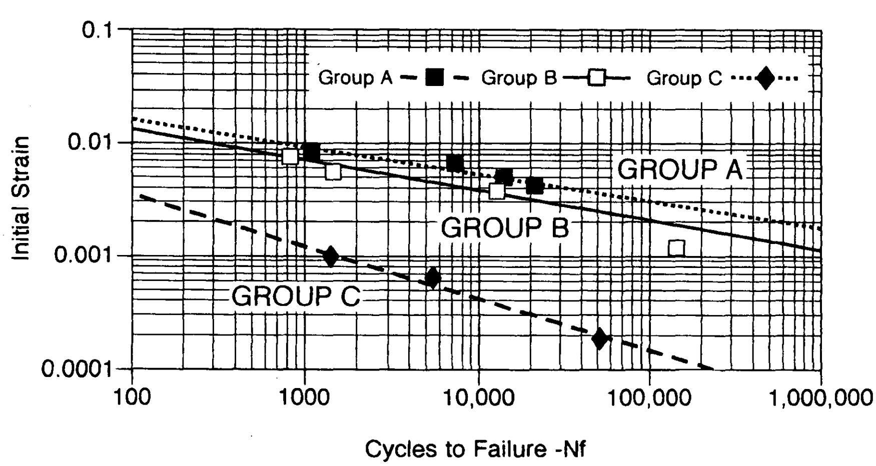

FIGURE 4 Fatigue characteristics.

TABLE 7 VESYS Modeling Parameters

	Season							
	Winter		Spring		Summer		Fall	
Resilient Modulus - MPa								
Mix A	18500		7270		3150		7270	
Mix B	13400		6350		1200		6350	
Mix C	14700		2300		300		2300	
Permanent Deformation Parameters								
	μ	α	μ	α	μ	α	μ	α
Mix A	0.14	0.8	0.67	0.79	0.53	0.79	0.67	0.79
Mix B	0.18	0.73	0.55	0.51	0.69	0.69	0.55	0.51
Mix C	0.4	0.59	0.65	0.84	0.71	0.19	0.65	0.84
Fatigue Coefficients								
	K_1				K_2			
Mix A	$1.4 * 10^{-7}$				4.81			
Mix B	$4.1 * 10^{-5}$				3.43			
Mix C	$2.6 * 10^{-4}$				2.28			

the fatigue mechanism of a paving mixture. The flatter the slope of the fatigue curve, the larger the value of K_2. If two materials have the same K_1 value, a large value of K_2 indicates a potential for longer fatigue life. On the other hand, a smaller K_1 value represents a lower fatigue life when fatigue curves are parallel (i.e., K_2 is constant). Two intersecting fatigue curves indicate that the magnitude of initial induced strain would determine which material would have a longer fatigue life.

Results in Figure 4 and Table 7 show an increase in K_2 and a decrease in K_1 as the RRM content increased. This indicates that the use of RRM increases fatigue properties of HMA mixes. To examine the combined effect of these parameters, K_1 and K_2 were used as input in the VESYS model to predict the fatigue distress (cracking index).

Moisture Damage

Moisture damage evaluation can be accomplished using a number of different methods. Procedures for evaluating the potential for long-term moisture damage as outlined in NCHRP 246 were used in this study. This method used either the ratio of the resilient modulus or the indirect tensile strengths of wet (moisture conditioned) and of dry (unconditioned) specimens as indicators of moisture damage susceptibility.

All three mixes were conditioned as prescribed. Diametral resilient modulus and indirect tensile tests were performed on conditioned and unconditioned specimens. Table 8 summarizes the results of moisture damage tests. Previous research indicates that moisture damage or stripping can occur in asphalt concrete pavements when the ratio of dry to conditioned test specimens is below 0.70 to 0.75 (*6*).

Results in Table 8 indicate that ratio values calculated for both M_r and tensile tests were above 0.9 which suggests that all mixes are not prone to stripping. The use of RRM had no adverse effects on the moisture damage resistance of asphalt concrete. Also noteworthy was that the tensile strength of mixes increased with an

TABLE 8 Moisture Damage Results

		Mix A	Mix B	Mix C
Resilient Modulus MPa	Dry	5120	4200	1900
	Wet	4850	4050	1810
	Ratio	0.94	0.96	0.95
Tensile Strength kPa	Dry	757	446	233
	Wet	734	407	217
	Ratio	0.97	0.92	0.93

increase in RRM content. Tensile strengths ranged from 233 kPa for the control mix to 757 kPa for a 25-percent RRM mix, which likely resulted from the reinforcing effect of fibres in RRM.

PERFORMANCE ANALYSIS

To assess the influence of RRM on pavement performance, three representative pavement sections were selected for analysis. Each section had a 150-mm (6-in.) asphalt concrete layer over a 300-mm (12-in.) base course layer. The difference between the three pavement sections was in the type of asphalt concrete mix used in the surface layer: two sections used RRM mixes (Mix A and Mix B), while the third had a crushed aggregate mix (Mix C). For the purpose of predicting pavement performance, the VESYS IIIA structural subsystem was used.

Performance can be expressed in terms of rutting, cracking, roughness, and present serviceability index (PSI). Full details of the VESYS model are described in the FHWA Report (*5*).

The mechanical properties of the asphalt concrete layers are summarized in Table 7. The properties of the granular base course and the subgrade layer were identical for all three structures. An analysis period of 20 years and an average traffic of 130 equivalent single axle load (ESAL) per day were used in the analysis. A summary of the VESYS model results are shown in Table 9.

Rut depth, a measure of permanent deformation in the wheel path, is a function of permanent deformation parameters, stiffness of the materials, and traffic volume. As shown in Table 9, pavement sections constructed with Mix C (0 percent RRM) and Mix B (15 percent RRM) will have 12.5 mm (0.5 in.) of rutting during service lives of 5 to 7 years, respectively. The pavement constructed with Mix A (25% RRM) will take 12.5 years to reach rut depth of 12.5 mm (0.5 in.).

The fatigue cracking index, a dimensionless parameter, is a function of fatigue parameters (K_1 and K_2), traffic loading, and layer thickness. It provides an indication of the amount of fatigue cracking over the service life of the pavement. Light cracking will occur between values 1.0 to 1.5; moderate cracking at 1.5 to 2.5; and severe surface cracking at 2.5 to 3.5. Results in Table 9 indicate that pavement constructed with Mix C (0 percent RRM) will experience severe cracking in 5 years of service, whereas pavement constructed with Mix B (15 percent RRM) will experience severe cracking in 9.5 years of service. On the other hand, pavement constructed with Mix A (25 percent RRM) had the lowest cracking index and will experience only moderate cracking over 17 years of service.

The present serviceability index (PSI) provides an indication of rideability of the pavement structure. As shown in Table 9, PSI values for pavement constructed with Mix C will reach the terminal serviceability index of 2.5 in 7 to 8 years of service. Pavement constructed with Mix B (15 percent RRM) will reach a PSI of 2.5 in 11 to 12 years of service. Pavement constructed with Mix A (25 percent RRM) will reach a PSI value of 2.5 after 16.5 years in service.

COMMERCIAL FEASIBILITY

For RRM to be commercially feasible on a large scale, certain criteria have to be met:

1. Satisfactory performance: as shown above, the addition of RRM to HMA can produce a mix that meets or exceeds perfor-

TABLE 9 Performance Modeling Results

	Time (years)				
	1	**5**	**10**	**15**	**20**
	Traffic $ESAL_{18}$ (x 1000)				
	48	240	480	720	960
Fatigue Cracking Index					
Mix A	0.11	0.69	1.38	2.37	3.17
Mix B	0.53	1.11	2.54	4.61	6.12
Mix C	0.54	2.97	6.55	10.03	13.01
Rut Depth, mm					
Mix A	4.10	8.10	11.90	14.90	18.08
Mix B	5.60	12.70	17.80	23.10	26.10
Mix C	7.60	15.50	22.90	28.70	36.09
Present Serviceability Index (PSI)					
Mix A	4.15	3.67	3.17	2.61	2.12
Mix B	4.01	3.29	2.76	2.01	1.69
Mix C	3.92	3.11	2.12	1.26	0.99

mance parameters set forth by the NSDOT&C. Mixes containing 15 percent and 25 percent of RRM will have performance properties superior to that of a comparable mix containing no RRM.

2. Economic considerations: in any commercial application, the cost of using RRM in HMA pavements should be comparable to that of conventional pavements. Extra costs should be offset by performance benefits of the new product. According to the results of other studies carried out, it is estimated that an initial set-up cost of \$500,000 is required to produce large quantities of shredded RRM (*4*). Production costs of shredded RRM are estimated at approximately \$8 to \$19 per ton (*4*). However, in batch mix operation, which resembles laboratory mixing of RRM mixes, the extra costs can be offset by savings in virgin asphalt cement of up to 50 percent. Extra costs can also be offset by a superior HMA pavement, which translates into lower maintenance costs in the long term.

3. Environmental impact: the use of RRM in a large-scale HMA pavement will have a positive impact on the environment. Recycling waste roofing material will ease the burden of disposing non-biodegradable roofing waste in landfills. Savings of up to 50 percent in virgin asphalt will reduce the demand on depleting resources of the petroleum industries. These advantages will, in part, offset the initial start-up and production costs mentioned above.

CONCLUSIONS

On the basis of the results of advanced testing and modeling of RRM mixes, the following conclusions can be made:

- Acceptable asphalt mixes containing up to 25 percent of RRM by weight can be produced at savings of approximately 3 percent asphalt cement compared to conventional HMA mixes.
- Permanent deformation and rut depth predication results strongly suggest that an increase in RRM content (up to 25 percent) reduces the rutting potential in pavements.
- The use of RRM in asphalt mixes improves fatigue lives of HMA pavements, especially at 25-percent RRM content.
- Although field verification is required, preliminary analysis using the VESYS model predicts that Mix A (25 percent RRM) will outperform the other two mixes, resulting in smaller rut depth, and less fatigue cracking. This in turn gives an improved serviceability index.
- Recycling waste roofing material in hot mix asphalt pavement is commercially feasible with existing technology. However, expensive start-up costs encountered in large-scale production may limit its usefulness.

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Evaluation of Rubber Modified Asphalt Demonstration Projects

JOHN EMERY

Eleven Ontario rubber modified asphalt demonstration projects were evaluated in terms of pavement performance and environmental impacts, including recyclability. On the basis of generally poor short-term performance of eight dry process (rubber modified asphalt concrete) projects, it appears that this method of crumb rubber modifier use should not be pursued unless there is considerable care in materials selection, mix design, and mix production and placement. The wet process (asphalt rubber) shows promise because it appears that asphalt rubber can enhance the durability of these asphalt mixes. Use of crumb rubber modifier in cold in-place recycling was not a technical success. A project with recycling of rubber modified asphalt concrete indicates no technical problems with recyclability. The economics (life-cycle cost) of the dry process are not favorable. If the incorporation of asphalt rubber does decrease maintenance costs or extend service life, there is a potential for savings through the wet process. Available asphalt technology, whether conventional or rubber modified, is capable of meeting environmental regulatory criteria. It is recognized that some technical issues require resolution to optimize rubber modified asphalt technology, and further work must be undertaken in such areas as long-term performance.

Eleven Ontario rubber modified asphalt demonstration projects were funded through the Ministry of Environment and Energy (MOEE) Tire Recycling Program or were completed by the Ministry of Transportation (MTO), between 1990 and 1992. An independent, comprehensive study of these demonstration projects was completed in 1993 (*1*). This study involved evaluation of the demonstration projects in terms of materials and pavements factors and environmental impacts, including recyclability; comparison of the findings to those of other jurisdictions, with identification of technical solutions to any issues; preparation of a summary of technically and cost-effective approaches to foster the use of rubber modified asphalt; and identification of policy and program impediments, with options for resolution. The materials and pavements factors and the economic analysis aspects of the study will be described in some detail, with an indication of the environmental impacts findings.

The 11 demonstration projects included 8 rubber modified asphalt concrete (RUMAC) projects in which the 1 to 3 percent recycled rubber from scrap tires, or crumb rubber modifier (CRM), which was introduced at the batch or drum plant (dry process), behaves essentially as rubber aggregate with some modification of the asphalt cement; 1 project with recycling of RUMAC placed the previous year (RRUMAC); 2 rubber modified cold in-place recycling (RUMCIP) projects in which the 3 percent CRM acts as unbound aggregate; and 1 rubber modified asphalt cement project in which the 0.5 percent fine CRM (7 percent CRM blended into asphalt cement, wet process) results in an asphalt rubber (AR) binder with reduced temperature susceptibility. The locations of these demonstration projects in southern Ontario are shown in Figure 1 (*1*).

It should be noted that there were Ontario AR trial sections (typically fine CRM dry process) placed as early as 1976 (*2,3*). Those sections placed by the Municipality of Metropolitan Toronto Transportation Department between 1977 and 1980 on major urban routes appear to be performing equivalent to, or somewhat better than, the overall pavement system (*4*).

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RUBBER-MODIFIED ASPHALT TECHNOLOGY

Various proprietary and generic technologies have evolved for the use of CRM in asphalt rubber (AR) binder and rubber modified asphalt concrete (RUMAC). From the late 1980s, the emphasis for this wet and dry process technology has been on its potential as a solution to the solid waste management problems of scrap tires. The material, process, technology, and product schematic for CRM use in AR and RUMAC is shown in Figure 2 (*5*).

There is some dry process interaction to modify the binder, as indicated by the dotted arrow lines, particularly for finer CRM or elevated mixing temperatures. The Ontario demonstration projects have mainly involved dry process generic RUMAC.

While the focus on CRM use in asphalt is currently RUMAC and AR, there is a wide range of other asphalt applications for recycled rubber from scrap tires, including hot-poured rubberized asphalt joint sealing compound, hot applied rubberized asphalt waterproofing membrane, hot applied rubberized mastic waterproofing membrane; protection board, paving "bricks," and recreational asphalt surfaces (running tracks, for instance) (*6*). Rubberized (CRM) asphalt joint sealant and waterproofing membrane are established and preferred materials technology in Ontario.

PERFORMANCE OF RUBBER-MODIFIED ASPHALT

Review of Available Information

In order to review the performance of AR and RUMAC pavements determined by various highway agencies, with emphasis on RUMAC, the available information was checked through DIALOG[R] (TRIS, EICOMPENDIX*PLUS, MATERIAL, and RAPRA), requests for information to major agencies and their contacts, in-house technical files, MTO technical files, and direct experience and contacts. This provided an excellent information base to consider along with the Ontario rubber modified asphalt demonstration projects.

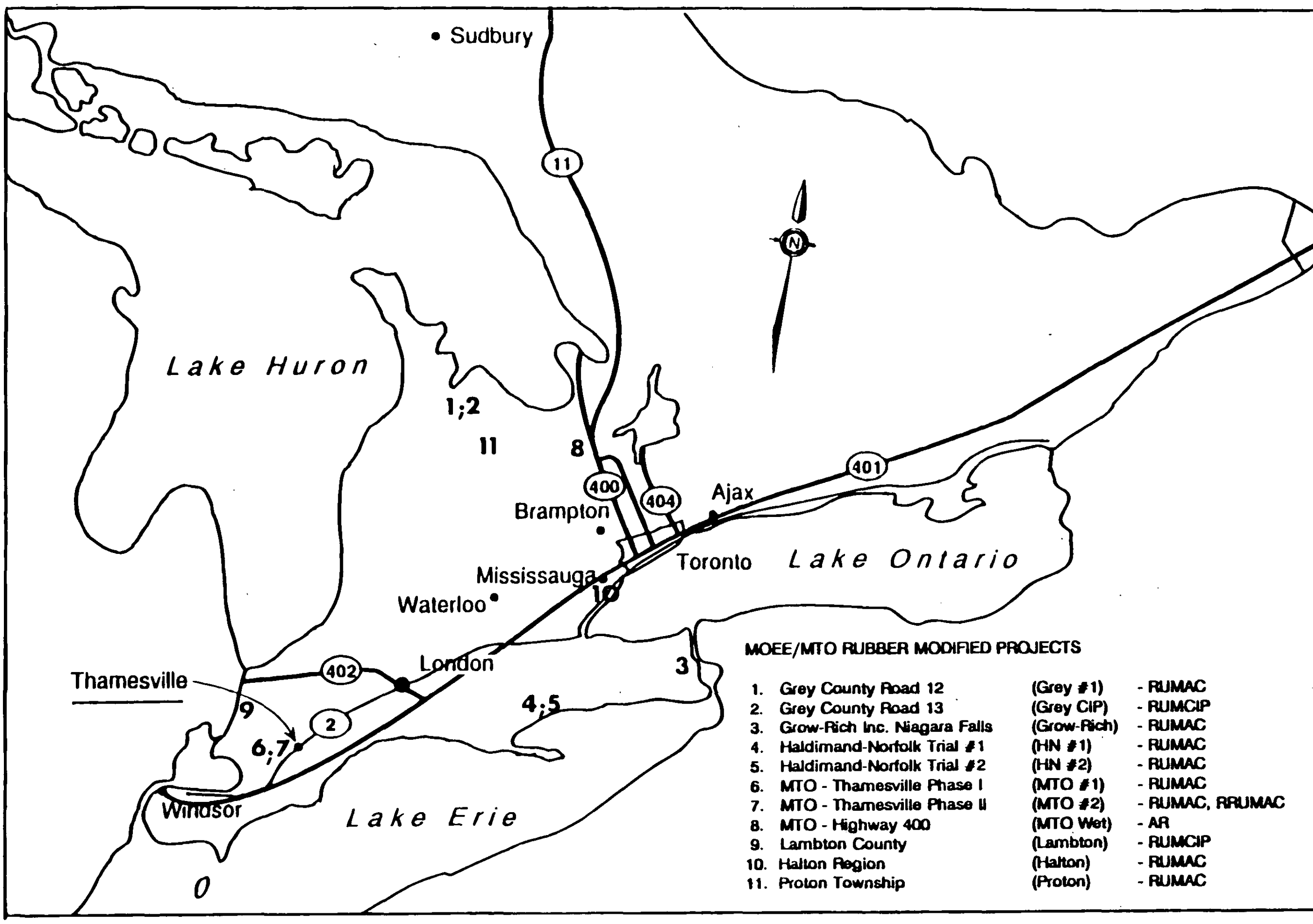

FIGURE 1 MOEE/MTO rubber modified asphalt demonstration project locations (*1*).

Much of the available technical information on CRM use up to 1993 has been reviewed and summarized in a Federal Highway Administration/Environmental Protection Agency (FHWA/EPA) study that is rather inconclusive (*7*). This FHWA/EPA study included the Ontario MTO Thamesville Phase 1 and Haldimand Norfolk Region Trial No. 1 rubber modified asphalt demonstration projects (Figure 1). Also, there is far more practical experience with AR as compared to RUMAC, particularly the generic RUMAC technology adopted for the Ontario demonstration projects.

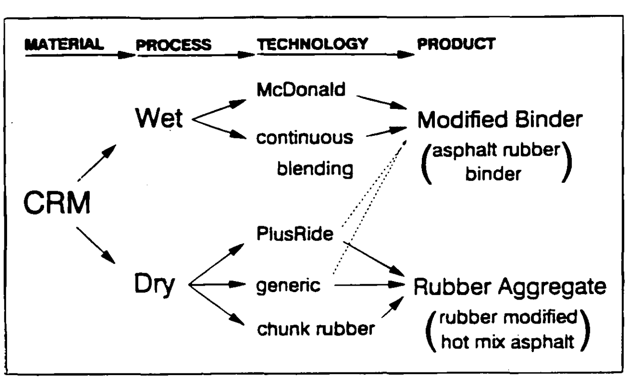

FIGURE 2 Use of recycled rubber from scrap tires (CRM) in asphalt (*5*).

The FHWA/EPA study conclusions for pavement performance and recyclability associated with CRM use in asphalt were (*7*)

1. When properly designed and constructed, there is no reliable evidence to show that pavements containing recycled rubber from scrap car tires will not perform adequately.
2. There is no reliable evidence that asphalt pavements containing recycled rubber cannot be recycled to substantially the same degree as conventional hot-mix asphalt (HMA) pavements.

Several factors need to be noted about the performance and recycling information from the FHWA/EPA study (*7*):

- The economics (life-cycle cost) of CRM use in AR and RUMAC were not considered.
- The differences between AR and RUMAC were not considered in detail.
- The issue of RUMAC reclaimed pavement (RUMAC-RAP) potential leachability was not addressed. However, there does not appear to be a problem in this regard from the available technical information (*8*).

TECHNICAL REVIEW OF ONTARIO DEMONSTRATION PROJECTS

Background and Performance Monitoring

The available reports on the 11 Ontario rubber modified asphalt demonstration projects were supplemented by experienced, independent pavement engineer (study team had no previous engineering involvement in the projects) site visits to each project, both to complete pavement condition evaluations and to review the information with the project engineers. A videotape was made covering the main site visit observations that also includes an overview (November 1989) of the Metropolitan Toronto Transportation Department AR trial sections placed between 1977 and 1980.

It should be noted for the technical review of the MOEE/MTO 1990 to 1992 CRM demonstration projects that, in common with most agencies, this was at a very early leading edge stage on the appropriate CRM asphalt technology learning curve for several reasons:

- Generic dry process RUMAC materials selection, mix design, production, placement, and testing requirements were, and are, still being developed, documented, and implemented [for instance, FHWA will be completing its Phase 2 CRM engineering study in 1999 (communication from B. H. Lord, McLean, Va., 1993)];
- While the "McDonald" technology for AR is well established, the generic wet process continuous blending technology for AR has only recently been established (in Florida, for instance) with the necessary blending equipment for terminal or hot-mix plant production now readily available;
- Use of CRM with cold in-place recycling (RUMCIP) appears to be unique to Ontario; and
- Ontario has been a leader in using cryogenic process recycled scrap tire CRM.

Clearly, it is important to learn from this early MOEE/MTO experience in order to incorporate appropriate CRM asphalt technology in future CRM asphalt paving projects. This was the focus of the technical review.

Technical and environmental project summary fact sheets were prepared for each rubber modified asphalt demonstration project and control section, including the pavement condition. Unfortunately, considerable information had been left out of the documentation for almost every project, such as details on mix production, smoothness of pavement, costing data, monitoring of tire-pavement noise levels, and monitoring of winter snow and ice development and its control. The rubber modified asphalt demonstration project profiles are summarized in terms of technology and application in Table 1.

An overall visual performance assessment of all the demonstration projects should be completed each spring and fall by a qualified pavement engineer. This will provide cost-effective continuing pavement performance information so that the current short-term conclusions and experience can be properly extended.

Demonstration Projects

The eight RUMAC (one also with RRUMAC, MTO No. 2), two RUMCIP and one AR demonstration projects summarized in Table 1 incorporated a wide range of CRM types (ambient and cryogenic), gradations (No. 4, 10, 20 and 80) and contents (1 to 3 percent). Ontario appears to be a leader in the use of cryogenic process CRM, and no apparent differences from ambient process CRM have been noted in the demonstration project reports. However, a direct comparison of cryogenic process CRM with ambient process CRM has not been made for the same RUMAC mix design, production, and placement.

The performance of surface course RUMAC incorporating fairly coarse CRM (No. 4 mesh) appears to be poor, with extensive raveling, considerable pop-outs and poor longitudinal and transverse joints (MTO No. 1 and HN No. 1, for instance). The RUMAC surface course performance appears to have been improved by incorporating No. 10 mesh CRM at a lower addition level of 1.5 percent (Grey No. 1 and HN No. 2, for instance), but this is again a short-term observation that may also reflect other factors such as better paving conditions. Regardless, it is important that the most appropriate CRM type, grading, composition and content, and CRM compatibility be established for typical RUMAC binder and surface courses. It must be recognized that space must be made available for the CRM in the asphalt mix matrix by volumetrically reproportioning the aggregates. This is a difficult proposition unless the CRM grading is similar to one of the fine aggregate gradings or the combined fine aggregate grading.

Equipment

There do not appear to have been any significant difficulties at batch or drum hot-mix plants with incorporating the CRM. However, care must be taken with poly-melt bags to ensure they are fully melted and mixed in. It is not clear how much additional dry mixing time, if any, is required for batch hot-mix plants to incorporate CRM, particularly finer CRM.

Production and Placement

It appears that the production of RUMAC and AR generally went smoothly, with the only significant placement problems associated with compaction of the somewhat tender RUMAC mixes, and the propensity for rubber-tired roller pick-up with the sticky mixes involved. These are somewhat experience-related problems, but the question of whether rubber-tired rollers should be used at all with RUMAC should be resolved. Practical production, placement, and compaction guidance for RUMAC should be developed in conjunction with contractor groups such as the Ontario Hot Mix Producers Association (OHMPA).

Rutting problems were associated with the placement of RUMCIP, and the efficacy of this CRM use must be critically assessed. It is not clear how the simple addition of CRM to cold in-place recycling can improve the process or the subsequent pavement performance. The use of cold in-place asphalt recycling is growing in Ontario as a method of mitigating reflection cracking (Grey County, for instance). It is important that the overall quality of cold in-place asphalt recycling be maintained to foster its use.

Quality Control and Quality Assurance

There were many quality assurance results to review and summarize in terms of RUMAC job mix formula (JMF) requirements and

TABLE 1 Summary of MOEE/MTO Rubber Modified Asphalt Demonstration Projects (2)

PROJECT AND LOCATION[a] IDENTIFICATION[b] /TIRES[c]	DATE	PROCESS[d] MIX TYPE	CRM PERCENT TYPE	PLANT FUEL CONTROL SYSTEM	'COMPARATIVE'[e] PAVEMENT PERFORMANCE	ENVIRONMENTAL MONITORING
Grey County Road 12 Grey #1 33,000	Nov./91 May/92	RUMAC HL 4	1.5 Ambient, No.10	Batch Oil Dry	Similar (Very Good) Poor Areas[e]	Yes
Grey County Road 13 Grey CIP 7,000	Aug./92	RUMCIP CIP	2 Cryogenic, No.4	CIP Process Not Applicable	Under Surface Course Similar (Excellent) Initial Problems	Not Applicable
Grow-Rich Inc. Niagara Falls Grow-Rich 16,000	Nov./92	RUMAC HL 8, HL 3	1.75, 1.75 Cryogenic, No.10, No.20	Batch Gas Dry	Covered by Pile Apparently Good No Control	Yes
Haldimand-Norfolk Trial #1 HN #1 78,000	Oct. to Nov./90	RUMAC HL 8, HL 3	3, 2 Ambient, No.4	Batch Oil Dry	Very Poor (Poor) Poor Areas	Yes
Haldimand-Norfolk Trial #2 HN #2 60,000	Aug./92	RUMAC HL 8, HL 3	3, 2 Cryogenic, No.10	Batch Oil Dry	Similar (Very Good)	Yes
MTO - Thamesville Phase I MTO #1 31,100	Oct./90	RUMAC HL 4	2 Ambient, No.4	Drum Oil Wet	Very Poor (Fair) Ravelling Pop-outs	Yes Comprehensive
MTO - Thamesville Phase II MTO #2 26,500	Oct./91	RUMAC RRUMAC HL 4	2 Ambient, No.10	Drum Oil Wet	Somewhat Poor to Similar (Very Good)	Yes Comprehensive
MTO - Highway 400 MTO Wet Not Known	July/90	AR HL 4, HL 1	0.5 (7 of AC) Ambient, No.80	Drum Gas Dry	Good (Excellent) 50 Percent Less Transverse Cracking	No
Lambton County Lambton 16,000	July/92	RUMCIP	3 Cryogenic, No.4	CIP Process Not Applicable	Under Surface Course (Excellent) No Control Initial Problems	Not Applicable
Halton Region Halton 37,000	Aug. to Sep./92	RUMAC HL 8, HL 3	1.5, 1.3 Ambient, No.10	Batch Gas Dry	(Excellent) No Control Little Traffic to Date	No
Proton Township Proton 7,000	May/92	RUMAC HL 4	1.0, 1.5 Ambient, No.10 (From Grey #1)	Batch Oil Dry	Similar (Very Good) 85/100 and 150/200 Used	No

Notes:
a. See Figure 1 for projects location map.
b. Abbreviations for project identification.
c. Passenger tire equivalents used.
d. RUMAC - rubber modified asphalt concrete (hot-mix asphalt)
RUMCIP - rubber modified cold in-place recycling
RRUMAC - recycled RUMAC
AR - asphalt rubber (rubber modified asphalt cement)
e. 'Comparative' is for September, 1993, overall assessment.
General condition is given in parentheses (Very Good), for instance. For example, for Haldimand-Norfolk Trial #1 (HN #1), the RUMAC HL 3 section's comparative performance to the control HL 3 section is very poor while the general condition of the RUMAC HL 3 section itself is poor.

RUMAC production range (minimum and maximum). There do not appear to be any significant problems in producing AR and RUMAC to the JMF requirements (asphalt cement content, aggregate gradation, and physical properties), but it is difficult with AR and RUMAC to accurately determine the asphalt cement content and complete conventional viscosity testing. Any necessary viscosity testing of AR or properly recovered asphalt cement from RUMAC could be completed using a rheometer, and at present this is not an issue for quality assurance testing. The monitoring of asphalt cement content in RUMAC requires pretesting to establish the amount of CRM that effectively becomes an asphalt cement component (some 10 to 20 percent) and must be adjusted for in conventional solvent-based extraction methods. The use of nuclear asphalt cement gauges has proved most promising (HN No. 2, for instance), provided proper calibration for the RUMAC is completed and the RUMAC is fairly consistent. Again, technical guidance on AR and RUMAC testing should be documented.

Problems and Resolution

Tender and sticky mix problems with the compaction of RUMAC were noted and have been a common problem with other agencies. These have been resolved through care during initial compaction to avoid pushing and shoving and the use of detergent-based release agents. The workability problem of RUMAC should also be considered at the mix design stage to ensure that adequate stability is being provided.

While experience with AR is limited (MTO wet), there should be no problems beyond placing and compacting a sticky mix, and contractors have had considerable experience with placing and compacting similar sticky polymer modified HMA.

No obvious problems were encountered during the processing stage of RUMCIP (Grey CIP and Lambton); however, in both cases extensive rutting and raveling of the cold in-place recycled material were experienced after being subjected to traffic. For the

Grey CIP this was resolved by reprocessing with additional emulsion and a remedial overlay; for Lambton this was resolved by reprocessing with additional emulsion. As indicated, the continuing use of RUMCIP must be critically reviewed. It should be noted that the RUMCIP is actually covered by a conventional HMA wearing surface (HL 4) so that direct observation of the RUMCIP is not possible.

Performance of Pavements

The relative short-term performance of the one wet process continuous blending AR project (MTO wet) has been good compared to the control section, with the AR section in excellent condition and 50 percent less transverse cracking. This favorable performance would generally be anticipated from polymer modified HMA experience. Regardless, wet process "continuous" terminal or site-blended AR should receive more attention.

The initial performance of the two RUMCIP projects was poor.

The relatively short-term performance of the RUMAC (surface course) projects has been quite mixed. But two performance groupings, compared to control sections, can be distinguished; they are

- Very poor: RUMAC surface course typically incorporating 2 percent No. 4 mesh CRM (HN No. 1, MTO No. 1 and to some extent MTO No. 2) and involving late paving season placement; and
- Similar: RUMAC surface course typically incorporating 1.5 percent No. 10 mesh CRM (Grey No. 1, HN No. 2, Halton and Proton) and involving reasonable paving season placement.

These Ontario demonstration project observations support the FHWA/EPA conclusion that properly designed and constructed RUMAC pavements should perform adequately (*7*). It should be noted that these are short-term performance considerations, and the key issue of long-term RUMAC performance remains to be addressed for the demonstration projects. The planned monitoring programs to 1996 for these RUMAC projects will not be adequate for assessing the comparative long-term performance, as conventional HMA surface course typically lasts for 15 years between resurfacings.

ECONOMIC ANALYSIS

Assessment Factors

To evaluate the economic benefits (or deficiencies) of Ontario RUMAC and AR use, a comprehensive economic analysis of conventional HMA, generic dry process RUMAC and AR asphalt pavements was completed, including sensitivity evaluations. The economic analysis focus was on RUMAC, as there is little current Ontario information available on wet process continuous blending AR, with the exception of the MTO 1990 Highway 400 AR (Rouse UltraFine™) test section (*9*). However, Rouse has provided comparative U.S. costing data for wet process continuous blending AR that appear to be appropriate in the Ontario context (communication from M. W. Rouse, Vicksburg, Miss., 1993).

When assessing the short and long-term economic impact of an innovative material such as generic dry process RUMAC, equipment requirements, serviceability and performance factors, societal concerns, and materials costs must be considered. Equipment requirements include any plant or equipment modifications necessary to meet environmental or production requirements, operating costs, maintenance costs, and effect (reduction in particular) on production. Serviceability and performance factors encompass such items as the relationship between any increment in costs and pavement performance, the effect of pavement salvage value on life-cycle costs, the quantity of recycled wastes to be used, and the recyclability of HMA containing wastes or by-products. Societal concerns include whether or not incentives should be provided for materials incorporating recycled wastes and if the waste generators should offer incentives to the highway construction industry to encourage use of the waste. Materials cost issues are somewhat more straightforward and include any increment in materials costs necessary to incorporate the waste into HMA, such as an increase in asphalt cement required, supply of CRM, additional aggregate requirements, and so forth. Each of these factors was evaluated as part of the overall economic assessment of generic dry process RUMAC pavements. This assessment also included consideration of the sensitivity of the analysis to various input parameters; for instance, what is the consequence of a lower CRM price on the overall economic analysis of RUMAC use?

Initial Costing Assumptions for RUMAC Economic Analysis

An initial cost comparison of conventional HMA and generic dry process RUMAC was completed as the first component of the economic analysis. It should be noted that there are minor additional costs (RUMAC mix design and quality assurance costs more than HMA, for instance) and cost savings (RUMAC has 2 to 4 percent more yield than HMA as lower bulk relative density, for instance), that tend to offset but they have not been considered.

From an equipment standpoint, generally no significant environmental or production modifications are involved. The CRM feed systems are quite conventional, particularly for hot-mix batch plants. However, the MOEE Certificate of Approval—Air for the HMA plant must be extended to cover the production of RUMAC. There are some additional staff and equipment requirements, including workers (at least two) to load the CRM into the plant, and a loader and operator to handle the pallets of CRM. The HMA production rate is not affected if a hot-mix drum plant is involved. If a hot-mix batch plant is used, some reduction in production can be experienced in order to ensure that the CRM is effectively distributed through the mix.

The materials requirements for RUMAC are relatively straightforward: for each 1 percent of CRM incorporated in RUMAC, the asphalt cement content increases by about 0.6 percent, based on practical experience. Therefore, if a conventional HL 3 mix has an asphalt cement content of 5.0 percent, a comparable HL 3 RUMAC mix with 1.5 percent CRM and the same aggregates (volumetrically adjusted fine aggregate proportions to accommodate CRM) will require an asphalt cement content of about 5.9 percent. The cost of the individual materials is therefore a major component of the economic analysis. The 1993 price of No. 10 mesh CRM was about $300/tonne plus about 10 percent delivery in the Greater Toronto Area (GTA), with asphalt cement costing about $150/tonne in 1993 plus about $10/tonne delivery in the GTA. The cost of processed CRM should decrease as RUMAC and AR use increases. Consequently, for the economic analysis of RUMAC, the cost implications of CRM at $330/tonne and $220/tonne have been evaluated. A $1/tonne of hot-mix increment for handling the CRM at the plant and 15 percent markup (industry standard practice) on the RUMAC have also been included.

Although incentives are currently not offered for the use of recycled scrap tire rubber (CRM) in RUMAC, some consideration has been given to the "value" of the potential savings in disposal costs owing to the use of scrap tires. Approximately 50 to 60 percent of a scrap tire is recovered through processing for CRM (the average tire mass is about 9 kg, and 5 kg of CRM can be produced from each tire). At the current disposal fee of about $100/tonne in the GTA, an HL 3 RUMAC mix containing 2 percent CRM represents about a $2/tonne of hot mix "saving" in disposal costs. Inasmuch as disposal costs have varied somewhat recently, dependent to some degree on the location (GTA is typically more expensive than other areas of Ontario), and societal factors must be considered, a range of incentives has been assumed for the economic analysis ($50/tonne, $100/tonne and $200/tonne), and of course the baseline case of no incentive.

The following parameters were included in the initial cost comparison of conventional HMA and RUMAC mixes:

Mix Types: HL 3 HMA (current price of about $34/t in GTA)
HL 8 HMA (current price of about $25/t in GTA)
HL 3 (1.5% CRM) RUMAC (requiring 0.9 percent additional AC)
HL 3 (2.0% CRM) RUMAC (requiring 1.2 percent additional AC)
HL 8 (2.0% CRM) RUMAC (requiring 1.2 percent additional AC)
HL 8 (3.0% CRM) RUMAC (requiring 1.8 percent additional AC)

Asphalt Cement Price: $ 150/t plus $ 10/t delivery in GTA

CRM Price: $300/t plus 10 percent delivery in GTA ($330/t total)
$200/t plus 10 percent delivery in GTA ($220/t total)

CRM Addition at Plant: $1/t of hot mix, plus 15 percent markup

Incentive for Scrap Tires:

	CRM Addition Rate		
Incentive	1.5%	2.0%	3.0%
None	0	0	0
$ 50/t of CRM	$ 0.75/t	$ 1.00/t	$ 1.50/t of CRM
$ 100/t of CRM	1.50/t	2.00/t	3.00/t of CRM
$ 200/t of CRM	3.00/t	4.00/t	6.00/t of CRM

(t = tonne in columns and tables)

An initial cost comparison of conventional HMA mixes and RUMAC mixes, based on the above parameters, is presented in Table 2. An example cost calculation for an HL 3 (1.5 percent CRM) RUMAC mix is as follows:

TABLE 2 Initial Cost Comparisons of Conventional and RUMAC Mixes (*1*)

MIX TYPE	PRICE/TONNE			
Incentive for Use of Scrap Tires (Per tonne of CRM)	0 (No Incentive)	$ 50.	$ 100.	$200.
HL 3 (Conventional)	34.00			
HL 3 (1½%) RUMAC (0.6% AC/1% CRM) CRM @ $ 300/t + $ 30. Del.	42.50	41.75	41.00	39.50
CRM @ $ 200/t + $ 20. Del.	40.60	39.85	39.10	37.60
HL 3 (2%) RUMAC (0.6% AC/1% CRM) CRM @ $ 300/T + $ 30. Del.	44.95	43.95	42.95	40.95
CRM @ $ 200/t + $ 20. Del.	42.42	41.42	40.42	38.42
HL 8 (Conventional)	25.00			
HL 8 (2%) RUMAC (0.6% AC/1% CRM) CRM @ $ 300/t + $ 30. Del.	34.92	33.92	32.92	30.92
CRM @ $ 200/t + $ 20. Del.	33.42	32.42	31.92	29.92
HL 8 (3%) RUMAC (0.6% AC/1% CRM) CRM @ $ 300/t + $ 30. Del.	40.92	39.42	37.92	34.92
CRM @ $ 200/t + $ 20. Del.	37.05	35.55	34.05	31.05

HL 3 Cost	=	$ 34.00	@ $ 220/t =	$ 3.30
CRM (1.5 %) @ $ 330/	=	4.95		
Additional AC (0.9 %) @ $ 160/t	=	1.44		
Labour Cost to Add CRM to Mix	=	1.00		
15% Mark-Up on Additional Items	=	1.11		0.86
		$ 42.50		$ 40.60
Less Incentive ($50/t rate)	=	0.75		0.75
Total:		$ 41.75		$ 39.85

Based on the initial cost comparison figures presented in Table 2, the conventional HMA mixes are less costly than the RUMAC mixes regardless of the incentive selected for disposal savings. At the most optimistic, RUMAC is still about 15 to 20 percent more costly than conventional HMA. If no incentive is attributed to RUMAC and CRM prices remain at their current relatively high level, the RUMAC mixes are approximately 32 to 39 percent higher in initial cost than equivalent conventional HMA mixes. These initial first-cost comparisons are similar to those suggested from recent U.S. RUMAC experience (Rouse 1993 Communication).

Life-Cycle Cost Analysis for RUMAC

To evaluate essentially equivalent pavement alternatives involving alternate materials, it is necessary to consider not only the initial cost of each alternative but also the total cost over its service life. The alternative having the lowest initial cost may not represent the most practical alternative once factors such as maintenance, rehabilitation, and inflation (and in contrast, the value of money invested today for future use, i.e., interest) are taken into account. The most effective method of measuring the cost-effectiveness of alternative designs is life-cycle cost analysis.

A number of life-cycle cost analysis approaches can be employed to evaluate construction materials. However, the most appropriate method appears to be that recommended by Kerr and Ryan (*10*), which has been used by the MTO and Asphalt Institute (AI). This method of measuring the cost-effectiveness of pavement alternatives equates present and future expenditures for each alternative, and associated maintenance and rehabilitation costs, by taking into account both inflation and interest rates over the life of the project. The concept of present value, or "discounting," is used to permit comparison of alternatives that require expenditure over an extended period of time, which allows the designer to consider the dual effects of interest rates (the time value of money) and inflation on project cost.

Despite the occasional relatively large apparent differences between interest and inflation rates, historically the discount rate, or the real difference between interest and inflation rates over an extended period of time (30 years), has been reported by Kerr and Ryan to be generally about 3 to 4 percent for privately financed projects. Recent provincially sponsored major transportation projects have assumed a long-term yield on Government of Canada bonds of 7.5 percent. Therefore, an interest rate of 7.5 percent has been assumed for the life-cycle costing of RUMAC pavements and an average inflation rate of 4 percent over the design life of the pavement, which results in a discount rate of 3.5 percent.

The service life of each alternative must also be taken into consideration for equivalent life-cycle cost comparisons. A conventional asphalt concrete pavement usually requires a major overlay after about 15 years to extend its functional service life. The timing for major maintenance and rehabilitation treatments required for each alternative must be taken into account; and the most appropriate service life must be selected for life-cycle cost analysis. For instance, the functional service life of an asphalt concrete pavement without major rehabilitation is about 25 years.

HMA and RUMAC Life-Cycle Cost Comparisons

Comparative life-cycle cost analyses of generic dry process RUMAC and conventional HMA mixes are presented in Table 3. To test the sensitivity of the life-cycle cost analyses to initial cost variations, life-cycle cost analyses were completed for three initial costs for each RUMAC type evaluated in Table 2. The highest initial cost (higher CRM price and no incentive) and the lowest initial cost (lower CRM price and maximum incentive for disposal savings) were used, as well as the average of the two. These figures were used to determine the cost per lane-km of pavement for each HMA and RUMAC mix, assuming a thickness of 40 mm for the surface course life-cycle cost evaluations and 50 mm for the binder course comparisons.

The life-cycle costing was conducted for a 30-year service life, with various performance assumptions and maintenance scenarios (both optimistic and pessimistic). For instance, each HL 3 RUMAC surfacing was life-cycle costed for replacement (milled off and replaced with the same mix type) at intervals of 5, 10, 15, and 20 years, respectively. Monitoring of the generic dry process RUMAC projects completed to date suggests that the mixes containing 1.5 percent CRM should last between 10 and 15 years, while 5 to 10 years is expected for some mixes containing 2 percent or more CRM when raveling will necessitate provision of an overlay.

The life-cycle costing of the conventional HMA and RUMAC surface course mixes also assumed that routine maintenance activities such as crack sealing, pothole filling, and so forth would be provided on a regularly scheduled basis. For example, crack sealing of an asphalt pavement would typically be completed within the first 2 years of construction or major rehabilitation (overlay) and again 5 years thereafter (properly applied, hot poured rubberized crack sealants last between 5 and 7 years). The same schedules and level of maintenance were assumed for both conventional and RUMAC mix types, with additional analyses completed assuming that the RUMAC mix would require half the maintenance.

Similar life-cycle cost analyses were completed for the conventional HMA and RUMAC binder course mix types. As these mixes would be covered with a surface course layer, the life-cycle costing does not include any routine maintenance operations, and only reflects the schedule for replacement (milling and replacement with the same mix type). Replacement schedules of 20, 25, and 30 years were costed for both conventional HMA and RUMAC binder course mixes.

The life-cycle cost analyses indicate that the lowest life-cycle costs are obtained for the conventional HL 3 surface course and HL 8 binder course mixes. Even when a relatively low CRM price and maximum incentive is assigned to the RUMAC mixes, the life-cycle costs of the RUMAC alternatives are still more than the conventional mixes. Even if the RUMAC mixes require half the level of maintenance, they are still more costly. Only when the RUMAC surface course design life is extended to 20 years, with low CRM pricing and maximum incentive, and compared to a 15-year design life for conventional HMA surface course is the life-cycle cost less than conventional. However, this is considered to represent a very optimistic scenario from the Ontario generic dry process RUMAC performance data to date.

TABLE 3 Life-Cycle Cost Analysis Summary for Conventional and RUMAC Mixes—30-Year Life (*1*)

MIX TYPE	LIFE CYCLE COST, dollars/lane-km			
	REPLACEMENT FREQUENCY			
SURFACE COURSE	5 YEARS	10 YEARS	15 YEARS	20 YEARS
HL 3 CONVENTIONAL			49753	
HL 3 (1½%) RUMAC				
CRM @ $ 300/t + 30 Del. + No incentive [42.50/t]	128703	74234	54632	41919
CRM @ $ 200/t + 20 Del. + maximum incentive [37.60/t]	121340	69971	51392	39438
Mean [40.05/t]	125024	72104	53013	40679
HL 3 (2%) RUMAC				
CRM @ $ 300/t + $ 30. Del. + no incentive [44.95/t]	132382	76364	56251	NA
CRM @ $ 200/t + $ 20. Del. + maximum incentive [38.42/t]	122572	70684	51934	NA
Mean [41.69/t]	127484	73528	54096	NA
BINDER COURSE	20 YEARS	25 YEARS	30 YEARS	
HL 8 CONVENTIONAL	26079	23756	21778	
HL 8 (2%) RUMAC				
CRM @ $ 300/t + $ 30. Del. + no incentive [34.92/t]	31266	28674	26467	
CRM @ $ 200/t + $ 20. Del. + maximum incentive [29.92/t]	28178	25742	23672	
Mean [32.42/t]	29720	27208	25077	
HL 8 (3%) RUMAC				
CRM @ 300/t + $ 30. Del. + no incentive [40.92/t]	34976	32192	29822	
CRM @ $ 200/t + $ 20. Del. + maximum incentive [31.05/t]	28872	26405	24304	
Mean [35.99/t]	31928	29302	27066	

Life-Cycle Cost Analysis for AR

The life-cycle cost analysis for wet process continuous blending AR was somewhat simplified compared to the detailed RUMAC analysis, in order to use U.S. data in the Ontario context. Several assumptions were made for the AR analysis: surface course (HL 3) use will be typical; Rouse technical and costing data are appropriate (about 20 percent increase in hot-mix materials cost, or $5/tonne of hot mix, attributable to AR incorporation, which also appears reasonable for Ontario) (Rouse 1993 communication); and no external incentives (tire buffings can be readily used in lieu of CRM). The life-cycle cost analysis was completed for several performance assumptions: HL 3 surface course service life is typically 15 years; AR HL 3 surface course service lives of 15 years and 20 years (Metropolitan Toronto Transportation Department apparently experiencing longer service life, for instance); same maintenance required for HL 3 and AR HL 3; and one-half the maintenance required for AR HL 3. The initial cost of the two mixes is taken as $34/tonne for conventional HL 3 and $39/tonne for AR HL 3.

Comparative life-cycle cost analyses for AR HL 3 and conventional HL 3 are presented in Table 4 for the various performance assumptions. If the incorporation of wet process continuous blending AR in HL 3 decreases the maintenance cost or extends the service life, as anticipated, then there is certainly a potential for considerable savings on a life-cycle cost basis. This is the position taken by U.S. proponents of AR use, who also emphasize the technical advantages of AR such as durability enhancement in open-graded hot mix, stress absorbing membrane (SAM), stress absorbing membrane interlayer (SAMI), and applications of a sulfur-asphalt module (Rouse 1993 communication).

SUMMARY OF ENVIRONMENTAL IMPACTS FINDINGS

The air emissions test results of the conventional HMA and RUMAC processes overlapped and exhibited a wide variability indicating that, except for the compound 4-methyl-2-pentanone

TABLE 4 Asphalt Rubber (AR) Life-Cycle Cost Analysis Summary—30-Year Life (*1*)

MIX TYPE	LIFE CYCLE COST, dollars/lane-km	
	REPLACEMENT FREQUENCY	
	15 YEARS	20 YEARS
HL 3 CONVENTIONAL Regular Maintenance	49753	
AR HL 3, Same Maintenance As HL 3	53167	40887
AR HL 3, Requiring half the maintenance as HL 3	47241	34509

(otherwise known as methyl isobutyl ketone or MIBK), there was no discernible difference between the emissions of the two processes. The wide variability is believed to be influenced by plant operation and maintenance practices and not because of differences due to the use of rubber modified asphalt. However, in five of the six demonstration projects where emissions testing was conducted, MIBK was emitted during RUMAC production and either was not detected or was emitted in orders of magnitude that were lower during conventional HMA production. In sufficient quantities, MIBK is a skin and mucous irritant and moderately toxic by inhalation but is not considered a carcinogen.

Occupational health exposures monitored for the two processes also measured overlapping levels that, in most instances, were at or below the detection limits for the compounds of interest. Worker exposures for the two processes of conventional HMA and RUMAC were similar.

The two issues of solid waste leachate and liquid effluent quality were not characterized for rubber modified asphalt. Although it is believed that these wastes are similar to conventional HMA, until this characterization is undertaken for rubber modified asphalt processes these issues will continue to be raised.

Available asphalt technology, whether conventional HMA, RUMAC, or AR, appears capable of meeting environmental regulatory agency criteria provided the process is designed, managed, and operated properly. This applies to air emissions, solid waste, liquid effluents, and occupational health.

NEED FOR RESEARCH AND DEVELOPMENT

Further research and development is needed in a number of areas of AR and RUMAC asphalt technology, pavement performance, and recyclability. They were identified during the review of the available technical information on CRM use in asphalt and the evaluation of the Ontario demonstration projects. These range from major research and development needs, such as research on the long-term performance of RUMAC pavements compared to conventional HMA pavements to relatively minor needs such as guidance to contractors on the best placement and compaction procedure(s) for RUMAC. Obviously, before the long-term performance of RUMAC pavements is considered through laboratory performance testing (SHRP protocols, for instance), accelerated pavement testing, and the monitoring of prototype pavement sections, RUMAC must be properly designed and constructed (*1*). The economic analysis of AR and RUMAC use (life-cycle costing) cannot be finalized until realistic long-term performance information is available. Technology transfer is a key element of this overall process that should also consider appropriate waste management incentives, if any, to establish technically sound, economically attractive AR and RUMAC use.

CONCLUDING COMMENTS

It is recognized that technical issues still require resolution to optimize the application of rubber modified asphalt technology and that further development will be undertaken in areas such as long-term performance. During this research, the environmental component should not be overlooked when appropriate.

Several asphalt technology recommendations have been made in the areas of CRM selection, generic dry process RUMAC placement, RUMAC long-term performance, life-cycle cost comparisons, wet process continuous terminal or site blending of AR, test procedures for AR and RUMAC, and influence of repeated RUMAC recycling. It is important that user agencies, contractors, and pavement consultants be kept informed, and involved, in the development of CRM use in asphalt paving in Ontario.

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Comparison of Carbon Black from Pyrolized Tires to Other Fillers as Asphalt Rheology Modifiers

DIDIER LESUEUR, DON L. DEKKER, AND JEAN-PASCAL PLANCHE

In order to dispose of old tires crumb rubber modified asphalts have been introduced. However, the research done on these products shows that they do not necessarily enhance the field applications. Since carbon black can be extracted from old tires, it might be a better way to recycle tires. This study presents carbon black (CB) from pyrolized tires as an asphalt modifier. CB was compared to aggregate fillers and ball clay, and mixed with asphalt to form mastics with similar weight contents (20 to 40 percent). If the high temperature (above 65°C) stiffening effect of the fillers is highly filler dependent, the low temperature rheological behavior is similar for all fillers. At temperatures below 65°C, the same rheological model can fit all experimental data points. However, a low temperature toughening effect was only found in the case of mineral fillers. This was attributed to the aggregate/asphalt interface. This interface is too weak in the case of CB and clay to toughen the asphalt at low temperatures and to prevent particle aggregation at high temperatures. As a result CB seems to be a good rheology modifier for binders already having a good low temperature behavior, such as polymer-modified asphalts.

In efforts to dispose of old tires, much research has been done to evaluate the performances of crumb rubber modified asphalts (*1,2*). Unfortunately, those studies show that crumb rubber does not necessarily improve the rheology of bitumens. An alternative to crumb rubber could be carbon black from pyrolized tires. When tires are heated to very high temperatures in the absence of oxygen, the polymers decompose (pyrolyze) into smaller organic molecules. The result is a combustible gas and a liquid fuel similar to diesel fuel. The carbon black originally added to stiffen the rubber remains behind as a granular char that can be easily ground to a fine powder, with 90+ percent passing the No. 325 mesh screen. Although this particle size is too large to enable the recycled carbon black to be reincorporated into new tires, it can be used effectively as a filler for asphalt concrete mixtures. One tire will yield approximately 7 to 8 lb of carbon black. Several companies are trying to commercialize the pyrolysis of tires to help dispose of used tires and to develop markets for the carbon black at a cost of 5 to 10 cents/lb.

The purpose of this study was to investigate the rheological behavior of carbon black modified asphalt. For a comparison purpose, two mineral fillers and one clay, also passing the No. 200 mesh (75 μm) screen, have been studied at similar fine contents (by weight). These fine contents were kept lower than 40 percent because of carbon black modified binders workability.

Because the Strategic Highway Research Program (SHRP) introduced new tools to evaluate the binders properties (*3*), the stiffening power of the fines was measured using the procedures defined by SHRP for binders. However, the purpose here was not to apply SHRP binder specifications to mastics, but rather to use SHRP procedures to compare the materials.

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EXPERIMENT

Samples

Two binders were used throughout this study: AC-10 and the same asphalt modified with reacted in situ Styrene-Butadiene copolymer (Styrelf MAC-10).

Four fillers passing the No. 200 sieve (75 μm) were used throughout this study:

- Carbon Black from pyrolized tires (from Jarrell Group);
- Ralston Quarries (from Colorado);
- Sievers Pits crushed (from Colorado); and
- Ball Clay (Super Seal No. 2 from Kentucky–Tennessee Clay Co.)

The mastics were made with 20 and 30 percent (mass) of each fine in both binders. In the case of ball clay, only the neat asphalt was used at three clay contents (20, 30, and 40 percent). The ball clay was included in this study because clays are a "bad-acting" filler. Ball clay is a kaolin clay and is highly plastic. ASTM 242-85 limits the plasticity index of fillers. It is used as a bonding agent in fired ceramics and as an emulsifying agent in coal tar emulsions. The binders were heated up to 185°C and the fillers were then added 3 to 5 g at a time, under constant stirring. Twenty minutes were needed to obtain each mastic.

Finally, 17 samples were studied (2 base binders and 15 mastics). Also, four mixes were made to evaluate the rutting resistance of the mastics, assuming the mastics were regular binders. The aggregates used for the mixes met the Michigan 4C specifications, which is close to a typical Indiana surface coarse aggregate. The binder content was 5.6 percent (mass) in each case and the air voids, 3 percent.

Testing

Specific fine testing was run on the fillers. This included:

- Particle size analysis (Figure 1), using a laser beam particle size analyzer, Microtrac II (Leeds and Northrup). Dispersion medium was water for all fillers except carbon black (2-propanol);

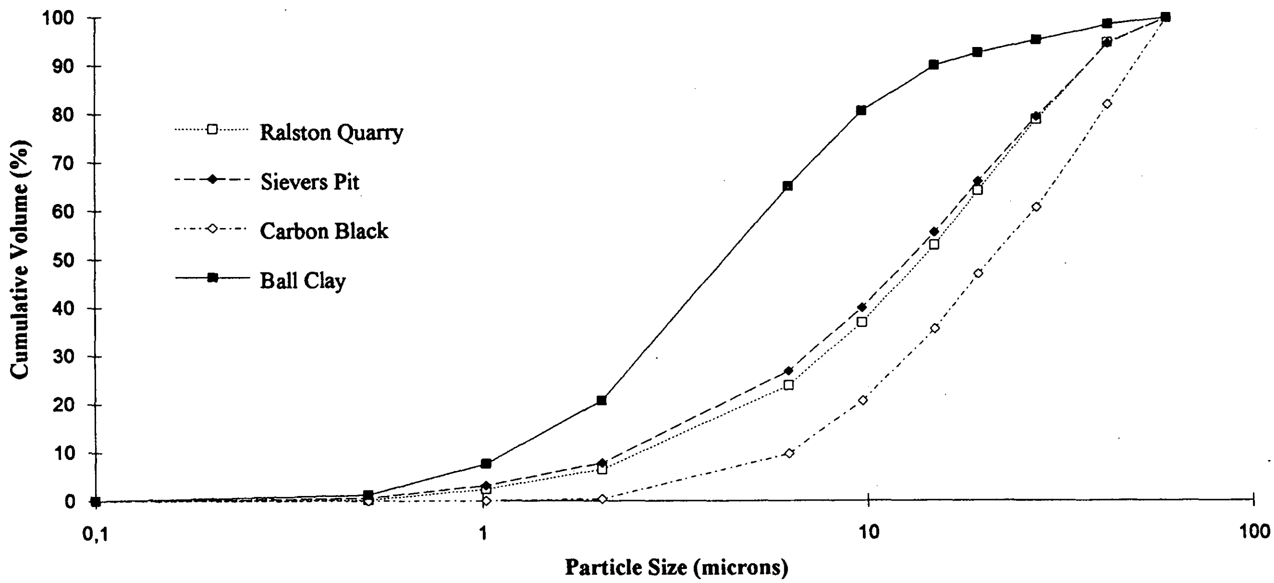

FIGURE 1 Particle size distribution of the fillers.

- Rigden air voids (compacity) measurements; and
- Specific gravity, using a Le Chatelier flask with kerosene.

Results of these tests are gathered in Table 1.

SHRP procedure [draft 8E, see (*3*)] was used to characterize the filled and unmodified binders. This included dynamic mechanical analysis (DMA—Rheometrics RDA II equipped with parallel plates) at high temperatures (around 60°C) on the unaged and rolling thin film oven test (RTFOT) aged materials; Brookfield viscosity on the unaged materials at 135°C; DMA at intermediate temperatures (around 20°C) on the pressure aging (PAV)-RTFOT residues; and bending beam rheometer (BBR—Cannon) and direct tension test (DTT) on the PAV-RTFOT aged materials at low temperatures (around −15°C).

Also, the Hamburg Wheel tracking test, described elsewhere (*4*), has been used to evaluate the rutting performance of four mixes.

RESULTS—SHRP APPROACH

Once again, the purpose of this study was not to apply SHRP binder specifications to the mastics. SHRP equipment was used as a tool to characterize the stiffening effect of the fines.

The temperatures where SHRP criteria (draft 8E) were met have been calculated by combining measurements at three temperatures (Figure 2 shows the temperatures where the criteria are met).

High Temperatures

As expected, $G^*/\sin\delta$ at a given temperature and frequency (10 rad/sec) increased with the fine content. The consequence of this stiffening effect on the unaged samples was that the temperature where the criterion (1 kPa) was met increased with the fine content (Figure 2). The increase caused by 20 percent carbon black in the base AC-10 was similar to that with 30 percent mineral fillers and to that with polymer modification. The same trend was found for the RTFOT aged samples (the criterion was then 2.2 kPa). However, in the case of RTFOT aged materials, two filled samples (MAC-10 + 20 % RQ and MAC-10 + 20 % SP) were found to have a lower modulus than the unaged mastic. This was probably caused by a decrease of the filler content owing to a loss of filler on the RTFOT jars.

To validate SHRP results, the rutting resistance of four samples was measured by using the Hamburg wheel tracking device. The tests were run at 50°C, under water. The samples were chosen for their similar limiting high temperature. They were the unmodified MAC-10 and the same MAC-10 filled with 20 percent (mass) of each fine (except ball clay). Figure 3 shows a similar rutting resistance for all filled samples, whereas the MAC-10 failed early. Also, the carbon black filled sample exhibited very little stripping compared to the mineral fillers. Carbon black is hydrophobic, and this may explain the low stripping in an underwater rutting test.

TABLE 1 Filler Properties

Filler	Supplier	Specific Gravity (g/cm3)	Fineness Modulus	Ridgen Air Voids (%)	m value from equation (2) @ 55°C	@ 65°C	@ 135°C
Ralston Quarry	Colorado	2.822	2.90	48.4	0.35	0.27	0.20
Sievers Pit	Colorado	2.717	2.83	45.9	0.36	0.19	0.20
Carbon Black	Jarrel Group	1.690	3.66	39.9	0.37	0.30	0.95
Ball Clay	KY-TN Clay Co.	2.960	5.59		0.49	0.34	0.67

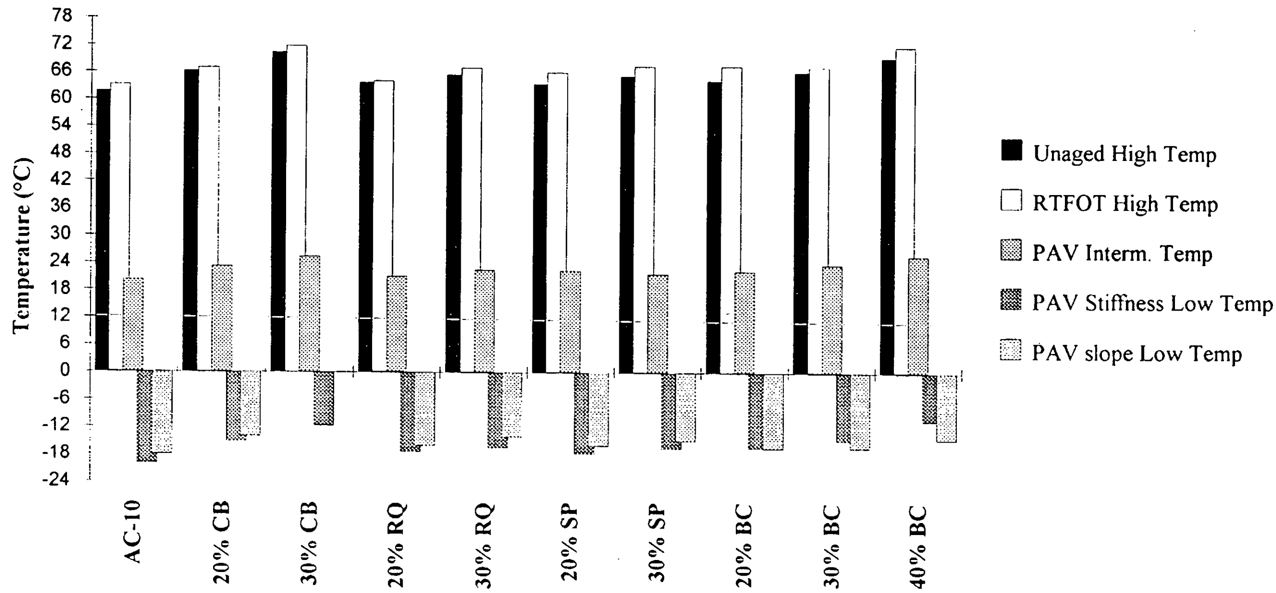

FIGURE 2*a* **Temperatures where SHRP criteria are met—base asphalt AC-10.**

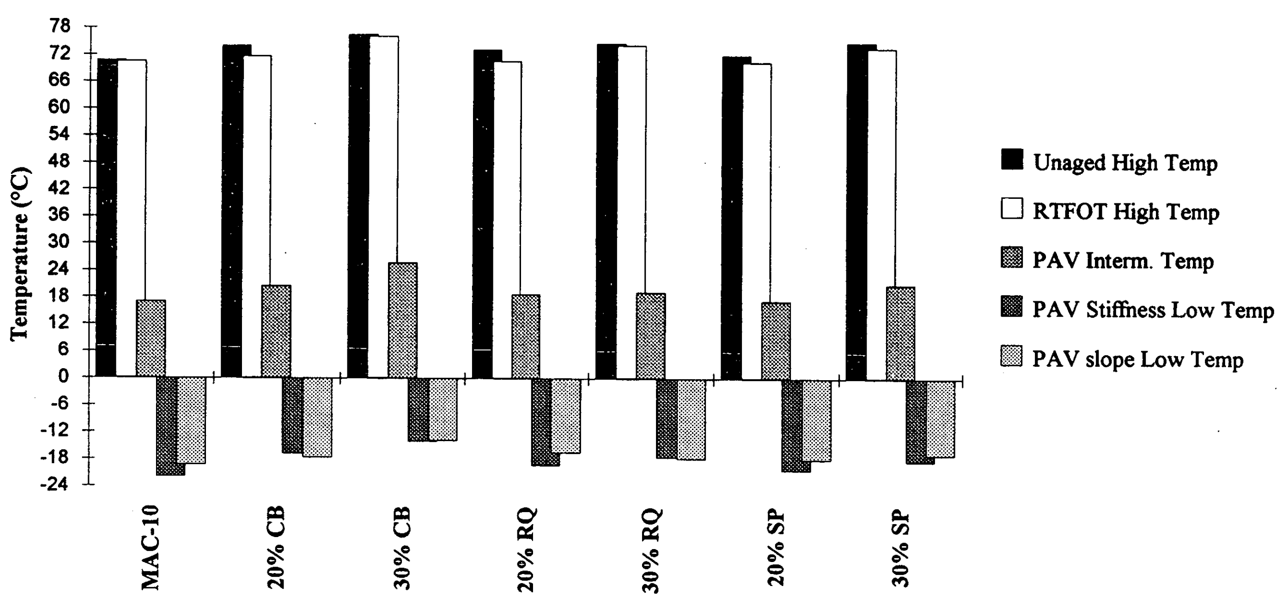

FIGURE 2*b* **Temperatures where SHRP criteria are met—base asphalt MAC-10.**

Finally, these rutting results on the mastics confirmed the predictions of SHRP criterion: the higher G*/sin δ at 10 rads/s on the unaged and RTFOT aged materials, the higher the rutting resistance, although this criterion is defined for binders only.

Intermediate Temperatures

At intermediate temperatures, the effect of the fines on the PAV-RTFOT aged samples was also to increase the modulus and, to a lesser extent, to lower the phase angle. The consequence was to increase G*x sin δ at given temperature and frequency, and then increase the temperature where the criterion was met (Figure 2). This would mean a lower fatigue resistance if SHRP binder specifications applied to mastics.

Unfortunately, no fatigue testing was run on the mixes to find out if SHRP criterion at intermediate temperatures could predict the fatigue resistance of the mastics.

Low Temperatures

At low temperatures, for the PAV-RTFOT aged samples, the stiffening effect was still obvious for all fines: the higher the fine content, the higher the stiffness and the lower the *m*-value. In terms of paving grades, MAC-10 with 20 percent carbon black passes the

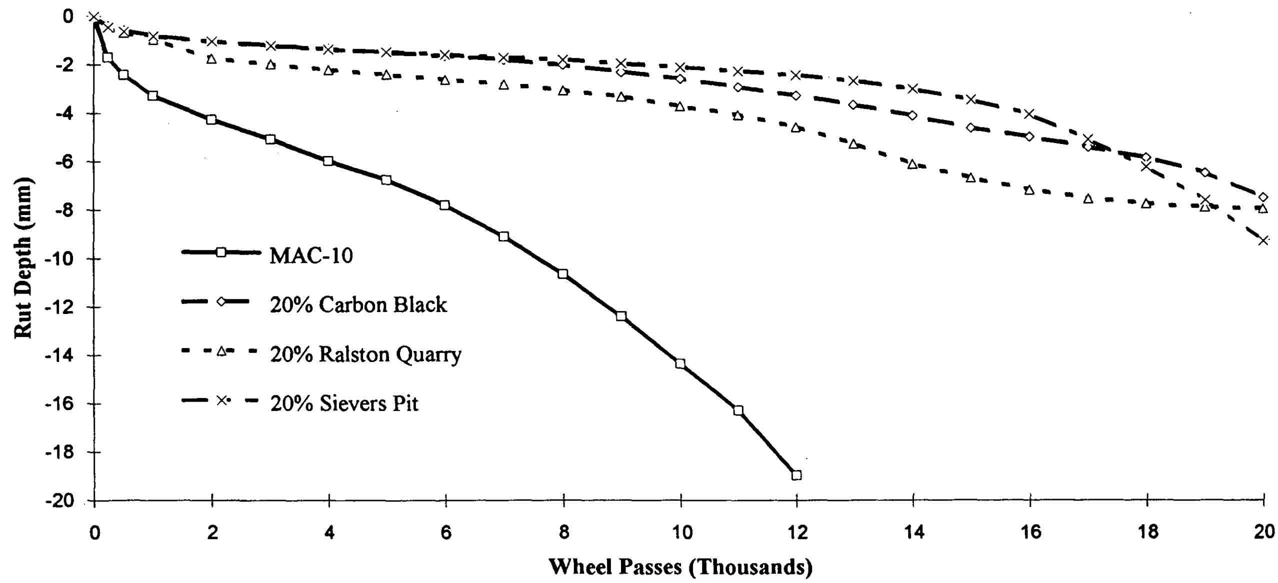

FIGURE 3 Rutting resistance of the mastics used as binders.

same low temperature grade as the unmodified base asphalt (AC-10). Polymer modification appears to be an easy solution to compensate for the stiffening effect of the fillers (Figure 2).

Due to lack of PAV-RTFOT aged materials, the temperature where the DTT specifications (1 percent strain at failure at 1 mm/mn) were met, could not be calculated. However, when looking at Table 2, a general trend can be observed. For carbon black modified samples, the higher the fine content, the lower the strain at failure. The increase of stiffness comes along with a lower failure resistance, which is the typical behavior of unmodified asphalt cements. Meanwhile, such an effect was not found in the case of mineral fillers. When DTTs were run on the unaged samples, a significantly higher strain to failure was found for filled samples with more than 20 percent filler (mass), although the stiffness is significantly higher (Figure 4). Therefore, the stiffening effect of mineral fillers comes along with a toughening effect at low temperatures.

Finally, at low temperatures, carbon black and mineral fillers exhibited the same stiffening effect, but a toughening effect was only found with the mineral fillers and not with carbon black and clay modified samples.

DISCUSSION

Rheology of Mastics

The effect of the fillers has been quantified using experimental results from the SHRP procedure.

These parameters were

- the Brookfield viscosity at 135°C;
- G*/sin δ at 55°C and 10 rad/sec;
- G*/sin δ at 65°C and 10 rad/sec;
- G* x sin δ at 20 and 25°C and 10 rad/sec; and
- the BBR creep stiffness at −15, −20, −25°C at 60 sec.

The zero-shear viscosity at 55°C and 65°C was first chosen to calculate the stiffening ratio. However, Figure 5 shows that there was no significant difference between the stiffening ratio calculated either with the zero-shear viscosity or with the SHRP criterion (G*/sin δ at 10 rad/s) at these temperatures. So stiffening ratios η_r were defined at all temperatures, as the given SHRP pa-

TABLE 2 Direct Tension Test Results on PAV Aged and Unaged Specimens and Corresponding Creep Stiffnesses at 60 S Loading Time

Aging	Temperature (°C)	Measurements	Units	Base Asphalt : AC-10 0% Filler	20% CB	30% CB	20% RQ	30% RQ	20% SP	30% SP	Base Asphalt : MAC-10 0% Filler	20% CB	30% CB	20% RQ
PAV	-6	Load at Failure	N	>50.0	71.3	83.1	71.3	102.0		92.8				
PAV	-6	Strain at Failure	%	>2.00	1.06	0.56	3.16	4.26		1.93				
PAV	-12	Load at Failure	N	49.8	74.6		72.7	81.1	94.1	88.5	83.7	79.7	76.8	121.2
PAV	-12	Strain at Failure	%	0.33	0.34		0.45	0.27	0.42	0.41	1.91	1.10	0.38	2.06
No	-15	BBR Stiffness	MPa	80			103	126	105	137				
No	-15	BBR Slope		0.49			0.48	0.47	0.48	0.48				
No	-18	Load at Failure	N	43.9			70.3	93.6	54.8	107.3				
No	-18	Strain at Failure	%	0.19			0.22	0.40	0.12	0.25				

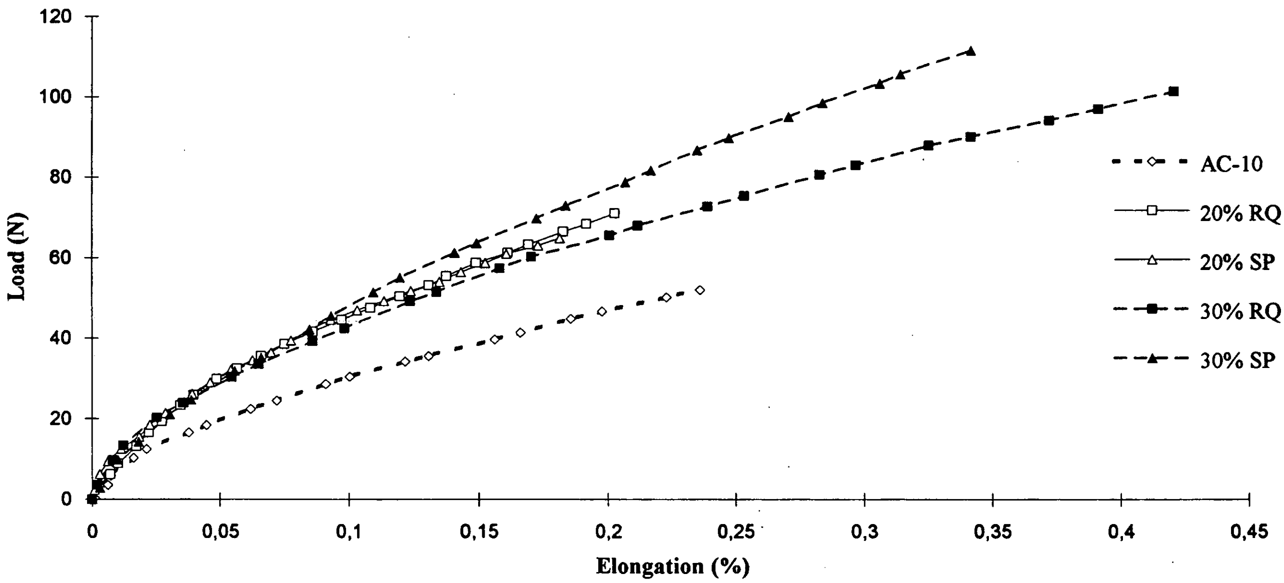

FIGURE 4 DTT results on the unaged samples at −18°C.

rameter of the mastic divided by the same SHRP parameter of the base binder.

Free Asphalt Theory

Mineral fillers in asphalt have been studied for years, and Anderson thoroughly overviewed the subject in 1982 (*5–10*).

Rigden found that the critical parameter to describe the stiffening effect of the mineral fillers was the solid phase volume fraction (*5*). Many studies confirmed this approach (*6–9*). Rigden's theory is based on the fixed asphalt concept. The fixed asphalt is the fraction of the asphalt needed to fill the air voids of the filler. Hence, a minimal amount of asphalt is required to obtain a fluid when blending fillers with asphalt. The liquid phase is called free asphalt. The free asphalt volume fraction is then directly related to the stiffening effect of the filler. In order to calculate the free asphalt volume fraction (ϕ_{fa}), Rigden developed a test (used in this study) to measure the percent air voids volume in the fillers (ϕ_{av}). Knowing the fine/asphalt ratio with respect to the volume $(F/A)_v$, the free asphalt volume fraction can be calculated as follows:

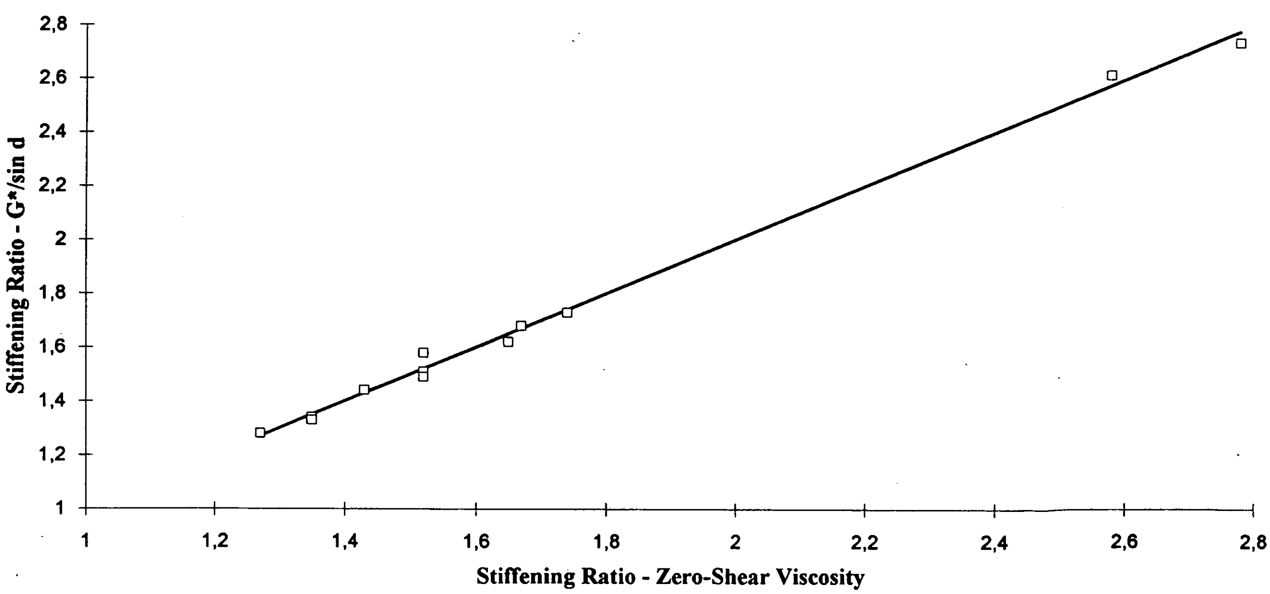

FIGURE 5 Stiffening ratios from G*/sinδ versus stiffening ratios from zero-shear viscosities.

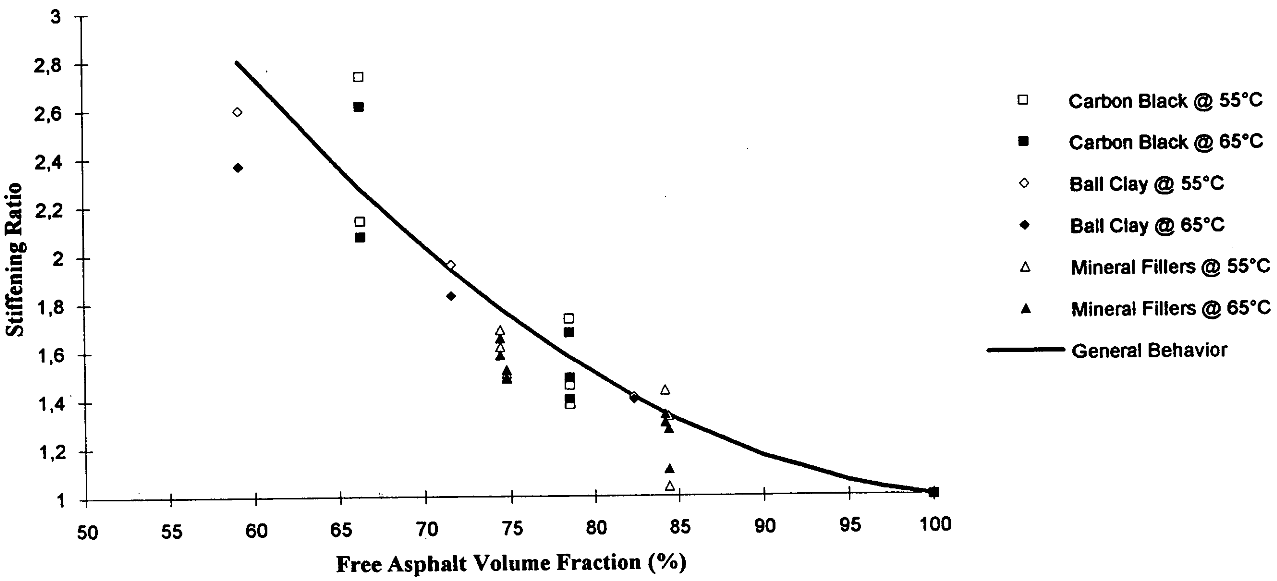

FIGURE 6*a* **Stiffening ratio versus free asphalt volume at 55°C and 65°C.**

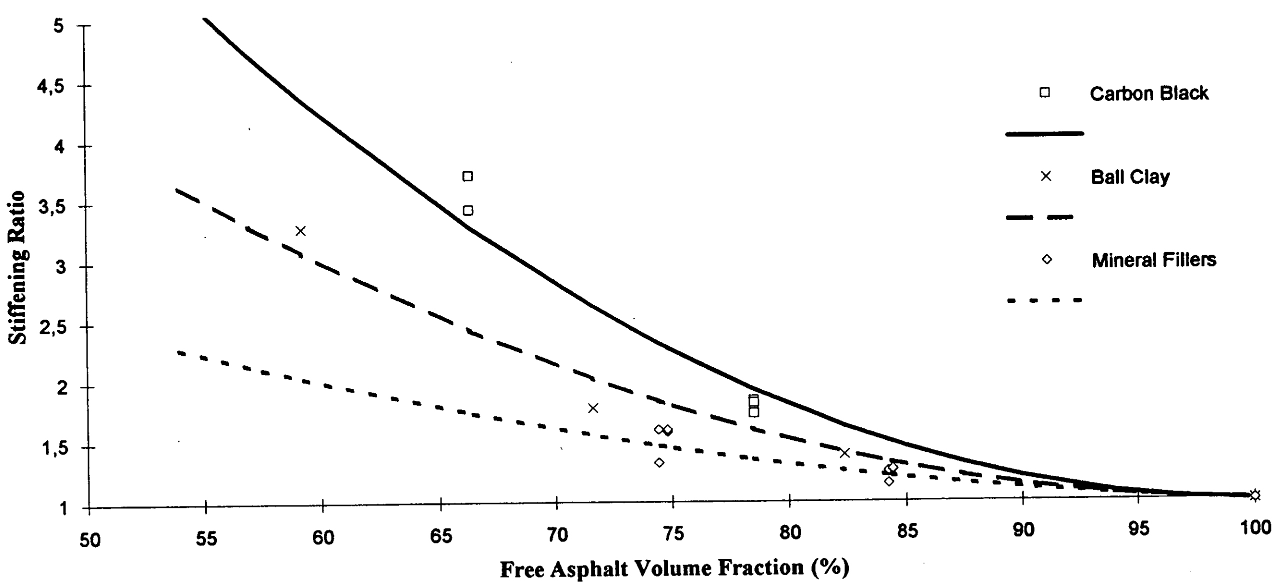

FIGURE 6*b* **Stiffening ratio versus free asphalt volume fraction at 135°C.**

$$\phi_{fa} = \{1 - (F/A)_v \times [\phi_{av}/(1-\phi_{av})]\}/[1 + (F/A)_v] \qquad (1)$$

Although Rigden's theory gives good predictions at a given temperature, it does not explain the temperature and shear rate dependency of filled asphalts.

The free asphalt fraction was calculated using equation (1) for all high temperature measurements. Then stiffening ratios were plotted versus free asphalt fraction at each temperature (Figure 6). Although one single curve can fit all experimental data points at 55°C and 65°C regardless of the filler, different curves per filler are needed at 135°C. At this temperature, neither carbon black nor ball clay behave like the mineral fillers and the free asphalt concept underestimates the stiffening effect of carbon black and ball clay.

The fines used in this study have similar Rigden air void indices (Table 1). The free asphalt fraction is rather similar from one sample to another, at a given $(F/A)_v$ ratio. So, looking at the $(F/A)_v$ ratio instead of the free asphalt fraction gives the same results. In terms of SHRP results, this effect is highlighted by the fact that 20 percent (mass) carbon black is equivalent to 30 percent of each mineral filler.

Finally, the free asphalt concept was in good agreement with the stiffening effect of the fines at 55°C and 65°C, but underestimated the stiffening effect of carbon black and ball clay at 135°C.

Suspension Approach

Heukelom and Wijga developed a semi-empirical approach to describe the increase of viscosity of dispersions, including fillers in asphalt (*10*). They proposed the following equation to calculate the viscosity of the blends, for $\eta_r < 100$:

$$\eta_r = [1 - 1.28 \times (1 + m) \times \phi]^{-2} \quad (2)$$

where η_r is the relative viscosity (the viscosity of mastic divided by the viscosity of the base asphalt), m, a parameter describing the virtual increase of concentration due to a poor peptization of the particles and/or the nonsphericity of these particles, and ϕ, the volume fraction of the filler. This latter parameter being related to the fine/asphalt ratio:

$$(F/A)_v = 1 / [(1/\phi) - 1] \quad (3)$$

Although the peptization is certainly temperature-dependent, Heukelom and Wijga did not look at the variation of m versus temperature.

Linear plots of $(\eta_r)^2$ versus ϕ, at a given temperature, led to the m-values, using the method described by Heukelom. This consists in evaluating ϕ_{max}, the volume concentration where $1/\sqrt{\eta_r}$ equals 0. Therefore ϕ_{max} represents the compacity of the filler and correlates well with Rigden Air Voids Index for most fillers (*10*). Knowing ϕ_{max}, the following equation was applied to calculate m:

$$m = (0.78/\phi_{max}) - 1 \quad (4)$$

The constant (0.78) corresponds to an ideal case where the particles are fully peptized spheres with distributed sizes, as measured by Heukelom for a regular asphalt emulsion.

So, m values in equation (4) were calculated from experimental results. Figure 7 shows the validity of Heukelom's law in the 55°C to 135°C temperature range. Table 1 shows the variation of m versus temperature for each fine. Whereas the m-values for carbon black, ball clay, and both mineral fillers were rather similar to 55°C and 65°C, results at 135°C were clearly different. Although the trend for the mineral fillers was a slight decrease, m for carbon black and ball clay increased at 135°C. This increase of the m-value means that the peptization of carbon black and ball clay is better at 65°C than at 135°C. A reasonable explanation is that carbon black and ball clay aggregate easily at high temperatures, creating clusters between the solid particles. Some asphalt is then entrapped inside the aggregates so that the apparent volume fraction of the filler increases. Thus the viscosity and, consequently, m increase. This behavior also explains why Rigden Air Voids indices do not correlate with ϕ_{max} in the case of carbon black at 135°C. However, this result does not occur at 65°C because the viscosity of the asphalt is still high and prevents this aggregation from happening during the measurements.

Finally, Heukelom's law was found to give a good prediction of the rheological behavior of filled samples at high temperatures.

Heukelom's law was then generalized for every SHRP measurement, including $G^* \times \sin\delta$ after PAV at intermediate temperatures and creep stiffness at low temperatures. The same model was used ($m = 0.238$) to fit all experimental results, for all aging levels (unaged, RTFOT, and PAV aged samples). Finally, the model used to fit the experimental results reduces to a Maron & Pierce law (*11*):

$$\eta_r = [1 - \phi/A]^{-2} \quad (5)$$

where A = 0.630 (which is rather similar to the compacity as measured by the Rigden Air Void index).

The comparison of the samples at any aging level can be made assuming the stiffening effect is not aging dependent. In other

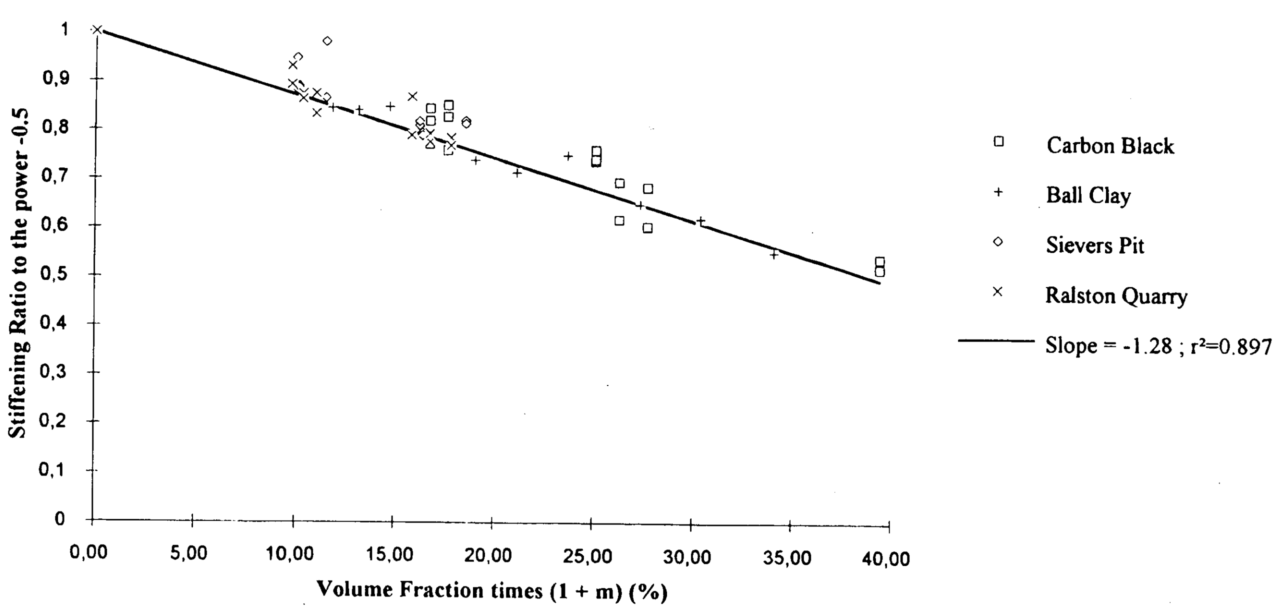

FIGURE 7 Experimental verification of Heukelom's equation (Equation 2).

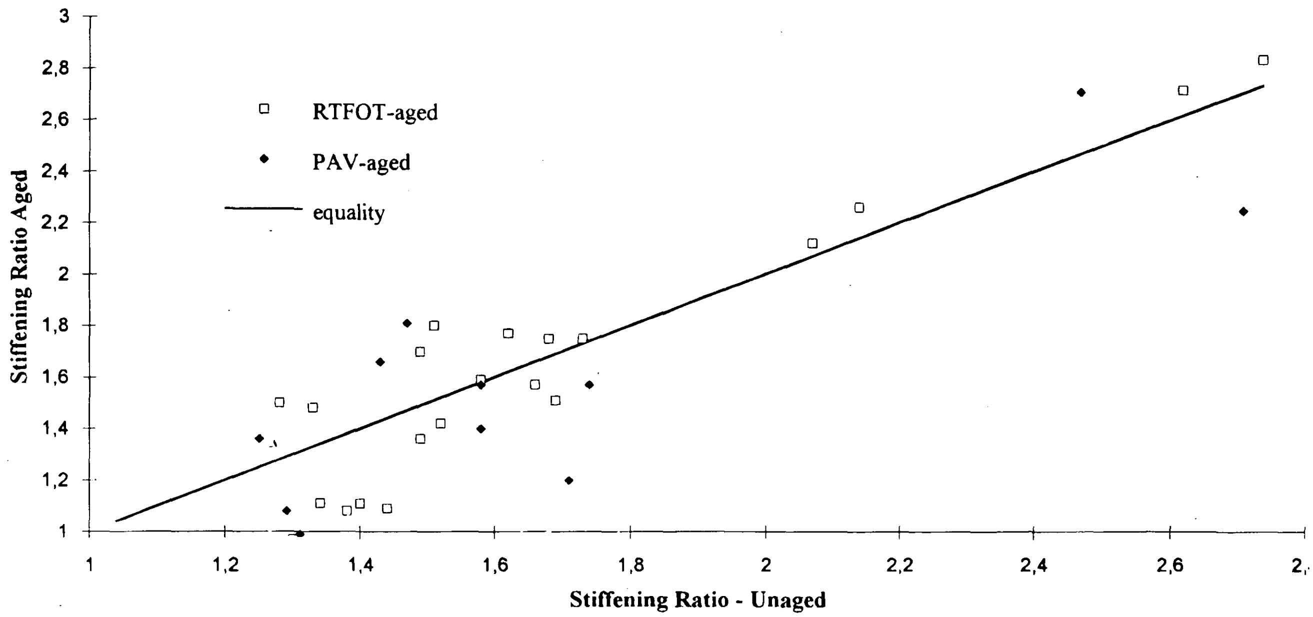

FIGURE 8 Stiffening ratios of the aged mastics versus stiffening ratios of the unaged mastics.

words, it means that the aging of the asphalt is not modified by the fillers. Figure 8 shows that the stiffening effect is fairly constant versus aging.

The correlation between calculated and experimental results (using Heukelom's law) was found to be highly significant, $r^2 = 0.95$ (Figure 9). The average error was found to be 12.7 percent and 80 percent of the data are within 20 percent error. The data points out of this range were either high temperatures measurements (for carbon black and ball clay) or aged samples. In the former case, the difference can be explained by an underestimated m-value (cf. previous section). In the latter case, a reasonable explanation is that the actual filler content was lower than the expected one (due to experimental problems such as pouring the RTFOT aged samples from the RTFOT jars).

More generally, all SHRP measurements can be estimated using equation (5). The only parameter needed is the volume fraction of the fines. As long as the asphalt matrix is too viscous to allow particle aggregation (i.e., at temperatures below 65°C), predictions from equation (5) are accurate (20 percent average error).

However, at temperatures above 65°C, the behavior of the fines becomes highly filler dependent. Particle aggregation becomes more probable, and another parameter, describing the peptization degree of the suspension, is needed to give a better prediction of their stiffening effect.

Toughening Effect of Mineral Fillers

A better fracture resistance was found with the mineral fillers but not with carbon black nor with ball clay. This toughening effect of mineral fillers resembles the enhanced fracture properties of materials reinforced with rigid particles (glass beads in epoxy resins, for example). Strong interactions between the fillers and the asphalt are needed for such a behavior to occur. In the case of ball clay and carbon black, the asphalt/particle interactions are weak, as shown by the high temperature aggregation of the dispersed particles. This explains why no toughening effect was found with them.

Also, fracture mechanics measurements made by Salam and Monismith (*12*) showed that the finer the aggregate, the higher the fracture toughness. These results were in good agreement with those presented herein.

This toughening effect could explain the better thermal cracking resistance of stone mastic asphalt (SMA), which is usually attributed to the high binder content (around 15 percent) (*13*). Since the thermal crack propagates through the brittle asphalt matrix, no big difference on the thermal cracking resistance should appear when considering varying binder contents. So the better thermal cracking resistance of SMA is more likely due to the high fine content. Furthermore, the asphalt layer surrounding the rocks is thick in SMA mixes. This toughening effect should be more pronounced in this particular case.

However, this qualitative result needs to be confirmed by specific experiments to quantify and understand the toughening effect of the fillers.

CONCLUSIONS

The effect of the fines passing the No. 200 mesh sieve has a considerable effect on the performance of a bituminous mix. It has been shown that both the amount of fines and their characteristics are important factors. Some of the important fine properties are compacity, density, and chemical composition. For example, if 20 percent (weight) carbon black is added to a mix, some of the naturally occurring fines must be removed or the mix will probably become too stiff. Also, the same amount of carbon black, ball clay, and natural fines will not produce the same results.

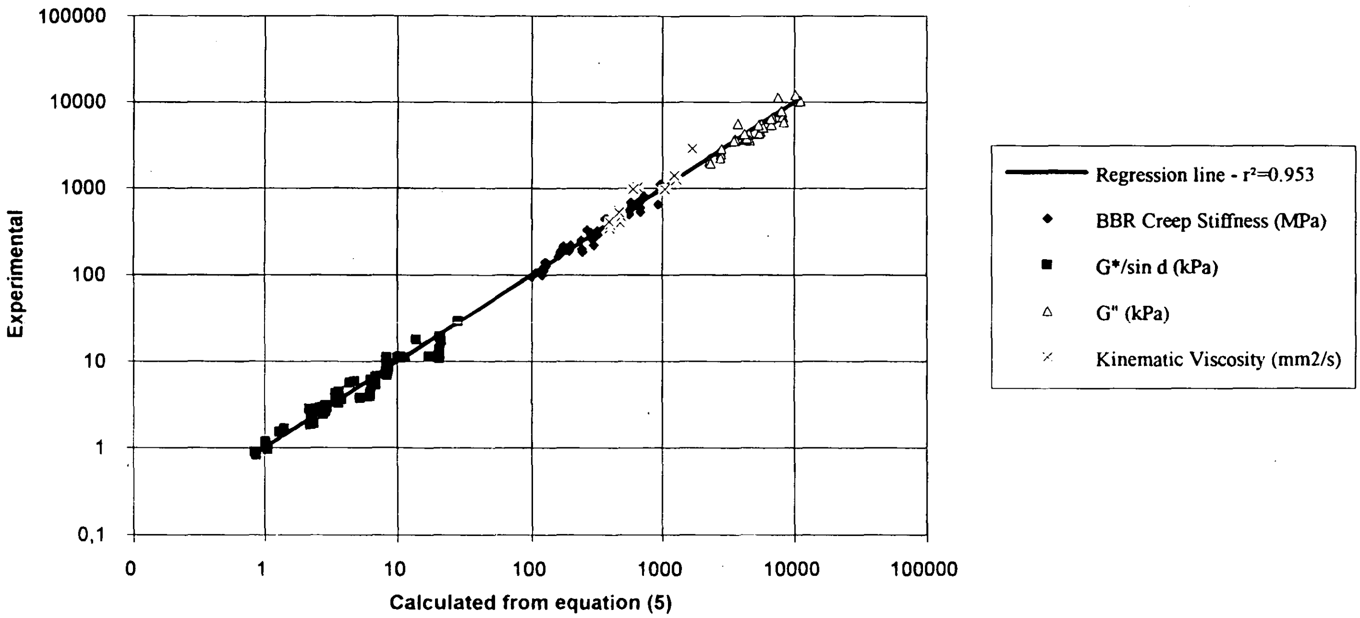

FIGURE 9 Experimental SHRP data points versus calculated ones.

This study showed that

- The new procedures introduced by SHRP can be used to investigate the effect of fillers.
- The rheology of mastics is mainly governed by the volume content of the fillers at temperatures less than 65°C, but other parameters such as the *m*-value in Heukelom's law are needed to predict the high temperature behavior of the filled asphalts. As a result, a Maron and Pierce law with A = 0.630 was found to be reliable in order to predict any SHRP measurement, knowing the volume content of the filler and the value of this measurement for the base asphalt. However, those results are only proposed in case of low filler concentrations (up to 40 percent weight) and stiffening ratios less than 100.
- Carbon black from pyrolized tires, and mineral fillers as well, were found to greatly improve the rutting resistance of the binders.
- A toughening effect occurred with mineral fillers at low temperatures, but not with carbon black nor with ball clay. Consequently, carbon black will be an efficient rheology modifier for a base asphalt having good low temperature properties, such as polymer-modified asphalts. Also, it would not be worth substituting carbon black for mineral fillers in specific applications (such as SMA) where a good thermal cracking resistance is necessary.
- Fracture mechanics experiments are needed to understand and quantify the low temperature toughening effect of the fines.

The binder and some of the fines combine to produce a mastic that binds the asphalt concrete together. The percentages of fines in this mastic may be as high as 40 percent. Therefore, the characteristics of these fines and the effect they have on this mastic should be carefully studied before a mix is placed on the road. The type and amount of No. 200 fines must be carefully selected to produce asphalt concrete with the desired properties.

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Modification of Asphalt Binders and Asphalt Concrete Mixes with Crumb and Chemically Devulcanized Waste Rubber

GEOFFREY R. MORRISON, ROB VAN DER STEL, AND SIMON A.M. HESP

For this study of the modification of asphalt binders with crumb and devulcanized waste rubber, rubber digestion aids were investigated for their ability to devulcanize waste tire crumbs. It was found that none of the commercially available chemical agents were able to totally devulcanize the crumb rubber. However, high yields of devulcanization were obtained with chlorothiophenols. The devulcanized and crumb rubbers were added to asphalt binders, and their low temperature performance was studied. It was found that the fracture toughness of the devulcanized rubber system was higher than that for both the control asphalts and a commercially available oxidized rubber system. Toughening is mainly due to crack pinning and shear yielding mechanisms for crumb rubber systems. However, only enhanced shear yielding mechanisms are responsible for the increase in toughness found in devulcanized rubber systems. Further work on the asphalt-aggregate mix should tell us if the improved performance of devulcanized binder systems will give enhanced pavement performance.

The development of rubber modified pavements has been a concern of highway materials engineers for nearly 30 years. The initial objective of research in this area was to improve the low and high temperature performance of roads. More recently, however, the focus has shifted due to environmental and political concerns about the growing scrap tire disposal problem. This has ultimately led to Section 1038 of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, which requires the use of scrap rubber tire in federally funded highway projects starting in 1995 (*1*).

BACKGROUND

Crumb Rubber Systems

C.H. McDonald, a materials engineer for the city of Phoenix, Arizona, developed the idea to use waste rubber as a modifier for asphalt binders in 1966. Since then, several variations of the original process have been widely used. The two main methods of modification are the dry process and the wet process. Each has certain limitations and advantages (*2–4*).

The dry process uses crumb rubber as part of the aggregate in a hot mix pavement. This process uses about 0.5 to 5 percent by weight of the aggregate of crumb rubber as small as 30 mesh, which usually fills a gap in the aggregate gradation. Results from Alaska and Sweden have suggested the use of different aggregate gradation bands depending on traffic volume. No special equipment is required for this process but it is limited to hot mix applications.

The wet process is a procedure in which 18 to 26 wt percent ground rubber tire, usually in the range of 8 to 20 mesh, is added to the asphalt binder and mixed at elevated temperatures (175°C to 220°C) for 1 to 2 hr. The rubber particles become swollen in the asphalt's oily phase forming a gel, to which some aromatic oil is often added to aid in processing. This process requires at least 20 percent more asphalt than a normal hot mix, which may be the main reason these pavements can show improved properties. The higher cost of the wet process has limited its use to stress absorbing membranes, to crack and joint sealing, and to a lesser extent to thin asphalt concrete overlays.

Rouse Rubber Industries has recently developed a process using very fine 80 mesh crumb rubber particles, for which they claim a much smaller cost increase than for both the conventional wet and dry processes (*5*). This process has recently been used in paving trials in both Ontario (*6*) and Florida (*7*). At present it is too early to comment about the performance of these test sections.

An extension of the wet process, by way of reactive compatibilization of the asphalt-crumb rubber interface, has shown that small amounts of crumb rubber reacted with a coupling agent can give low temperature fracture performance as well as, or better than, more costly binders with a high rubber loading (*8*). The added effect on the low temperature fracture toughness of interfacial grafting a low molecular weight polybutadiene onto the crumb rubber is shown in Figure 1.

Devulcanized Systems

By devulcanizing rubber waste in asphalt, a homogeneous binder may be produced that can be stored at high temperatures for prolonged periods of time without the complications associated with slow sedimentation of the rubber that occurs when crumbs are added.

The idea of adding waste rubber that will dissolve in the asphalt to produce a homogeneous single phase system was first patented by Scott in 1972 (*9*). The process involved oxidative degradation of rubber in a liquid hydrocarbon that was then added to the asphalt binder. Similar patents that use solvent oils and soluble rubbers to produce gel-like compositions in which the crumb rubber is swollen with oil were assigned to Winters and McDonald (*10*), and Nielsen and Bagley (*11*). Subsequent inventions use blends of elastomeric rubbers and crumb rubber in asphalt (*12,13*). It is important to note that unless specific devulcanization agents or very high temperatures and long devulcanization times are used, as in Scott's 1972 invention, the crumb rubber particles will only swell in the added oils. Recently, Liang and Woodhams (*14*) have used a similar approach to that of Scott. They also add the crumb rubber to a hot aromatic oil and subsequently shear it until the rubber dissolves; the oil-rubber mixture is then diluted with asphalt.

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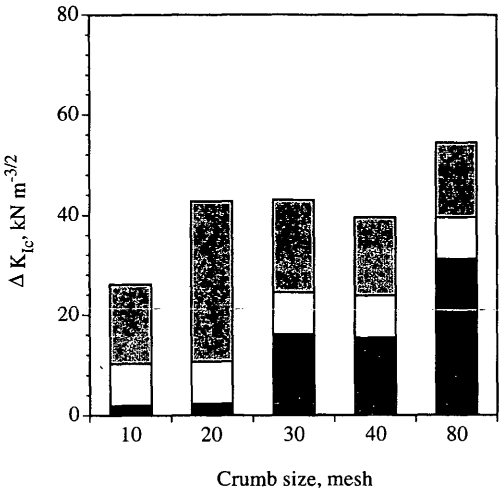


(Black–effect of crumb rubber, white–effect of additive and grey–effect of interfacial modication)

FIGURE 1 Effect of interfacial modification on the low temperature fracture toughness of crumb rubber modified asphalt binder (*8*).

In general, devulcanization of rubber may be accomplished by using either an oxidative or a reductive path. The cross-linked molecular structure of a typical rubber is shown schematically in Figure 2 (*15*). The relative distribution of sulfur among the various structures will depend on the structure of the rubber, the temperature at which vulcanization took place, the composition of the vulcanization mixture, and the age of the rubber tire. High vulcanization accelerator concentrations tend to lead to more monosulfide cross-links and pendant accelerator complexes, whereas high temperature and old age tend to lead to more backbiting with an accompanying loss of functionality. In recycled rubber tire, however, it is reasonable to assume that most sulfur remains in polysulfide linkages.

Oxidative Devulcanization

Oxidation of disulfide linkages may be accomplished by reaction with peroxides or peracids, singlet oxygen, and by ozonolysis. A general mechanistic scheme for these oxidation reactions is shown in Figure 3 (*16*). Obviously, the oxidation of sulfur linkages is a complex process, and a number of products can be formed. Moreover, the oxidative agents used can also attack the SBR polymer backbone, eventually reducing the molecular weight of the devulcanized rubber.

Following the approaches taken by Schmidt et al. (*17*), Biegenzein (*18*), and Hoehr et al. (*19*) for the compatibilization of polymers in asphalt, Duong and Boisvert (*20*) have recently developed a process for the oxidative devulcanization of crumb rubber in asphalt binders. The procedure involves air blowing under pressure a mixture of crumb rubber and asphalt between 220°C and 260°C. The rubber is devulcanized by an electrophilic oxidation reaction with singlet oxygen, which cleaves sulfur-sulfur bonds to give a sulfonic acid (see Figure 3).

However, it is well known that air blowing of asphalt at temperatures well above 200°C may result in the oxidation of the asphalt itself, producing a harder and more brittle binder with poor low temperature and long-term fatigue performance (*21*). All oxidative methods of in situ rubber devulcanization are rather undesirable because of the associated oxidative degradation of the asphalt.

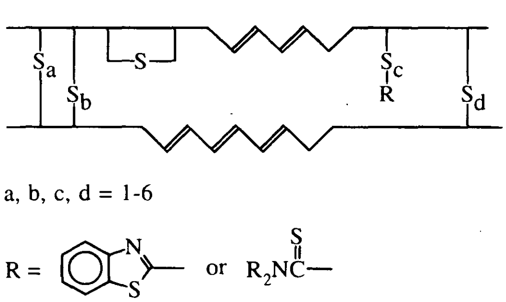


FIGURE 2 Illustrative structure for cross-linked rubber (*15*).

Reductive Devulcanization

Reduction of disulfides to thiols by homolytic cleavage results from heating in the presence of reducing agents such as tetralin, decalin, secondary amines, or chlorothiophenols. Three examples for the reductive cleavage of sulfide linkages are given in Figure 4 (*16*). The thiols formed can subsequently be oxidized and react with labile groups within the asphalt resulting in an asphalt graft onto the rubber molecule.

In this study a number of oxidizing and reducing agents have been investigated for their efficiency in devulcanizing crumb rubber systems in a high boiling solvent (1,2,4-trichlorobenzene). The devulcanized rubber was subsequently added to asphalt to investigate the low and high temperature performance of the binders.

EXPERIMENTAL PROCEDURES

Materials

The 1,2,4-trichlorobenzene used for the devulcanization of crumb rubber, the tetrahydrofuran used in the gel permeation chromato-

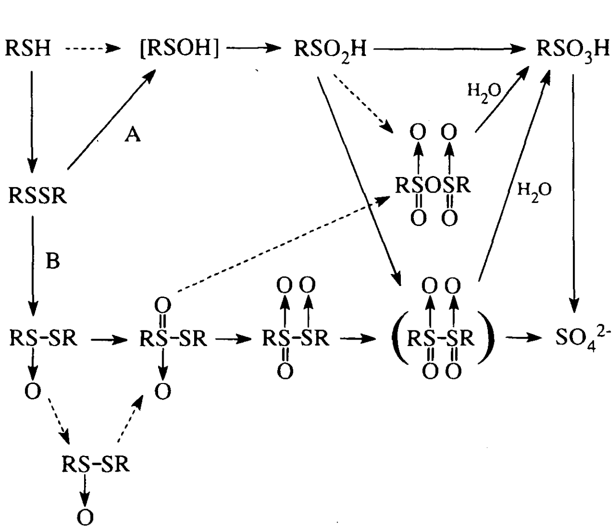


FIGURE 3 General disulfide oxidation scheme (*16*).

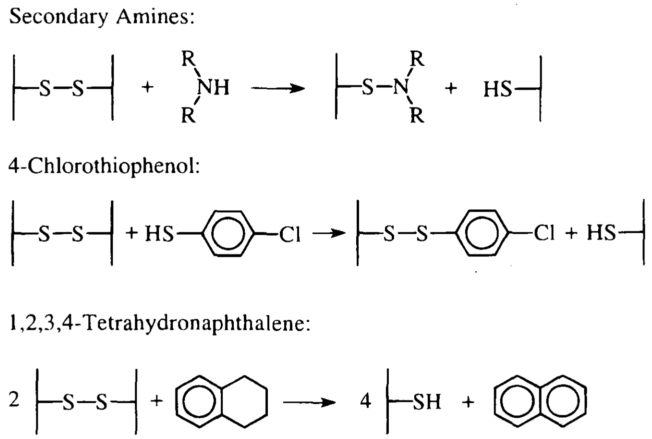


FIGURE 4 Reductive pathways for the devulcanization of cross-linked rubber (*16*).

graph, and the 4-chlorothiophenol were obtained from the Aldrich Chemical Company of Milwaukee, Wisconsin.

The Leegen 6130 devulcanization aid contained approximately 50 percent sulfonated petroleum products and was obtained from R.T. Vanderbilt of Norwalk, Connecticut. The Renacit 7 devulcanization aid contained approximately 50 percent pentachlorothiophenol as the active agent and was obtained from Miles of Akron, Ohio. The Plastone devulcanization aid contained a complex mixture of fatty acids, rosin acids, and isopropanolamines and was obtained from Harwick of Akron, Ohio. The MOTS 1 devulcanization aid contained between 45 and 55 wt percent thiuram disulfide as the active agent and was obtained from the American Cyanamid Company of Wayne, New Jersey.

The cryogenically ground rubber tire was obtained from Recovery Technologies of Mississauga, Ontario. The binder containing 10 wt percent oxidatively devulcanized rubber was obtained from Bitumar of Montreal, Quebec. The liquid polybutadiene, Ricon R-134, was obtained from Ricon Resins of Grand Junction, Colorado. The elemental sulfur used for the interfacial modification of the crumb rubber-asphalt systems was obtained from the Aldrich Chemical Company.

The 85-100 and 150-200 penetration grade asphalts were obtained from the Oakville, Ontario, refinery of Petro-Canada. Both binders were prepared from Bow River crude.

Devulcanization of Crumb Rubber

Devulcanization of crumb rubber was carried out by refluxing the rubber crumbs in an organic solvent at high temperature in the presence of a chemical devulcanizing agent. In a typical reaction, 300 ml of 1,2,4-trichlorobenzene (boiling point 214°C) containing 20 grams of No. 10 mesh cryogenically ground rubber and 10 percent devulcanizing agent by weight of the rubber were charged to a 1000 mL roundbottom flask and refluxed with stirring for 2 hr. After cooling, the mixture was filtered through a 100 mesh stainless steel screen and the residue dried under 28 in. of mercury vacuum at 50°C for at least 8 hr. The mass of the dry residue on the screen was subtracted from the original mass of the rubber to determine the yield of the reaction. The effectiveness of several different devulcanizing agents was examined.

Devulcanized rubber was recovered by removing most of the trichlorobenzene in a rotary evaporator under vacuum, and then removing any remaining solvent in a vacuum oven under standard conditions. Molecular weight analysis of the devulcanized rubber was conducted using a Waters Associates gel permeation chromatograph with tetrahydrofuran as the eluent solvent.

Binder Preparation

Crumb rubber modified binders were prepared by adding 10 wt percent 20 mesh cryogenic ground car tire and 2 wt percent Ricon R-134 liquid polybutadiene (LPBD) of molecular weight 12,000 g mole^{-1} to a 150-200 penetration grade asphalt. A surface compatibilization reaction was facilitated by adding a further 1 wt percent sulfur to one of the crumb rubber/LPBD-asphalt mixtures. Samples with and without sulfur were mixed for 2 hr at 150°C to 170°C by using a high shear Polytron mixer manufactured by Brinkmann Industries of Mississauga, Ontario.

The devulcanized rubber modified binders were made by adding 10 wt percent devulcanized rubber, obtained from the chemical devulcanization studies, under high shear mixing to a 150-200 penetration grade asphalt.

Asphalt Binder Tests

Dynamic Mechanical Testing

Samples were heated until liquid at 130°C to 140°C, poured into a combined melts and solids (CMS) test fixture, and then allowed to cool to room temperature. Testing was done on a Rheometrics Dynamic Analyzer, RDA II. The CMS fixture consists of a cup 42 mm in diameter and a bilevel plate that has a serrated surface 8 mm in diameter concentric with and projecting from a plate 25 mm in diameter. Curves for $G''/\sin\delta$ (i.e. $1/J''$) were generated using values of G'' and δ measured from temperature sweeps between 52°C and 70°C at 10 rad/sec, taking a measurement every 6°C after a soak time of 2 min.

Fracture Toughness

The elastic fracture toughness, K_{Ic}, was determined according to procedures based on ASTM E399-90 (*22*). A notched asphalt beam was cast in a silicone mold, cooled, removed, and left for a minimum of 12 hr at −20°C before being tested under temperature-controlled conditions in an MTS Sintech 2/G test frame. The beams were loaded in a three point bend configuration at 0.01 mm/sec until failure occurred. The failure load was then used to calculate the linear elastic fracture toughness. For a more detailed description of the experimental procedure, the reader is referred to a previous paper by the Lee and Hesp (*23*).

Strain Rate Sensitivity

Sample preparation was the same as that used for determining the fracture toughness except that for these tests the temperature was raised to −10°C. Various crosshead speeds were used to break the asphalt beams. Results are expressed as failure energy, which was

TABLE 1 Aggregate Gradation Used for All Mix Tests

Sieve Size	Percent Retained Limestone Coarse Aggregate	Limestone Screenings	Natural Sand
1	-	-	-
7/8	-	-	-
3/4	-	-	-
5/8	-	-	-
1/2	1.3	-	-
3/8	30.9	-	-
4	91.9	7.6	1.7
8	95.4	35.8	7.5
16	96.7	59.7	14.0
30	97.5	75.9	30.0
50	98.0	85.6	62.2
100	98.4	91.7	93.1
200	98.7	94.7	98.4

taken as the energy under the stress-strain curve at various crosshead speeds. Plotting these data on a semilogarithmic graph shows a sharp transition from brittle to ductile behavior. The strain rate at which this transition occurs gives an indication of the yield characteristics of the binder.

Asphalt Concrete Mix Preparation

The aggregate used in the asphalt concrete mixes consisted of three components, namely, 45 percent coarse limestone aggregate, 15 percent limestone screenings, and 40 percent natural sand. The gradation for the aggregate mix is given in Table 1. The aggregate was heated for a minimum of 12 hr at a temperature of 175°C. The asphalt was heated to 160°C just prior to mixing with the aggregate. All mixes contained 6 wt percent binder on the aggregate, which previously had been determined to be the optimum for this aggregate gradation. Compacting was done by using a Rainhart Series 142 Gyratory Compactor set for 250 gyrations at an angle of 1°. The compaction temperature was 160°C for all mixes.

Asphalt Concrete Testing

Tensile Failure Strains

Small rectangular blocks measuring approximately 30 mm × 35 mm × 60 mm were cut from the large briquettes and epoxied between 20-mm-thick steel plates. The glue was allowed to harden for a minimum of 24 hr after which the samples were cooled in a freezer at the desired test temperature for a minimum of 12 hr.

The mix samples were tested in tension using a crosshead speed of 0.0025 mm/sec with a computer interfaced linear variable displacement transducer (LVDT) accurate to ± 5 μm mounted on each side of the test specimen to measure the extension. The actual strain rate within the specimens was found to be comparable to that of typical shrinkage rates in pavements exposed to rapidly dropping temperatures during autumn and winter months. Failure strain values reported were calculated from the measured displacements, each value being the average of five tests, the value for each test being the average of the two sides.

RESULTS AND DISCUSSION

Devulcanization

Crumb rubber samples were devulcanized to determine the effectiveness of various devulcanization chemicals.

The devulcanization residues were analyzed by gel permeation chromatography to determine the molecular weight of the devulcanized rubbers and to see whether there is any difference between the effect of different devulcanizing agents. The data showed that there is not a great deal of difference between different devulcanizing chemicals. This may be explained by the selectivity of the devulcanization reaction for cleaving sulfide bonds only, leaving the backbone of the styrene-butadiene rubber intact. However, the different devulcanization agents give rather different yields. Table 2 gives the devulcanization yields for the reagents used in this study. These results show that the two thiophenol reducing agents are clearly the most effective on an equal weight basis. The order of effectiveness may change though when these products are compared on an equal cost basis.

TABLE 2 Solvent Devulcanization Yields

Additive	Active Ingredient (@ 10 wt % on the GRT)	Devulcanization Yield, %
None	-	0.0
4-Chlorothiophenol	-	84.1
MOTS No.1 Accelerator	50% Thiuram Disulfides	70.2
Renacit 7	Pentachlorothiophenol	85.3
Plastone	Complex Mixture	39.2
Leegen 6130	Sulfonated Products	75.7

It is important to note that none of the commercial devulcanizing agents are fully effective under these reaction conditions. It is therefore appropriate that further research into finding a more effective and economical process for the devulcanization of waste rubber be continued.

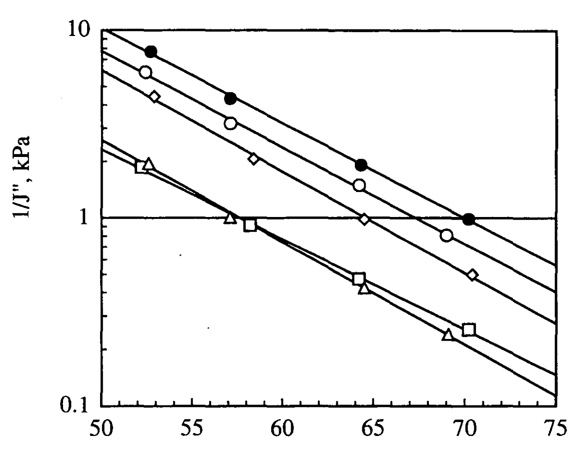
Binder Performance

High Temperature Performance

The high temperature performance for a number of control and rubber modified binders was determined according to SHRP procedures. Figure 5 gives the inverse creep compliance for both crumb and devulcanized rubber modified binders together with data for 85-100 and 150-200 penetration grade control binders. The addition of the reductively devulcanized rubber provides only a modest improvement to the 150-200 binder because of the low molecular weight of the devulcanized rubber. A hazard involved in this process is the degree of solvent removal; a small amount of residual solvent will have a large effect on the binder properties at high and low temperatures. It should be noted that the 150-200 control and the solvent devulcanized and crumb rubber systems may be compared with each other but may not be compared directly to the oxidatively devulcanized system and the 85-100 control because of the differences in penetration grades. A stiffer binder will naturally perform better at high temperature, so at best one may only examine relative increases of modified binders of different penetration grades over their respective unmodified binders.

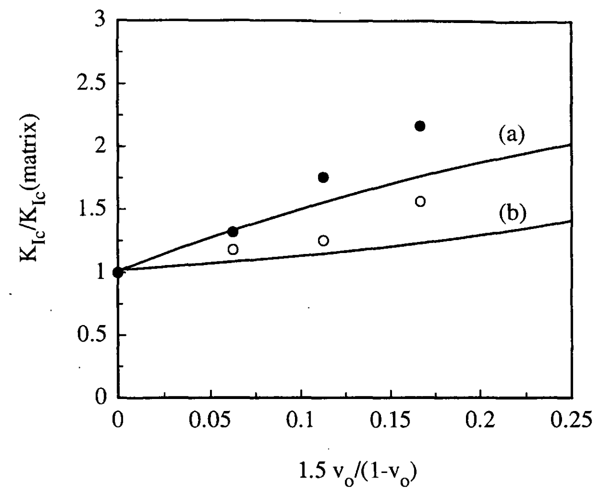
Low Temperature Performance

The low temperature fracture resistance for crumb and devulcanized rubber modified binders was determined in a notched three point bend test. The average fracture toughness was calculated from the failure loads of approximately 10 to 20 samples.



(●-20 mesh crumb rubber, O-oxidized system, ◇-85/100 control, Δ-150/200 control, □-solvent devulcanized system)

FIGURE 5 Inverse creep compliance for rubber modified asphalt binders.

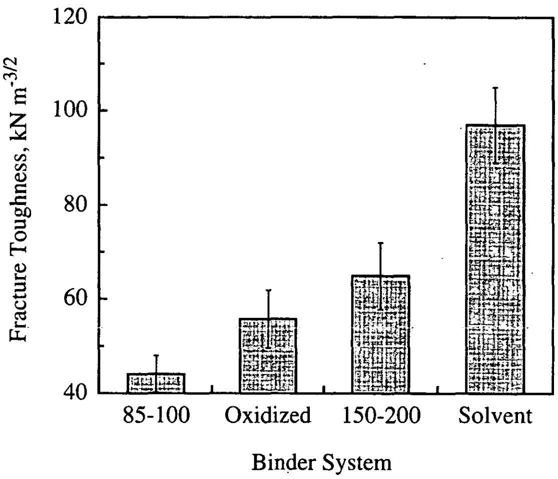


(●-surface modified, o-unmodified, (a)-theoretical curve for non-interacting secondary cracks, (b)-theoretical curve for interacting secondary cracks)

FIGURE 6 Relative fracture toughness increase in crumb rubber modified asphalt binders.

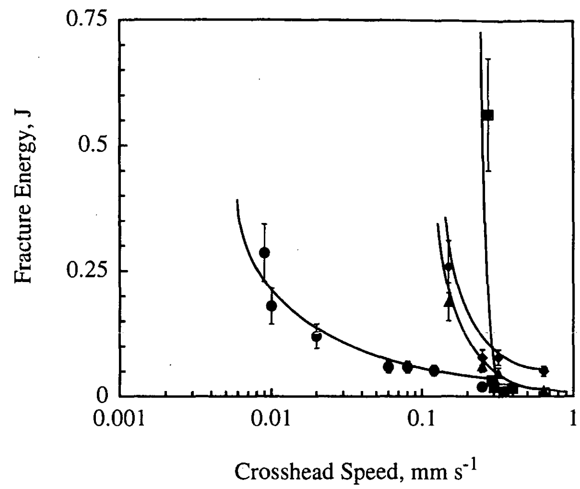
Figure 6 gives the relative improvement in fracture toughness of the crumb rubber modified binders over the matrix fracture toughness as a function of the volume fraction v_o of rubber in the matrix. This graph includes the theoretical contribution to the fracture toughness from crack pinning effects. The crumb rubber particles form obstacles for a propagating crack front and therefore pin the crack and produce a tougher binder. The fracture toughness increase in the presence of the obstacles is now determined by the load required to propagate a series of secondary cracks bowing out from between the particles. A more extensive discussion on the crack pinning theory appears elsewhere (*24*). Figure 6 does show, however, that the cause for the increase in fracture toughness in these systems is due to both crack pinning and viscoelastic effects. The energy consumed to produce the new fracture surfaces is dissipated through viscous and elastic flow mechanisms. The surface grafting of the polybutadiene onto the crumb rubber particles improves the fracture toughness substantially (Figures 1 and 6) owing to the additional viscous energy that is dissipated when the well-bound particles are pulled out from the fracture surfaces. For the unmodified crumb rubber systems, this effect is not present and particles can easily debond from the matrix allowing cracks to propagate at much lower stress levels.

In the case of devulcanized rubber binder systems the crack pinning mechanism is not occurring simply because there are no obstacles to pin the propagating crack front. Increases in fracture toughness for such systems can be attributed solely to enhanced shear yielding of the matrix and localized yielding in front of the propagating crack tip, whereas unmodified asphalt fails in a brittle fashion. Figure 7 gives the fracture toughness for the oxidatively devulcanized rubber and the fracture toughness for the solvent devulcanized systems. The fracture toughness for two control binders is also given for comparison. These data show that the relative increase in low temperature performance is much higher for the solvent devulcanized system than the oxidatively devulcanized system because of the adverse effect of the oxidative air-blowing process on the binder, a fact that is well known in the literature (*25*).



(crosshead speed of 0.01 mm/s, error bars show 90% confidence limits)

FIGURE 7 Fracture toughness for rubber modified binders at −20°C.



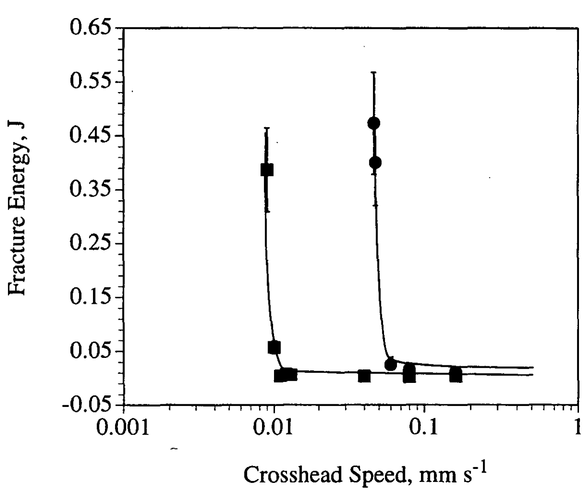
(●-oxidized rubber binder, ▲-unmodified crumb 20 mesh, ◆-surface modified 20 mesh crumb, ■-solvent devulcanized system, error bars estimated at 20%)

FIGURE 9 Brittle-to-ductile transitions for various rubber modified binders.

Variable Strain Rate Tests

The fracture toughness test was conducted at various strain rates at −10°C to learn more about the yield behavior of the rubber modified binders. At low temperatures the pavement shrinks and the asphalt binder may allow for yielding to release the thermal stresses. Binders that yield at higher rates of thermal shrinkage and at lower temperatures are more favorable for use in cold climates.

Figures 8 and 9 give the failure energy as a function of the strain rate for 85-100 and 150-200 grade control binders, unmodified crumb, surface modified crumb, solvent devulcanized and oxidatively devulcanized binders. From these results it is apparent that the position of the brittle-to-tough transition is shifted by the addition of rubber to the binder.



(■-85/100 penetration grade, ●-150/200 penetration grade, error bars estimated at 20%)

FIGURE 8 Brittle-to-ductile transition for bow river source asphalts.

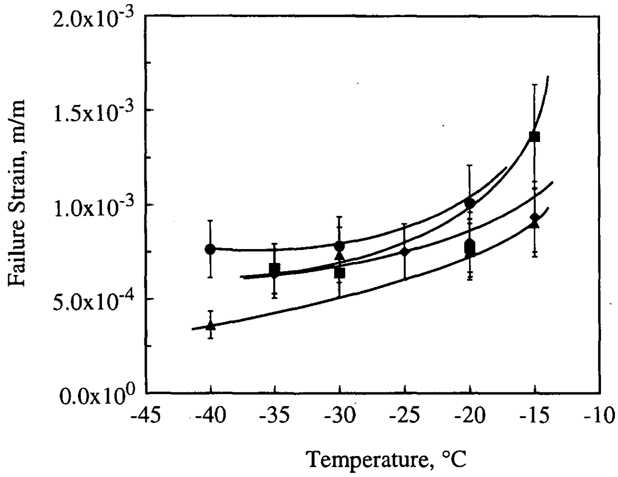
Asphalt Concrete Tests

Eventually it matters most how rubber modified binders perform in a pavement on a road subjected to both environmental and traffic-induced distress mechanisms. For this reason many paving authorities are using test sections to evaluate polymer and rubber additives. However, obtaining results from test sections can take several years; therefore, laboratory tests are often used to simulate extreme pavement conditions to obtain accelerated performance results. One of the main problems with laboratory tests is their inability to simulate true distress conditions.

In this work we have measured the failure strains for a number of crumb and devulcanized rubber modified binders in order to compare the different binders. Figure 10 shows the results for crumb rubber and surface-grafted crumb rubber, solvent-devulcanized and oxidatively devulcanized binders, respectively. The solvent-devulcanized system shows the highest failure strain, which can be explained by this binder's lower modulus and higher fracture toughness. The data also show that there is no great difference between the surface-modified and the unmodified crumb systems. This may be due to the type of failure in the mix; further research should look at this in more detail. Finally, the oxidatively devulcanized binder shows the lowest failure strains at all temperatures.

CONCLUSIONS

The properties of several rubber modified asphalts have been evaluated at high and low temperatures. High temperature inverse creep compliance values of the modified binders show modest improvement for a solvent devulcanized rubber and good improvements for crumb and oxidatively devulcanized rubbers. Relative increases in low temperature fracture toughness of the modified over unmodified binders show large differences, with the solvent-devulcanized rubber imparting a greater improvement. Data on the low temperature failure strains of mix specimens prepared from the modified



(●-solvent devulcanized system, ▲-oxidized system, ◆-surface modified 20 mesh binder, ■-unmodified 20 mesh crumb rubber binder, error bars estimated at 20%)

FIGURE 10 Failure strains for rubber modified asphalt concrete samples.

binders are presented, revealing the greater susceptibility of the stiff, oxidatively devulcanized rubber system to thermal stress induced cracking. The two crumb rubber systems perform very similarly, while the mix made with the reductively devulcanized rubber modified binder shows the highest failure strains.

FURTHER WORK

The results of this research indicate that the devulcanized rubber modified binders have the potential to give improved low temperature performance. An economical and environmentally acceptable process should be developed to produce such binders. These systems would have the advantage of being stable during storage at high temperatures.

Further work should continue by investigating mix performance. Thermal stress restrained specimen tests that reflect thermal shrinkage, stress relaxation, and failure characteristics may give an indication of how these rubber modified binder systems change the brittleness temperature of the mix.

ACKNOWLEDGMENTS

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Use of Incinerator Residue in Asphalt Mixtures

T.F. FWA AND N.K. AZIZ

Earlier studies in Singapore on the use of incinerator residue in asphalt mixtures did not arrive at a satisfactory mix design to meet requirements of local road authority. This study reexamines the outstanding issues and performs a series of tests with the aim of arriving at an acceptable asphalt mix using incinerator residue as a partial replacement for the aggregate. Results of aggregate tests on the incinerator residue redirected the focus of the study to using incinerator residue as mineral fillers in the local standard mix. Mix design analysis and stability tests were performed to identify the mix formula that would meet the local specifications. Durability tests with respect to moisture resistance of compacted mix were also carried out. These tests led to the recommendation of using the portion of incinerator residue passing sieve size 0.3 mm to replace the corresponding sizes of granite aggregate in the standard local design mix. The modified mix provides improved stability to the standard local mix and offers better moisture-resistance properties. This utilization of incinerator residue represents about 35 percent by weight of the incineration plant residue discharge, thereby presenting a partial solution to the disposal problem of the waste material.

Incineration is one of the most effective refuse-disposal methods for achieving 90 percent reduction in volume. Incineration plants with modern combustion controls are widely used today, especially in urban environments where other forms of refuse disposal are undesirable. Residue from the combustion process has been commonly disposed of as landfill materials. In regions such as the city state of Singapore where land is scarce and the availability of dumping ground is limited, it is logical to look into other possible means of disposing of the incinerator residue.

Research on the possible use of incinerator residue as an aggregate replacement in asphalt mixtures has been ongoing in Singapore since 1989 (*1–4*). Based on the findings of these earlier studies that identified several limitations in using incinerator residue as aggregates in asphalt paving mixtures, this study focuses on the use of incinerator residue as partial replacement for the fine aggregate in asphalt mixtures. The feasibility of this scheme is studied by examining the engineering properties and moisture sensitivity of the following three asphalt mixtures: (a) a standard local dense-graded asphalt mix, designated as W3, which is commonly used for road construction in Singapore; (b) the W3 mix with incinerator residue as partial replacement of aggregate; and (c) the W3 mix with the addition of hydrated lime.

FINDINGS OF EARLIER STUDIES

The suitability of incinerator residue from Singapore incineration plants as a complete replacement of aggregate in dense-graded asphalt mixtures was studied by Kang and Aziz and Ramaswamy (*1,2*). Difficulties were encountered in the following areas: (a) there were relatively large fluctuations in test results owing to wide variations in composition of incinerator residue; (b) the gradation of the incinerator residue did not meet the grading requirements specified for local asphalt road construction; (c) the asphalt mix prepared with the incinerator residue could not satisfy the percent of air void and the Marshall flow requirements set by the local authority; and (d) the optimum binder content varied from 10 to 12 percent, more than double the 5.5 percent used in the original asphalt mix with granite aggregate. In view of these limitations, Aziz and Ramaswamy (*3*) recommended that use of incinerator residue in asphalt mixtures be considered only for secondary road construction.

A work by Lum and Tay (*4*) explored the possibility of incinerator residue as replacement for fine aggregate in asphalt concrete. All aggregates passing sieve size 4.75 mm were replaced by incinerator residue. The final aggregate blend contained 86 percent of incinerator residue by weight. This revised mix formula did not produce much improvement over the mix studied by Aziz and Ramaswamy discussed in the preceding paragraph. The optimum binder content maintained at 12 percent and the air void requirements still could not be met. Lum and Tay concluded that the revised mix formula would only have potential applications in areas with low traffic volume and light axle loadings.

OBJECTIVE OF PRESENT STUDY

Because the local road authorities do not accept mixes that fail to meet their specified requirements (see Table 1), the mix formulas proposed in the earlier studies could not be used in Singapore. Therefore, the objective of this study was to identify an aggregate blend with the incinerator residue that would produce an asphalt mix that meets the specified requirements.

By testing the relevant engineering properties for road aggregates, this study first established that the coarse aggregate portion of the incinerator residue did not qualify for use in road construction in Singapore. This result, together with the findings of earlier studies, led this study to focus on the portion of incinerator residue passing sieve size 0.3 mm as replacement for granite aggregate of corresponding sizes in asphalt mixtures. With the grading requirement of W3 mix shown in Table 1, the incinerator residue would constitute 20 percent by weight of the aggregate blend. Based on the gradation of the incinerator residue shown in Figure 1, the amount of incinerator residue used for the mix represents about 35 percent of the total weight of the incinerator residue discharged from an incineration plant.

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TABLE 1 Specification Requirements for Mix W3 by Local Authority

(a) Asphalt : Penetration grade 60/70
Binder content = 5.5% by weight of total mix

(b) Aggregate : Granite aggregates with the following gradation

Sieve Size	19mm	13mm	9.5mm	6.4mm	3.2mm	1.2mm	0.3mm	0.075mm
Percent Passing	100	95	90	77	58	37	20	6

(c) Marshall Design Criteria

Marshall Stability (75-blow compaction)	Minimum 9 kN
Marshall Flow	2 mm - 4 mm
Air Voids in Mix	3% - 5%
% of aggregate voids filled with asphalt	75% - 82%

PROGRAM OF STUDY

The test program consisted of three parts. Part A covered testing of the engineering properties of the coarse aggregate portion of the incinerator residue. Part B determined the optimum binder content for the proposed mixture and the corresponding mix properties, and checked against mix requirements set by the local road authorities. Part C examined the moisture-resistance properties of the proposed mixture.

Part A: Aggregate Properties

The desirable properties of a road-construction aggregate are high resistance to crushing, abrasion and impact, and strong adhesion to bitumen. The tests carried out in this study were (a) an aggregate crushing test in accordance with British Standard BS 812 Part 110 (*5*); (b) a Los Angeles abrasion test in accordance with ASTM C131 (*6*); (c) an aggregate impact value test in accordance with British Standard BS 812 Part 112 (*5*); and (d) a static immersion test in accordance with ASTM D1664 (*7*). For comparison purposes, the same tests were also performed on granite aggregate, the only type of natural aggregate available for road construction in Singapore.

The aggregate crushing value measures the percentage of fines (passing through the 2.36-mm sieve) generated from about 6.5 kg of aggregates passing the 12.5-mm sieve and retained on the 10-mm sieve under a gradually applied load of 40 tonnes (39.4 tons). The Los Angeles abrasion loss refers to the percentage wear caused by the abrasive action of the test to aggregates of sizes ranging from 19 mm to 2.36 mm. The aggregate impact value provides an indicative measure of an aggregate's resistance to fracture under impact. In this test, aggregates passing the 12.5-mm sieve and retained on the 10-mm sieve were subjected to 15 blows of a 14-kg hammer dropping from a height of 38 cm. The resulted percentage of fines passing the 2.36-mm sieve is the aggregate impact value. For all three tests, stronger aggregates would produce lower test values.

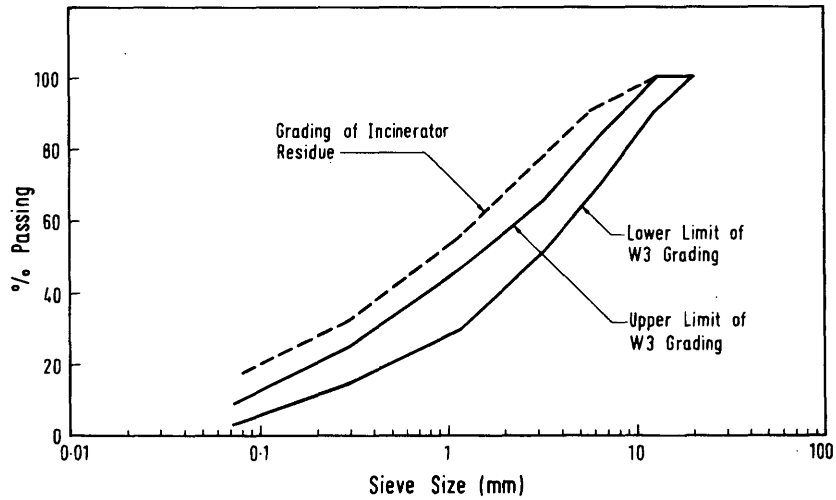

FIGURE 1 Gradation curves for W3 mix and incinerator residue.

Part B: Properties of Compacted Asphalt Mixtures

The following three different mixes were tested in this study:

(a) Normal W3 mix—W3 mix with granite aggregate, as specified in Table 1.
(b) Modified W3 mix 1—W3 mix as in the normal mix, but replacing all aggregates passing sieve size 0.3 mm with incinerator residue.
(c) Modified W3 mix 2—W3 mix as in the normal mix, with the addition of hydrated lime at a dosage equal to 1.5 percent by weight of total aggregate.

Mixes (a) and (c), each having an optimum binder content of 5.5 percent by weight of total mixture, are standard asphalt mixes used by local road authorities. Hydrated lime is used as an additive to increase the resistance of asphalt mixtures to moisture damage. They were included in the test program to provide comparative assessment for mix (b). Since the Marshall test procedure (*8*) is adopted by all local road authorities for asphalt mix design, the mix parameters examined in this part of the study were Marshall stability, Marshall flow, percent of air void, voids in mineral aggregate (VMA), and bulk density.

Part C: Moisture Sensitivity of Compacted Asphalt Mixtures

Specimens of the three mixes described in Part B were tested for their moisture sensitivity by subjecting them to a cyclic wetting-drying treatment. This treatment exposed the test specimens to alternating wetting and drying in an enclosed chamber. The detailed operating features of the chamber are described elsewhere (*9*). This is essentially a moisture treatment to induce moisture into the specimens treated. Wetting of the specimens was achieved by spraying tap water, at about 28°C, by means of shower heads positioned in the chamber. Drying was effected by heating using ceramic heaters after the spraying of water was cut off.

This study adopted a 4-hr treatment cycle that consisted of 2 hr wetting followed by 2 hr drying. The length of the wetting phase was selected to represent approximately the mean duration of rainfall in Singapore (*10*). The 2-hr drying period was chosen so as to attain the desired maximum specimen temperature of about 60°C. Previous experience (*9*) with the chamber has indicated that an equilibrium moisture state in the specimens treated would be reached after about 150 cycles. For this study 150 cycles of 4-hr treatment were used. The treatment therefore took 600 hr (or 25 days) to complete.

The resilient modulus and the indirect tensile strength were adopted as the basis for assessing the relative performance of the three mix types. The resilient modulus was determined according to the procedure outlined in ASTM D4123 (*11*) using a load of 1.6 kN (0.36 kip) applied at a frequency of 1 Hz with loading duration equal to 400 ms. The indirect tensile strength was determined using a rate of loading equal to 50.8 mm/min (2 in./min).

RESULTS OF PART A: AGGREGATE PROPERTIES

Table 2 summarizes the average results of the following three tests: the aggregate crushing test, the Los Angeles abrasion test, and the aggregate impact value test. The maximum allowable value of test result for each test, as specified by the local road authorities, is also indicated. It is apparent from the results that the incinerator residue was significantly inferior to granite aggregate in the three aspects tested. It failed to satisfy the specified limit in each case by a rather wide margin. This effectively excluded the possibility of using the incinerator residue as coarse aggregate and the coarser end of fine aggregate in asphalt mixtures.

The static immersion test on bitumen coated granite aggregate and incinerator residue showed that both were able to retain more than 95 percent of the coating. Observation indicated that the bituminous film on the granite aggregate displayed a rich black and shiny texture, while that on the incinerator residue was dull with traces of brown spots. The static immersion test apparently was not severe enough to tell the difference between the two types of aggregate.

RESULTS OF PART B: PROPERTIES OF COMPACTED MIXTURES

Figure 1 shows an example of the grading distribution of the incinerator residue together with the upper and lower envelopes of W3 mix. The incinerator residue passing sieve size 0.3 mm constituted about 35 percent by weight of the incinerator residue. This portion of the incinerator residue was used in the modified W3 Mix 1 to replace the corresponding sizes of granite aggregate.

The properties of the compacted specimens of W3 mix and the modified W3 mix 1, each prepared in accordance with the Marshall procedure outlined in ASTM D1559 (*8*), are summarized in Table 3. The choice of 5.0 percent binder content for modified W3 mix 1 was governed by the air void requirement. At this binder content, all specified ranges of test properties as shown in Table 1 were satisfied.

It is noted from Table 3 that the modified mix produced higher Marshall stability than the original mix. This was also

TABLE 2 Results of Tests on Aggregates

	Granite Aggregate	Incinerator Residue	Specified Maximum
Aggregate Crushing Value	23.8%	42.3%	35%
Los Angeles Abrasion Value	22.4%	44.1%	30%
Aggregate Impact Value	18.9%	38.4%	30%

TABLE 3 Results of Marshall Test

Properties	Normal W3 Mix (Granite Aggregate)	Modified W3 Mix I (with Incin. Residue)
Optimum Binder Content	5.5%	5.0%
Bulk Specific Gravity	2.383	2.392
Marshall Stability	12.8 kN	15.2 kN
Marshall Flow	2.9 mm	3.2 mm
Air Voids in Mix	3.5%	3.0%
% voids with asphalt	76%	80%
Voids in Mineral Agg.	14.6%	15.0%

the case in earlier studies (*1–4*) for mixes with partial replacement of aggregate by incinerator residue. The most significant improvement over the mixes of earlier studies is the reduction of binder content requirement from as much as 12 percent to 5 percent—a level compatible with the normal 5.5 percent used for W3 mix.

RESULTS OF PART C: MOISTURE SENSITIVITY TEST

This part of the study involved three mix types: the normal W3 mix, W3 mix with incinerator residue (modified W3 mix 1), and W3 mix with hydrated lime (modified W3 mix 2). Thirty-five Marshall size specimens per mix type were prepared according to the ASTM D1559 (*8*) and subjected to the cyclic wetting-drying treatment.

Figure 2 shows how the 35 specimens were used in the test program. Five specimens, not subjected to the wetting-drying moisture treatment, were tested for resilient modulus and indirect tensile strength to provide the reference for evaluating the effect of moisture treatment. The remaining six groups of five specimens each, all subjected to 150 cycles of wetting-drying treatment, were wiped surface-dry immediately following the treatment and tested after the following six periods of drying in room condition at 28°C: 0 hr, 3 hr, 6 hr, 24 hr, 3 days, and 7 days.

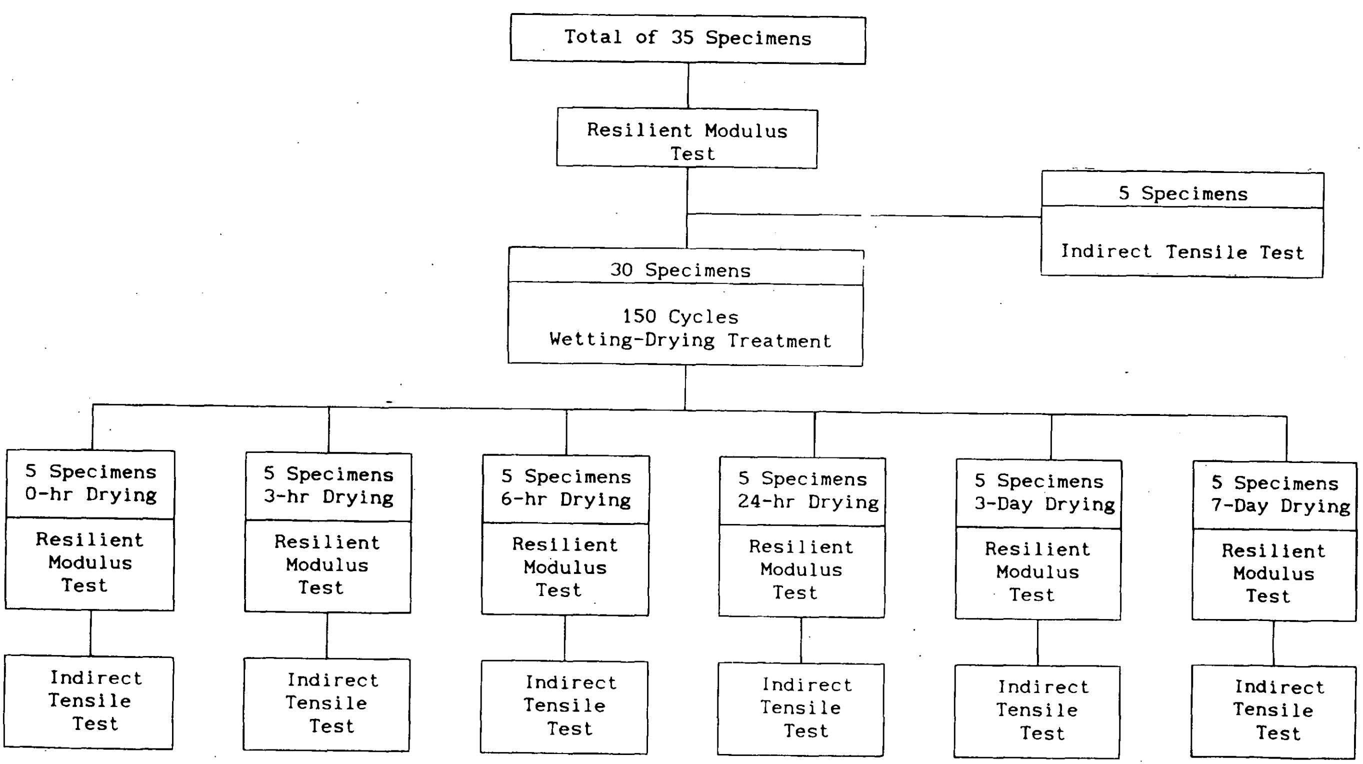

FIGURE 2 Flow diagram of experimental program.

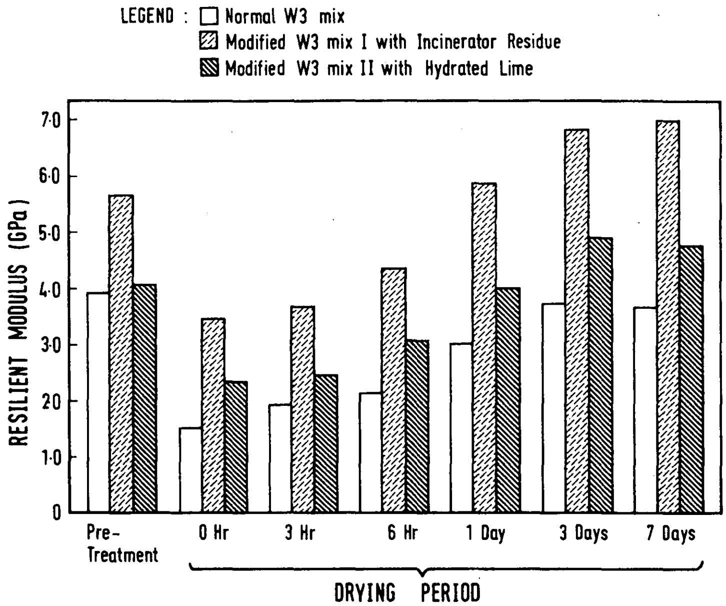

FIGURE 3 Changes in resilient modulus after moisture treatment.

Evaluation Based on Resilient Modulus

The results of resilient modulus tests are presented in Figure 3. The before-treatment value is the mean resilient modulus of all 35 specimens, and the after-treatment values for different drying periods are each mean resilient modulus of a group of 5 specimens. It is noted that the modified W3 mix 1 with incinerator residue produced specimens with resilient modulus more than 30 percent higher than those of the other two mixes.

Figure 3 indicates that the resilient moduli of the specimens of all three mix types fell significantly immediately after the moisture treatment. The normal W3 mix suffered the largest loss in resilient modulus, while the W3 mix with incinerator residue had the highest resilient modulus both before and after treatment.

For all three mix types, the resilient moduli gradually increased as the specimens were allowed to dry under room condition. The difference in the final recovered resilient modulus values of the three mix types is particularly interesting. Specimens of the normal W3 mix did not achieve full recovery in resilient modulus after 7 days. On the other hand, specimens of both the modified mix 1 and 2 not only regained their respective pretreatment levels of resilient modulus after about 24 hr of drying, but also exceeded these levels upon further drying.

Since the resilient modulus test is nondestructive in nature, and the test was performed on all specimens before and after moisture treatment, it was possible to compute directly for each specimen the change in its resilient modulus caused by the treatment. The comparison based on this computation is presented in Figure 4. The results show that immediately after the moisture treatment, the normal W3 mixes suffered approximately 60 percent loss in resilient modulus, whereas the other two modified mixes had losses of about 40 percent. For all three mixes, part of the resilient modulus losses were regained as the specimens were allowed to dry gradually. After 7 days of drying, specimens of the normal W3 mix retained about 95 percent of pretreatment value of resilient modulus. For both modified W3 mixes it is significant that, after 3 days of drying, there were net gains of 20 percent and 15 percent over their respective pretreatment resilient modulus values. These results reveal a superior moisture-resistance performance of the two modified mixes to the normal W3 mix.

Evaluation Based on Indirect Tensile Strength

As shown in Figure 5, there were almost no differences in the indirect tensile strength of freshly compacted specimens of the three

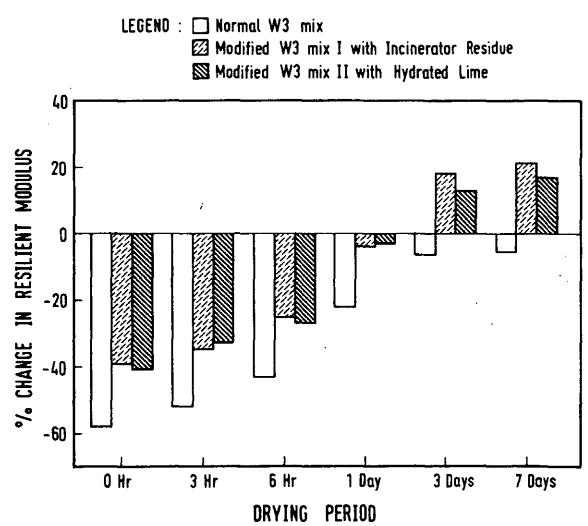

FIGURE 4 Percentage changes in resilient modulus after moisture treatment.

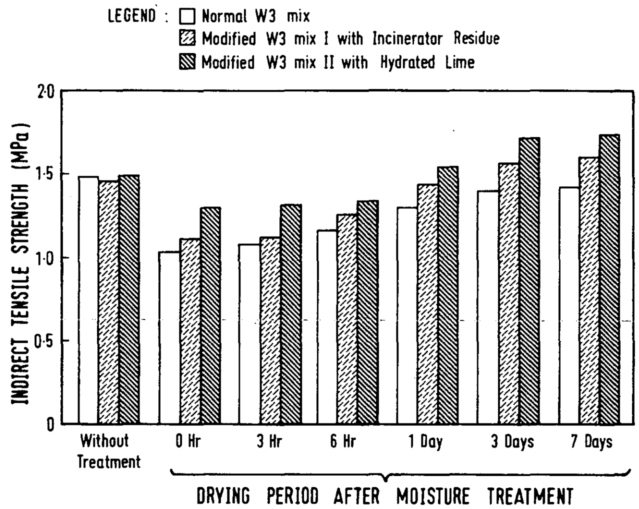

FIGURE 5 Changes in indirect tensile strength after moisture treatment.

mix types. However, their respective performances after moisture treatment in terms of indirect tensile strength are not the same. Because the indirect tensile test is a destructive test, the effect of the moisture treatment could only be assessed by comparing the mean test value of a group of specimens that had received moisture treatment with that of a reference group of specimens not subjected to moisture treatment.

The bar chart in Figure 5 presents the trends of variation of measured indirect tensile strength with respect to the length of drying period after moisture treatment. As in the case of resilient modulus tests described in the preceding section, there were substantial drops of indirect tensile strength immediately after the moisture treatment for all three mix types. The normal W3 mix suffered the highest drop, of about 30 percent, followed by about 25 percent for modified W3 mix 1 and about 15 percent for modified W3 mix 2.

For each of the mix types, there was a gradual recovery of indirect tensile strength as the specimens were allowed to dry. Again, the normal W3 mix fell short of full recovery after as long as 7 days of drying, while the two modified mixes registered positive gains over the indirect tensile strength of the reference group after 7 days. However, the magnitudes of difference in the indirect tensile strength among the three mix types were relatively small compared to the magnitudes of difference for resilient modulus.

Comments on Response to Moisture Treatment

It is of interest to study possible factors that led to the different responses of the three mix types to moisture treatment. The following two factors were examined in this study: the moisture absorption characteristics of the mixtures and the properties of binder used.

Moisture Absorption Characteristics

A specimen was assumed to be fully saturated when all its air voids were filled with water. Therefore, the amount of moisture in a compacted specimen at full saturation, *Ws*, was computed as the product of water density and the total air void volume of the specimen. It follows that the moisture content state of a specimen can be defined by the degree of saturation given by the following equation:

$$S\% = \frac{w}{Ws} \times 100\% \qquad (1)$$

where

$S\%$ = degree of saturation expressed as percent air void filled with moisture,
w = mass of moisture in air voids of specimen, and
Ws = total mass of moisture in the specimen at full saturation.

The mass of moisture w in a specimen was obtained by taking the difference in weight between the specimen before and after moisture treatment.

Figure 6 plots the mean moisture absorption levels for various groups of specimens of the three mix types. The degree of saturation achieved after the moisture treatment was about 30 percent for all the specimens. In terms of degree of saturation, there were no statistically significant differences among the moisture absorption capability of the three mix types. The patterns of variation of degree of saturation with drying period were also very similar for the three mix types. Therefore, it is unlikely that the different moisture-treatment responses of the mix types in this study could be related to their moisture absorption characteristics.

Properties of Binder

Hydrated lime is widely recognized as an effective agent to increase the resistance of asphalt mixtures to moisture damage (*11–13*). The superior performance of modified W3 mix 2 to that of the normal W3 mix, as depicted in Figures 3, 4, and 5, is within expectation. The mix with hydrated lime suffered lower loss in resilient modulus and indirect tensile strength when subjected to the moisture treatment, and led to additional gains in both properties upon drying.

The interesting aspect of the test results is with the performance of the modified W3 mix 1, which contained incinerator residue as partial replacement of the mineral fillers in W3 mix. Examination of Figures 3, 4, and 5 indicates that the moisture-resistance performance of this modified mix was very similar to that of modified W3 mix 2 and superior to that of the normal W3 mix. Table 4 shows an

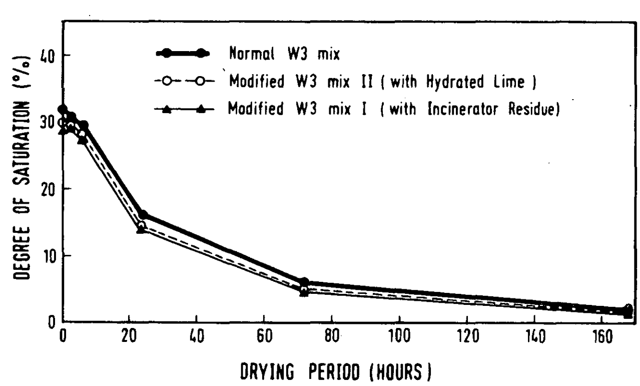

FIGURE 6 Variation of moisture content with drying period after cyclic wetting-drying treatment.

TABLE 4 Composition of Incinerator Residue

Constituent	% by Weight
SiO_2	35.02%
Fe_2O_3	28.33%
Al_2O_3	22.83%
Na_2O	1.93%
K_2O	1.20%
CaO	4.98%
MgO	1.09%
Others	4.62%
(Total)	(100.00%)

average material composition of the incinerator residue determined using the atomic absorption spectrophotometer. Besides the high concentration of oxides of silica and iron, there was also nearly 5 percent by weight of calcium oxide (CaO) in the residue. It was likely that the presence of CaO had contributed to the enhanced moisture-resistance performance of the mix. However, this factor could not explain fully the performance of mix because the response of the mix in terms of resilient modulus (Figures 3 and 4) and indirect tensile strength (Figure 5) were not exactly the same as those of the mix with hydrated lime.

FINDINGS AND CONCLUSIONS

This study presents the results of various tests performed to examine the suitability of a local supply of incinerator residue as paving aggregate. The tests on aggregate properties found that the incinerator residue did not satisfy the strength and stability requirements of local road authority, thereby redirecting the emphasis of the project to studying the feasibility of using the incinerator residue as mineral fillers for paving mixes.

Mix design analysis following the Marshall procedure concluded that by using the portion of incinerator residue passing sieve size 0.3 mm as replacement for the corresponding sizes of granite aggregate in the standard local design mix, all mix property requirements specified by the local road authority could be satisfied. While maintaining the optimal binder content within the normal allowable range of 5 to 6 percent for local practice, a higher Marshall stability was achieved with the modified mix as compared to the standard local mix.

Tests on mix durability with respect to moisture-damage resistance revealed that the modified mix with incinerator residue was superior to the standard mix. It had higher resilient modulus both before and after moisture treatment. It suffered less loss in resilient modulus immediately after moisture treatment, and recovered faster upon drying in regaining the pretreatment level of resilient modulus. While the standard mix could achieve only about 95 percent of the original resilient modulus after 7 days of drying, the modified mix with incinerator residue recorded a gain in resilient modulus over its pretreatment value after only 3 days of drying. In this regard, the modified mix with incinerator residue showed behavior similar to that of the modified mix with hydrated lime. It is believed that the presence of CaO in the incinerator residue could have contributed to such behavior. Moisture-resistance comparison based on indirect tensile strength led to similar conclusions although the differences in the performance among the three mix types studied were relatively less.

The above test results suggest that the incinerator residue could serve as mineral fillers in the paving mixtures used in Singapore. It provides improved stability to the standard local mix and offers better moisture-resistance properties. Because about 35 percent of the incinerator residue can be used for this purpose, this application could be employed as part of local engineers' efforts to use waste materials for modern construction.

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