Characteristics of Rubberised Bitumen Blends

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ABSTRACT. This paper presents some findings from the recently completed research project on "Rubberised Asphalt Testing to UK Standards", funded by WRAP (Waste & Resources Action Programme). One of the aims from this study was to provide information to potential users of rubberised binder on the properties that can be expected from various sources/types of crumb rubber available in the UK, when blended with typical UK bitumens, specifically without using any additive. It was found that the selected three best blends satisfied target requirements for ageing index, softening point and viscosity. In addition, the blends exhibited improvements in short and long term ageing, cohesion characteristics, Fraass breaking point, service temperature range and flash point values compared to typical penetration grade bitumen, and satisfactory residual properties after storage stability testing.

KEYWORDS: ageing, storage stability, rheology, cohesion, resilience, rubberised bitumen.

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1. Introduction

This paper presents some findings from the recently completed research project on "Rubberised Asphalt Testing to UK Standards", funded by WRAP (Waste & Resources Action Programme). One of the aims from this study was to provide information to potential users of rubberised binder on the properties that can be expected from various sources/types of crumb rubber available in the UK, when blended with typical UK bitumens, particularly without using any additive or oil extender. This paper describes the assessment of the rubberised binders; a companion paper describes the assessment of the rubberised asphalt mixtures (WIDY 2009).

2. Materials for Testing

The base bitumen used to manufacture the rubberised bitumen blends comprised four different binder types (two hard and two soft binders) that were produced from Venezuelan and Middle Eastern crudes. Results of initial tests on these binders are presented in Table 1.

Table 1. *Properties of Base Bitumen*

Bitumen	Penetration at 25℃ (dmm)	Softening Point (℃)	Viscosity at 177℃ (cP)	Bitumen Composition			
				Saturates (%)	Aromatics (%)	Resins (%)	Asphaltenes (%)
Venezuelan 40/60	54	52.2	109	7.0	55.9	20.4	16.7
Venezuelan 100/150	135	42.0	72	8.5	55.1	18.9	17.6
Middle Eastern 40/60	50	52.5	80	3.4	60.9	21.2	14.4
Middle Eastern 100/150	124	41.0	75	4.4	64.8	19.9	10.9

The rubber reclaimed from either car or truck tyres, from either ambient or cryogenic production methods, were supplied in sizes available as "stock products". Hereafter, rubber reclaimed from car and truck tyres and produced at ambient temperature are called ambient car (AC) and ambient truck (AT), respectively; rubber reclaimed from car tyre and produced at cryogenic temperature is called cryogenic car (CC). Their particle size distributions (gradings) were subsequently determined. The results are summarised in Table 2 (note: a question mark "?" denotes the possibility of the grading falling within the stated category).

Table 2. *Grading of Reclaimed Tyre Rubber (UK 'Stock Products')*

Type	Target Grading	Cryogenic Car (CC)			Ambient Car (AC)		Ambient Truck (AT)		
Rubber Size		0.4-0.63 mm	0.63-1.4 mm	1.4-2.0 mm	Fine	Coarse	Truck No.30	Truck No.12	Truck 4-6mm
Sieve Size (mm)	Cumulative % Passing								
5	-	100	100	100	100	100	100	100	100
3.35	-	100	100	100	100	100	100	100	92
2	100	100	100	100	100	80	100	100	61
1.18	65 – 100	100	85	12	100	9	100	77	12
0.600	20 – 100	100	23	6	88	0	87	41	2
0.300	0 – 45	71	5	2	29	0	7	34	2
0.063	0 – 5	2	0	0	1	0	0	0	2
PAS 107 Category	-	*Fine*	-	*Granulate*	*Fine?*	*Granulate*	*Fine?*	-	*Granulate?*

According to Publicly Available Specification (PAS) 107, rubber particles having sizes of 1 – 10mm, 0 – 1mm, and 0 – 0.5mm are classified as granulate, powder and fine powder respectively. These categories are also shown in the above table, for cross-reference purposes. Some of the UK "stock products" presented in the above table do not comply with the PAS 107 categories. This is an issue that needs to be addressed by the recycled rubber industry, and it is hoped that they can agree with standardised classes for rubber particle size (such as that proposed in PAS 107) in the future. Nevertheless, it is expected that a blend of two rubber sizes may be required in order to meet the target grading for use in rubberised bitumen.

3. Preliminary Screening Test

According to the ASTM D8 definition, rubberised bitumen is a blend of bitumen, reclaimed tyre rubber and certain additives in which the rubber component is at least 15 percent by weight of the total blend and has reacted in the hot bitumen sufficiently to cause swelling of the rubber particles. Consequently, therefore, rubberised bitumen is not a homogenous blend; traces of swelled rubber particles are still visible in the finished blend. Experience in the US, Australia, South Africa and parts of Europe indicate that rubber is typically added at a rate between 18 and 22% by weight of the total binder. Therefore, rubber additions of 15, 18.5 and 22% by weight of the total binder were adopted for this study, to cover the above range of minimum and maximum rates of rubber addition. Four different sources of rubber (one of which is a blend of two sources) and four different base bitumens were selected for this project. Combinations of these variables resulted in a total of 48 rubberised bitumen blends. As previously stated, the use of oil extender or additive was excluded from this study.

Each of the 48 rubberised bitumen blends was manufactured using a harmonised blending procedure. This involved blending/mixing in a Silverson L4RT high shear apparatus, at 177℃ and 6000 rpm for 15 minutes and 1500 rpm for 165 minutes, resulting in 180 minutes (3 hours) of blending. The specification for rubberised binder adopted in this study required the viscosity test results (at 177℃ and 20 rpm) of these blends to be between 1500cP – 5000cP. Subsequently, viscosity measurement on the 48 rubberised bitumen blends and 4 unmodified bitumen control (i.e. base bitumen) samples was carried out. The viscosity testing was initially carried out to AASHTO; however, during the course of the testing, it was found that another method, ASTM D6114 is more universally adopted for testing rubberised bitumen. Consequently, the latter was adopted for the remainder of the tests.

A Brookfield Viscometer with a No. 27 spindle was used in the main viscosity test programme at a speed of 20 rpm. In addition, a parallel test was also carried out using a Bohlin viscometer using different spindle sizes and rotational speeds; however the results are not presented in this paper. Variations in test results due to different test protocols and arrangements have been observed. Subsequently, the main analysis was based on the results obtained from the Brookfield Viscometer testing under the above test conditions. Testing was performed on samples taken at 30, 60, 120 and 180 minutes from the commencement of blending/mixing. In addition, penetration (BS EN 1426) and softening point (BS EN 1427) tests were carried out on each blend at the end of the mixing period (180 minutes).

A target grading envelope for the rubber particles similar to that successfully used in the US was adopted in this study. As mentioned previously, there were four different sources of rubber i.e. 100% AC, 100% AT, 100% CC and a 50/50 blend of AC and AT; their particle size gradations are summarised in Figure 1.

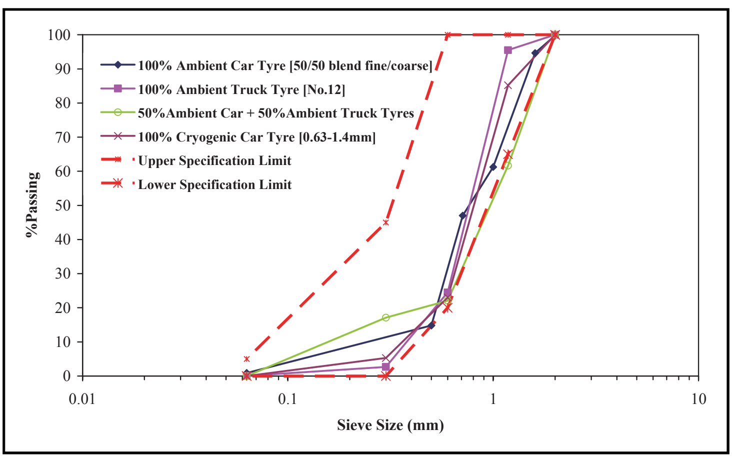
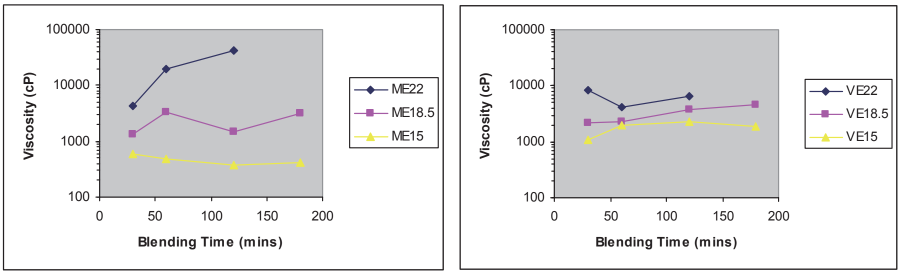


Figure 1. *Target "Standard Size" Gradation for Rubber Particles*

With the exception of rubber particles produced at ambient temperature from car tyre (AC), appropriate sizes of the stock products available in the UK can be readily used to meet the target "standard size" grading. For the AC tyre rubber, a 50/50 blend of fine and coarse car tyre rubber was required in order to meet the grading requirements.

Results for the 48 blends are summarised according to the binder type and rubber percentage; for example VE22 stands for Venezuelan bitumen mixed with 22% rubber and ME22 for Middle Eastern bitumen mixed with 22% rubber. Some examples are represented in Figure 2.



40/60pen, Ambient Car 40/60pen, Ambient Car

Figure 2. *Changes in viscosity with blending time and crumb rubber type*

The viscosity test results were compared against target viscosity values between 1500cP – 5000cP. In addition, one would expect the viscosity to rise as the rubber dissolved in the bitumen and stabilise when solution was essentially complete. Where this did not occur in the required time, or the viscosity fell with time, or was outside the permitted range, this bitumen/rubber combination was not considered favourably. It should be noted that the sample size of the test specimen and any lack of complete homogeneity in mixing could also lead to outlying results.

Results from the penetration and softening point tests, carried out on samples taken at the end of 180 minutes blending, are summarised Table 3 (note: 22% addition was also tested but is not reported here). "Best" rubberised bitumen blends with "standard size" rubber particles were subsequently selected based upon the viscosity (main criterion), together with the penetration and softening (additional criteria) test results. The following "best" blends were subjected to further assessment:

- 81.5% ME 40/60 + 18.5% AC
- 81.5% ME 40/60 + 18.5% CC
- 81.5% VE 40/60 + 18.5% AC
- 81.5% VE 40/60 + 18.5% CC
- 85% VE 40/60 + 15% AT
- 81.5% VE 100/150 + 18.5% CC

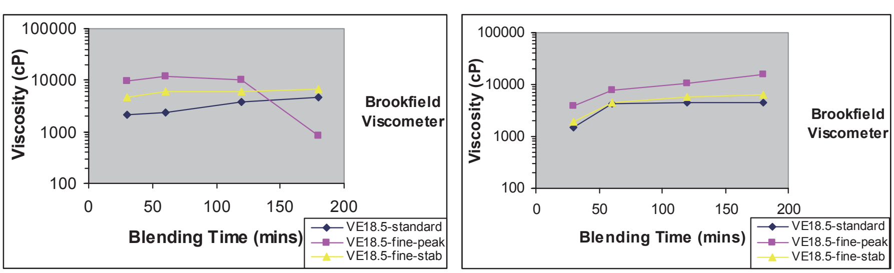
Table 3. *Penetration and softening point tests rubberised bitumen*

Binder Sample	15% Rubber/85%Bitumen				18.5% Rubber/81.5%Bitumen			
	AC	AT	50:50 AC/AT	CC	AC	AT	50:50 AC/AT	CC
	Penetration at 25℃ (dmm)							
Venezuelan 40/60	36	32	54	39	30	34	31	30
Venezuelan 100/150	69	54	67	61	-	41	52	66
Middle East 40/60	44	41	44	36	35	70	47	46
Middle East 100/150	63	77	91	71	74	124	80	91
	Softening Point (℃)							
Venezuelan 40/60	67.5	67.3	52.2	64.5	72	76.5	72	70.2
Venezuelan 100/150	59	59	59.8	57	-	71	61	63
Middle East 40/60	60.4	68.4	64	63	67.6	62.8	64	63.8
Middle East 100/150	54.8	58.2	51	55	55.6	52.4	55.8	51.4

4. Effect of rubber size

Since the adopted "standard" rubber particle size was already on the coarser side of the grading envelope, the effect of using a finer rubber particle size (still within the grading envelope) was evaluated using a similar suite of testing. The viscosity measurement of each rubberised bitumen blend at 177℃ and 20 rpm was carried out by using the Brookfield Viscometer on samples taken at 30, 60, 120 and 180 minutes after the commencement of blending/mixing.

In this exercise, both maximum (peak-ASTM D6114) and "stabilised" (AASHTO T316) viscosity values were recorded as "fine-peak" and "fine-stab" respectively, as well as penetration test (BS EN 1426) and softening point test (BS EN 1427) results. Some examples of the test results are represented in Figure 3. The viscosity results for blends using "standard" size rubber are also reproduced in this figure to ease the comparison.



40/60pen, Ambient Car

40/60pen, Cryogenic Car

Figure 3. *Changes in viscosity with blending time and crumb rubber size*

Finer rubber particle size, having larger surface area than the ‘standard’ size, might be expected to yield greater reaction and interaction with base bitumen and ultimately lead to the increased viscosity observed in the above test results. All blends incorporating a finer rubber particle size exceeded the maximum viscosity values of 5000 cP; consequently none of them was considered suitable for adoption in the next stage of assessment. Therefore, the options were limited to those incorporating the “standard” size rubber as used in the preliminary tests. The following three best blends (all incorporating the “standard” size rubber), were selected:

- 84% ME 40/60 + 16% AC
- 81.5% VE 40/60 + 18.5% AC
- 81.5% VE 40/60 + 18.5% CC

The above three blends were subsequently subjected to the following comprehensive tests: (i) determination of Ageing Index (BBA HAPAS SG4 protocol); (ii) penetration testing at 4℃ (BS EN 1426); (iii) softening point testing (BS EN 1427); (iv) rotational viscosity at 177℃ (ASTM D6114); and, (v) resilience at 25℃ (BS EN 13880-3). The results obtained from this assessment are detailed in Table 4.

Table 4. *Comprehensive Test Results*

Tests	Binder Sample					Target values for rubberised bitumen with 50/70 base bitumen
	100% ME 40/60 (Base Bitumen)	100% VE 40/60 (Base Bitumen)	16% AC + 84% ME40/60	18.5% AC + 81.5% VE40/60	18.5% CC + 81.5% VE40/60	
Ageing Index	5.28	7.31	3.16	2.16	2.18	< 15
G* at 25℃ (Unaged), Pa	6.59E+05	3.98E+05	6.64E+05	8.33E+05	7.67E+05	-
G* at 25℃ (After RTFOT and HiPAT), Pa	3.48E+06	2.91E+05	2.10E+06	1.80E+06	1.67E+06	-
Penetration (4℃, 200g, 60s), dmm	-	-	29	20	19	> 15
Softening Point, ℃	51	50	63	72	70.2	> 54
Rotational Viscosity (177℃), cP	80	109	2550	4630	4510	1500-5000
Resilience (25℃), %	0	5	13	29	26	>25

Table 4 shows that ageing the bitumen progressively increased the G* value (complex shear (or stiffness) modulus at a given temperature and loading rate), hence indicating a stiffened or hardened binder. A road paving bitumen typically used in the UK is expected to have an Ageing Index (the ratio between G*HiPAT and G*Unaged at 0.4 Hz) of less than 15; generally the lower the value, the less susceptible the binder would be expected to be to ageing. The level of increase in softening point, viscosity and resilience, and the reduced penetration values, can be linked to the increase in rubber content in these blends. With respect to the typical target reference values for rubberised bitumen incorporating 50/70pen base bitumen, the test results indicate that the selected blends satisfied the requirements for ageing index, softening point and viscosity. Furthermore, both rubberised blends with Venezuelan bitumen met the criterion for resilience; however, the Middle Eastern blend fell short of the recommended resilience value although remained higher than that of the base bitumen.

5. BS EN 14023 Tests on the Three Best Performing Blends

The three preferred rubberised bitumen blends selected above were further assessed in accordance with BS EN 14023 (i.e. the framework specification of polymer modified binder) but excluding the elastic recovery tests. Test results are presented in Table 5.

Table 5. *Binder Test Results*

Tests	Methods	Binder Sample		
		16%AC + 84%ME40/60	**18.5% AC + 81.5%VE40/60**	**18.5% CC + 81.5%VE40/60**
Penetration (25℃, 100g, 5s), dmm	EN 1426	35	30	30
Softening Point, ℃	EN 1427	63	72	70.2
Cohesion - Vialit pendulum, J/cm^2	SHW Clause 939	1.9	1.4	1.5
Temperature at Peak Cohesion, ℃		50	50	53
Temperature range at cohesion above 0.5 J/cm^2, ℃		>55	50	50
RTFOT, Change of mass, %	EN 12607-1	-0.16	-0.23	-0.21
RTFOT, Penetration (25℃, 100g, 5s), dmm	EN 1426	24	24	27
RTFOT, retained penetration, %	-	69%	80%	90%
RTFOT, softening point, ℃	EN 1427	65	79.5	73
RTFOT, increase in softening point, ℃	-	2	7.5	2.8
Flash Point, ℃	EN ISO 2592	>300	298	>300
Fraass breaking point, ℃	EN 12593	-12	-8	-11
Plasticity range, ℃	EN 14023 Sub-clause 5.1.9	75	80	81.2
Storage stability, penetration at the top, dmm	EN 13399, EN 1426	39	35	30
Storage stability, penetration at the bottom, dmm	EN 13399, EN 1426	41	33	32
Storage stability, difference in penetration, dmm	-	2	2	2

Tests	Methods	Binder Sample		
		16%AC + 84%ME40/60	**18.5% AC + 81.5%VE40/60**	**18.5% CC + 81.5%VE40/60**
Storage stability, softening point at the top, ℃	EN 13399, EN 1427	56.5	71.5	63
Storage stability, softening point at the bottom, ℃	EN 13399, EN 1427	67	76	77
Storage stability, difference in softening point, ℃	-	*10.5*	*4.5*	*14*
HiPAT, Penetration (25℃, 100g, 5s), dmm	EN 1426	22	18	24
HiPAT, retained penetration, %	-	*63%*	*60%*	*80%*
HiPAT, softening point, ℃	EN 1427	71	90	83.5
HiPAT, increase in softening point, ℃	-	*8*	*18*	*13.3*

Table 5 indicates that the increased softening point and the reduced penetration values after the short-term ageing test (RTFOT) can be linked to the increased rubber content of these blends. A further reduction in penetration and increase in softening point of the residual binders after the long-term ageing test (HiPAT) is also found, consistent with the trend observed on the RTFOT samples.

Cohesion is commonly accepted as an indicator of a bitumen's ability to resist tensile stress under rapid impact loading, such as due to skidding or vehicle braking, and hence the ability to retain aggregate intact within an asphalt material (resist fretting). The higher the cohesion value, the better the aggregate retention. Another related parameter, the temperature range over which the cohesion is at or above 0.5J/cm^2 (TR) is commonly considered to be an indicator of service temperature range; the higher the value, the wider the service temperature range of a bituminous binder. The peak cohesion values and the temperature range over which cohesion values are at or above 0.5 J/cm^2 for the rubberised bitumen blends are much higher than (at least double) those of typical 40/60pen bitumen, indicating significantly improved cohesion characteristics for these blends, implying an improved ability to retain aggregate.

Fraass breaking point (FBP) is the temperature at which thin bitumen films start to show cracks under a small deflection. The lower the FBP value, the better the low temperature performance of the binder. The test results show that the rubberised bitumen blends had FBP values not greater than -8℃. This is similar to that measured for a typical UK 40/60 penetration binder (around -7℃), suggesting that the addition of rubber does not have a detrimental effect on the low temperature properties of the binder.

The plasticity range is the numerical difference between the softening point and Fraass breaking point, which is sometimes considered as the range of service temperatures that a binder may be expected to perform satisfactorily over. The plasticity ranges of the rubberised bitumen blends (i.e. 75℃ or higher) will meet the requirements for BS EN 14023 polymer modified binder Classes 2, 3 and 4, and are considered wider than those found typically for a 40/60pen bitumen (i.e. around 60℃). This suggests the rubberised bitumen blends can be expected to have a relatively wide service temperature range.

As with any other bituminous material, rubberised bitumen will release combustible fumes when heated to sufficiently high temperatures. The flash point provides an indication of the temperature at which a heated bituminous sample will instantaneously flash in the presence of an open flame. The flash point values of the rubberised bitumen blends comply with those generally specified for polymer modified binders, i.e. higher than 250℃. Thus the risk that these blends could cause a fire during material production can be expected to be no greater than that for polymer modified bitumens. However, the blends were observed to release visibly thick white fume with strong odour during laboratory testing, specifically when the test temperature was increased above 200℃. It is recommended that the significance of the presence of this fume is assessed in more detail.

The results on the stability of the blends during hot storage suggest only a marginal difference in the penetration value of the top and bottom part of the storage container, whilst there is up to a 25% difference in the softening point values. Whilst the latter difference is significantly greater than that expected from 40/60pen bitumen or some polymer modified bitumens, the results are considered not unusual and consistent with those reported in the USA for similar materials. Rubberised bitumen blends are known to be susceptible to phase separation (being a two phase, non-homogeneous, blend) and/or devulcanization (thinning due to rubber degradation) after prolonged hot storage.

6. Concluding Remarks

The preliminary assessment highlighted the sensitivity of rubberised bitumen to blending arrangements and test conditions (specifically viscosity testing). Factors that may affect the test results include: blending protocol (duration, shear rate), viscometer (type/make, shear rate, spindle size) and viscosity data interpretation (peak value/ASTM D6114, stabilised value/AASHTO T316). For this research project, harmonised blending and test conditions were adopted, i.e. high shear blending at 6000 rpm for 15 minutes, with rubber being fed in during the first 10 minutes, followed by reducing the shear rate to 1500 rpm for the rest of the remaining blending time of 180 minutes. A Brookfield Viscometer with spindle size No. 27 was used in the main test program.

A total of 48 rubberised bitumen blends of different compositions were manufactured and subjected to viscosity (ASTM D6114), penetration (BS EN 1426) and softening point (BS EN 1427) testing. None of these blends contained any additive or oil extender. The viscosity test results were compared against target viscosity values between 1500cP – 5000cP and, based upon the viscosity criterion and consideration from the penetration and softening point test results, the 'best' six blends were selected for further assessment, of the effect of rubber particle size on the blend properties. During this exercise, both maximum (peak-ASTM D6114) and "stabilised" (AASHTO T316) viscosity values were recorded as well as penetration and softening point test data. It was found that finer particle sizes, which have a larger surface area, led to greater reaction and interaction with base bitumen and hence a higher viscosity of the rubberised bitumen. Three best performing rubberised bitumen blends were selected, all containing UK "standard" size rubber:

- 84% ME 40/60 + 16% ambient car tyre rubber
- 81.5% VE 40/60 + 18.5% ambient car tyre rubber
- 81.5% VE 40/60 + 18.5% cryogenic car tyre rubber

The selected blends satisfied the project requirements for ageing index, softening point and viscosity and both rubberised blends with Venezuelan bitumen met the criterion for resilience. The Middle Eastern blend fell short of the recommended resilience value, although its performance was better than that of unmodified binder.

Short-term and long-term ageing test results generally suggest improved properties of the rubberised bitumens over those expected for unmodified 40/60pen bitumen. The peak cohesion values and the temperature range over which cohesion values are at or above 0.5 J/cm^2 for the rubberised bitumen blends are much higher than those for typical 40/60pen bitumen, indicating significantly improved cohesion characteristics for these blends and hence improved ability to retain aggregate. Fraass breaking point values for the rubberised bitumen were at least similar to or better than those for 40/60pen bitumen.

The plasticity ranges of the rubberised bitumen blends met the requirements for BS EN 14023 polymer modified binder Classes 2, 3 and 4, and were wider than those found typically for 40/60pen bitumen. This suggests that the rubberised bitumen blends can be expected to have a relatively wide service temperature range. The flash point values of the rubberised bitumen blends comply with those specified for polymer modified binders, although the blends were observed to release visibly thick white fume with strong odour during testing, specifically when the test temperature was increased above 200℃. It is recommended that the significance of the presence of this fume is assessed in more detail.

The results on the stability of the blends during hot storage suggested only a marginal difference in the penetration value of the top and bottom part of the storage container and a difference of up to 25% in the softening point values. However, this was not considered unusual and is consistent with results reported in the USA for similar materials.

7. Acknowledgement

The authors would like to compliment and thank Ronald Kibuuka and the laboratory teams in Scott Wilson and NTEC for the efficiency and effectiveness in producing the test data in this paper. The work was won from WRAP by Scott Wilson in partnership with NTEC, by competitive tender. WRAP helps individuals, businesses and local authorities to reduce waste and recycle more, making better use of resources and helping to tackle climate change; more information on WRAP's work can be found on www.wrap.org.uk. Contributions from Aggregate Industries, Tarmac, Nynas, UK Bitumen, Moncton Rubber Technology, Tyregenics, Allcock and Sons, and Singleton Birch, in providing the materials and the streams of advice from the Technical Advisor to WRAP, Mr Ian Walsh, are greatly appreciated.

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Mechanical properties of HMA mixes using crumb-rubber modified binders

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ABSTRACT: This research aims at evaluating the effects of crumb rubber and shale-oil residue on some mechanical properties of HMA mixes. Eight HMA mixes were produced, using asphalt binders compounded with different proportions of crumb rubber and shale-oil residue, in order to evaluate the effect of these two modifiers on resilient modulus and tensile strength values. The overall results pointed out that both crumb rubber and shale-oil residue reduce resilient modulus, but oil shows a more expressive effect. Both crumb rubber and shale-oil residue reduce tensile strength with similar intensity. Response-surface analysis indicated that crumb-rubber modified binders formulated with smaller proportions of crumb rubber and shale-oil residue lead to HMA mixtures of higher tensile strength values.

KEY WORDS: crumb-rubber modified binder, shale-oil residue, resilient modulus, tensile strength, statistical analysis

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1. Introduction

Understanding how modifiers affect physical properties of asphalt binders and HMA mixes is the key-point to pick out adequate modifier proportions. The technique of experiments with mixtures (Cornell, 2002) has been used with success to formulate crumb-rubber modified binders (Faxina *et al*., 2006a; Faxina *et al*., 2006b; Faxina *et al*., 2008a; Faxina *et al*., 2008b), likewise the applications in industries of medicines, food and others, when the focus is to model the effects that component proportions have on some properties. Modeling the effects that modifier proportions have on properties of interest for paving industry allows one to predict new values for non-tested conditions, avoiding extra labour to prepare samples in specific settings and facilitating the development of researches on new materials.

Incorporating crumb rubber to asphalt binder makes the asphalt binder stiffer and more elastic, what can make HMA mixes more resistant to rutting and fatigue and thermal crackings. Although an asphalt binder of higher performance is obtained, workability is usually reduced. Enhancement of asphalt binder performance is directly related to rubber concentration, but only until a certain percentage, once viscosity at mixing and compaction temperatures can reach impracticable levels.

Adding aromatic oils to crumb-rubber modified binders is a practical alternative to reduce viscosity during mixing and compaction operations of HMA mixes, improving workability. On the other hand, aromatic oils bring the asphalt binder consistence down at room temperatures, what may damage its quality and, as a consequence, drop the performance of HMA mixes off. Although aromatic oils are volatile, only a small amount of oil volatilizes during construction and asphalt layer lifespan. Selecting appropriate proportions of extender oil is crucial in obtaining crumb-rubber modified binders of good quality.

As an aromatic oil, shale-oil residue can play the role of an extender oil. But appropriate proportions of this modifier must be chosen, depending on rubber content and asphalt-binder grade, in order to obtain crumb-rubber modified binders which combine workability and good mechanical response. Adding extender oils to asphalt binder reduces to a typical formulation problem.

This research aims at evaluating the effects that the variation of the proportions of crumb rubber and shale-oil residue, used as an extender oil, have on resilient modulus and tensile strength of HMA mixes. Eight asphalt binders were used to produce specimens that were submitted to resilient modulus and tensile strength tests. Regression models were fitted and, based on these models, response-trace plots and response-surface plots were generated, in order to evaluate both principal and interaction effects among asphalt binder, crumb rubber and shale-oil residue.

2. Effects of crumb rubber and extender oil on resilient modulus and tensile strength

Epps (1994) pointed out that HMA mixtures containing rubber from discarded tires can show resilient modulus higher or smaller than that obtained from asphalt mixtures compounded with conventional asphalt binder, but typically smaller values are obtained when crumb-rubber modified binders are used. Ayres and Witczak (1995) verified that asphalt mixtures containing crumb-rubber modified binder show resilient modulus values from 10 to 20% smaller than that obtained from the reference mixture (AC 20). These authors pointed out that resilient modulus values reduce as rubber proportion increases.

Leite *et al*. (2000) concluded that HMA mixtures containing crumb-rubber modified binder have shown a reduction of 50% in resilient modulus compared to the reference mixture (AC 20). Faxina and Soria (2003) observed reduction of resilient modulus of HMA mixtures containing crumb-rubber modified binder and shale-oil residue, showing that addition of these modifiers makes asphalt mixtures more flexible. Cordeiro (2006) has come to conclusions similar to those obtained by the last two authors. Lemes (2004) concluded that the resilient modulus of asphaltic mixtures increases when mixtures are submitted to short-term accelerated aging.

Faxina and Soria (2003) showed that tensile strength of HMA mixes with crumb-rubber modified binder and shale-oil, at 25℃, reduces expressively in comparison to the reference mixture (approximately 50%). Pinheiro *et al*. (2003) also obtained tensile strength values 40% smaller when crumb-rubber modified binder was used. Other researches performed in Brazil (Momm and Salini, 2000; Specht *et al*., 2003) also observed reduction of tensile strength of HMA mixtures containing crumb-rubber modified binder. Hanson *et al*. (1994) got similar tensile strength values for asphalt mixtures compacted in laboratory and cores extracted from the road.

Bertollo *et al*. (2003) also concluded that tensile strength reduces as rubber proportion increases and pointed out that rubber gradation, as used in their research, does not affect tensile strength values. On the other hand, Leite *et al*. (2000) obtained tensile strength values for an asphalt mixture using crumb-rubber modified binder 26% higher than the tensile strength of the reference mixture (AC 20).

3. Design of the experiment

The HMA mixtures were produced with asphalt binders formulated according to an experiment designed with the statistical tool of experiments with mixtures (Cornell 2002), taking into account restrictions in component proportions. Denoting x_1 as asphalt proportion, x_2 as rubber proportion and x_3 as oil proportion, restrictions imposed in percentage are: $68 \leq x_1 \leq 100$, $0 \leq x_2 \leq 18$ and $0 \leq x_3 \leq 14$. The following restrictions also must be met: $x_1 + x_2 + x_3 = 100$ and $x_1, x_2, x_3 > 0$. Figure 1 shows the simplex and the sub-region where the selected mixtures are indicated. Design mixtures are: 100-0-0, 93-0-7, 86-0-14, 91-9-0, 77-9-14, 82-18-0, 75-18-7 and 68-18-14, where the numbers represent, from left to right, the proportions of asphalt binder, crumb rubber and shale-oil residue.

Mineral aggregate is not considered a fourth component, what would make the experiment much more complex. As the proportion of mineral aggregate in a specimen, for a certain binder content, does not change and each model is defined to binder contents that are controlled, it is supposed that the proportions of mineral aggregate and asphalt binder are constant for the eight asphalt mixtures, for each binder content. Due to this assumption, it is possible to assess the effect of varying the proportions of asphalt, rubber and oil on resilient modulus and tensile strength, supposing that the effects on these properties are due exclusively to the variations of the proportions of the three components.

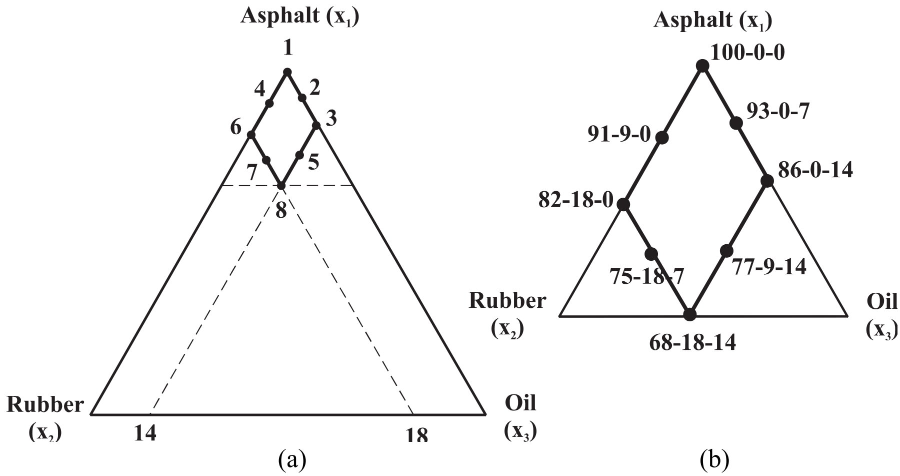

Figure 1. *(a) Simplex, restrictions and selected mixtures and (b) detailed view of the constrained region with selected mixtures*

4. Materials

The crumb rubber used is a commercial product obtained from tires of heavy vehicles (tractors and trucks). Its particle-size distribution is shown in Table 1. Thermogravimetry analysis indicated the following composition: 64.3% elastomers and plasticizers, 31.4% carbon black and 4.3% inorganic materials. Base asphalt binder (AC-40) is graded as a PG 70-10. A shale-oil residue type AR-5 according to ASTM D4552 was used. Mineral aggregate was obtained from crushed basaltic rock. Aggregate gradation is one that coincides with the middle of the specification "C" for dense-graded mixes of the paving handbook of the Sao Paulo State Highway Department. Aggregate composition was obtained by separating mineral aggregate in fractions correspondent to each specification sieve with posterior mixing. Los Angeles abrasion is 11% and apparent density of the mineral composition is 2,873 kg/m³.

Table 1. *Crumb rubber particle-size distribution*

sieve opening, mm	1.19	0.59	0.42	0.297	0.175	0.150	0.074
% passed	100.00	99.24	59.74	44.99	15.84	11.31	2.46

5. Experimental method

Laboratory tasks were divided in three steps: preparation of asphalt binders, specimen compaction and acquisition of resilient modulus and tensile strength data. Eight asphalt binders were prepared, according to the design of the experiment, and, subsequently, Marshall specimens were produced in five binder contents: 4.5, 5.0, 5.5, 6.0 and 6.5%. Effective specific gravity of HMA mixtures was measured according to Rice method (ASTM D 2041-00) and the apparent specific gravity of aggregate composition was obtained by weighing the apparent specific gravity of fine and coarse aggregates, measured according to protocols ASTM C 128-93 and ASTM C 127-91. Coarse aggregate is that retained in the 4.8mm sieve and fine aggregate is the one that passes the 4.8 mm sieve. As the mineral aggregate showed small asphalt absorption, this variable was not taken into account.

5.1. Preparation of crumb-rubber modified binders

Mixtures containing only asphalt binder and crumb rubber and those mixtures containing asphalt binder, crumb rubber and shale-oil residue were prepared in a high-shear mixer (Silverson model L4RT) at 4,000 rpm, at 170℃, during 90 minutes. Mixtures containing only asphalt binder and shale-oil residue were prepared in a low shear mixer (Fisatom model 722D), at 400 rpm and 135℃ during 25 minutes. Seven formulations were prepared, to which the reference asphalt binder (AC 40) is added, totalizing eight samples.

5.2. Compaction of Marshall specimens

Mineral aggregate and asphalt binder were mixed in an industrial mixer adapted to this aim. The amount of 1,200 grams was placed in the oven during 2 hours at compaction temperature. Seven specimens were compacted for each binder content: 4.5, 5.0, 5.5, 6.0 and 6.5%. Table 2 shows mixing and compaction temperatures.

Table 2. *Mixing and compaction temperatures (°C)*

asphalt binder	1	4	6	2	3	7	5	8
mixing	158	165	180	150	143	175	155	165
compaction	148	155	170	140	132	165	145	155

5.3.Acquisition of resilient modulus and tensile strength data

Resilient modulus values were obtained according to the AASHTO TP 31 protocol and tensile strength values were obtained according to ASTM D4123 protocol, at 25℃. For resilient modulus tests, one hundred loading cycles of 1s were applied by a pneumatic equipment. Each pulse is divided in a loading period of 0.1s and a resting period of 0.9s. Resilient modulus is obtained using the total displacement recovered during the resting period. Total displacement is calculated as the difference between the maximum displacement of the loading cycle and the displacement registered at the end of the cycle. Seven replicates were tested for eight asphalt mixtures with five binder contents, totalizing 280 measurements.

Tensile strength values were determined through the indirect tensile test using a pneumatic equipment. Specimens were subjected to a monotonic loading until failure, when the load is measured. Tensile strength values were calculated based on failure loading and sample geometry. Two replicates were tested for eight asphalt mixtures with five binder contents, totalizing 80 measurements.

6. Analysis procedure

The statistical package Minitab 15.1 has been used to run the statistical regression. Residual analysis has been performed through visual inspection, in order to check trends in residuals, and through the Anderson-Darling test, to check normal distribution of residual. Multicollinearity effects, common in experimental regions highly constrained, has not been considered in this research, as modeling is focused exclusively on prediction. The model used is called special cubic model, with the general format shown in equation 1.

$$\eta = \sum_{i=1}^{q} \beta_i^0 x_i + \sum_{i<j}\sum^{q} \beta_{ij}^0 x_i x_j + \beta_{ijk}^0 x_i x_j x_k \tag{1}$$

with i, j, k = 1, 2, ... q, where q is the number of components, β_i, β_{ij} and β_{ijk} are the coefficient estimates for the terms x_i, x_{ij} and x_{ijk}, respectively, with x_i, x_j and x_k corresponding to the proportions of asphalt, rubber and oil, respectively.

Statistical analysis is based on the evaluation of component effect plots, called response trace plots, and of response surface plots. A response trace plot depicts the effect that each component has on the response, in accordance with the fitted model. This kind of plot indicates the effect of varying the proportion of one of the components, keeping the ratio between the other two constant. This means that when analyzing the effect of varying rubber proportion, for instance, the proportions of asphalt binder and shale-oil residue also vary but the ratio between these two components is kept constant. Each line in the plot depicts the effect of changing the proportion of each component along an imaginary line (direction) linking the reference blend (usually the centroid of the simplex) to the vertex and the reference blend to the side opposite to the vertex.

It is important to understand that the component effect analysis depicted by this kind of plot is only one among several possibilities of analysis (several directions). The direction chosen in this analysis procedure corresponds to the line linking the vertex to the opposite side of the simplex, passing through the reference blend, that in this case is the centroid of the simplex shown in Figure 1. This kind of plot can be used to identify which component is more influential on response. The horizontal amplitude indicates in which ranges the component proportions vary, taking account of the limits indicated in the experimental design, starting from the reference blend to +50% of the range of variation and to -50% of this range. The vertical amplitude indicates the intensity of the effect of changing the component proportion on the monitored property. An easier way of evaluating the effects is to look at the percentage of each component as increasing from left to right, in its respective range of variation.

While analyzing a response trace plot, one should keep in mind that these results are obtained along a certain direction within the simplex. Additional analyses are possible in other directions. This kind of graph represents a statistical artifact in order to try to understand how each component affects response individually, as it is not possible to vary the proportion of one of the components keeping the proportions of the others constant. In practical terms, the three components act together and their effects on response depend on their interaction effects. These interaction effects depend on the component proportions and are depicted in a more effective and realistic manner by response surfaces.

When interpreting a response trace plot, it is important to keep in mind that: (a) all components are interpreted relatively to the reference blend; (b) components with the greatest effect on response will show the steepest response traces; (c) components with larger ranges (upper bound – lower bound) will have longer response traces, components with smaller ranges will have shorter response traces; (d) the total effect of a component depends on both the range of the component and the steepness of its response trace, the total effect is defined as the difference in the response between the effect direction point at which the component is at its upper bound and the effect direction point at which the component is at its lower bound; (e) components with approximately horizontal response traces, with respect to the reference blend, have virtually no effect on response; (f) components with similar response traces will have similar effects on response; (g) components do not have effects completely isolated: to change the proportion of one of the components implies to change the proportion of at least one of the complementary components.

Response surface plots depict how a response is related to mixture components, based on the fitted model. In this kind of plot, the surface is seen as a bidimensional plan in which all points that represents the same response value are connected to produce contour lines. Response surface plots are useful to select ideal values for a property and the correspondent ideal mixtures. Therefore, response surface plots represent an effective tool to delimitate regions where shale-oil residue could be used with rubber, working as an extender oil.

It is important to remember that the models fitted in this research are restricted to the type of asphalt binder, crumb rubber and shale-oil residue used, to the levels of the process variables and to the experimental conditions under which mixtures have been prepared. The fitted models are not intended to be applied to either any kind of asphalt binder, crumb rubber and shale-oil residue or any level of process variables different from the levels used here.

7. Findings

Table 3 shows the fitted models relative to resilient modulus and tensile strength based on which response trace plots and contour plots were generated. Figures from 2 to 6 depict the response trace plots and Figures from 7 to 11 show the response trace plots for resilient modulus. Figures from 12 to 16 present the response trace plots and Figures from 17 to 21 depict the contour plots for tensile strength.

Table 3. *Fitted models for resilient modulus (RM) and tensile strength (TS)*

property	**regression model**	**R^2 (%)**	**n**
RM 4.5%	$= 15{,}100x_1 + 7{,}893x_2 - 33{,}538x_3$	64.5	55
RM 5.0%	$= 16{,}709x_1 - 228{,}251x_2 - 54{,}181x_3 + 256{,}188x_1x_2 - 829{,}002x_2x_3 + {}$ $+ 1{,}837{,}430x_1x_2x_3$	96.7	54
RM 5.5%	$= 14{,}988x_1 - 12{,}9848x_2 - 14{,}5248x_3 + 161{,}236x_1x_2 - 217{,}952x_1x_3$	90.7	55
RM 6.0%	$= 14{,}758x_1 - 96{,}283x_2 - 63{,}478x_3 + 112{,}993x_1x_2 + 541{,}389x_1x_2x_3$	95.2	53
RM 6.5%	$= 12{,}304x_1 + 150{,}432x_2 + 217{,}702x_3 + 180{,}505x_1x_2 - 302{,}847x_1x_3$	97.5	51
TS 4.5%	$= 2.448x_1 + 0.543x_2 - 2.756x_3$	58.5	16

property	regression model	R^2 (%)	n
TS 5.0%	$= 2.716x_1 + 0.013x_2 - 2.984x_3$	66.0	16
TS 5.5%	$= 2.865x_1 + 0.311x_2 - 4.153x_3$	82.2	16
TS 6.0%	$= 2.818x_1 + 3.129x_2 - 7.120x_3$	88.1	16
TS 6.5%	$= 2.620x_1 - 23.280x_2 - 6.910x_3 + 27.620x_1x_2 + 71.280x_1x_2x_3$	97.1	16

x_1: asphalt proportion; x_2: rubber proportion; x_3: oil proportion; n: number of specimens

The response trace plot for the 4.5% asphalt content (Figure 2) shows that both rubber and oil reduce resilient modulus, but oil has a more intense effect. Rubber brings resilient modulus down at a rate of 35MPa to each 1% rubber added and oil reduces it at a rate of 477MPa/1%. Figure 3, relative to 5.0% asphalt content, depicts that rubber has a parabolic effect: resilient modulus increases for rubber from 0 to 8% and decreases from 8 to 18%.

For the asphalt contents of 5.5, 6.0, and 6.5%, rubber effect is also parabolic with maximum point around the reference mixture (84-9-7). For the asphalt contents of 5.5% (Figure 4) and 6.5% (Figure 6), resilient modulus values increase for rubber from 0 to 8%. For the 6.0% asphalt content (Figure 5), the maximum point occurs near 11% rubber. The overall results indicate that rubber tends to reduce resilient modulus values when intermediate to high rubber proportions are used, for instance, superior to 8%. When rubber proportions from small to intermediate are used, rubber tends to increase resilient modulus. This influence seems to be unexpected, but it could be interpreted as a result of interaction effects among components.

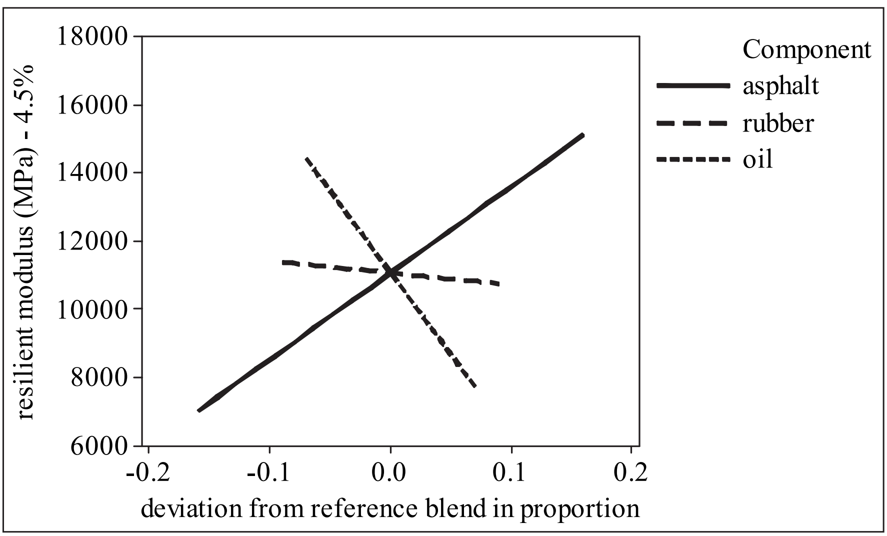

Figure 2. *Response trace plot of resilient modulus for binder content of 4.5%*

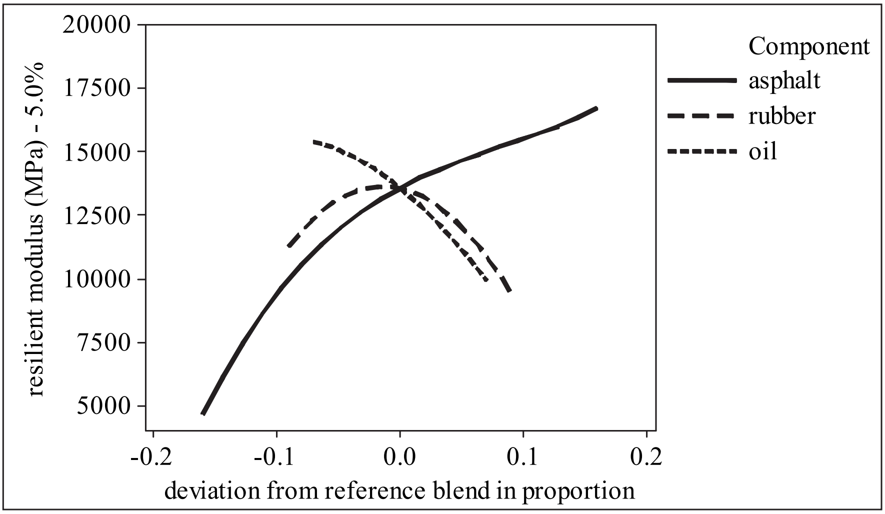

Figure 3. *Response trace plot of resilient modulus for binder content of 5.0%*

Figure 3 shows that oil acts reducing resilient modulus in an approximately linear way and that its effect is less expressive for low concentrations. Oil reduces resilient modulus at a rate of 390 MPa per 1% oil added. This linear tendency is kept for the other asphalt contents, but with some particularities. For the 5.5 and 6.5% asphalt contents (Figures 4 and 6), oil effect on resilient modulus is unexpressive for high proportions. For the 6.0% asphalt content (Figure 5), oil effect is linear for all concentrations. For the 5.5, 6.0 and 6.5% asphalt contents, rubber reduces resilient modulus at the following rates: 512, 507 and 433 MPa/1% oil added.

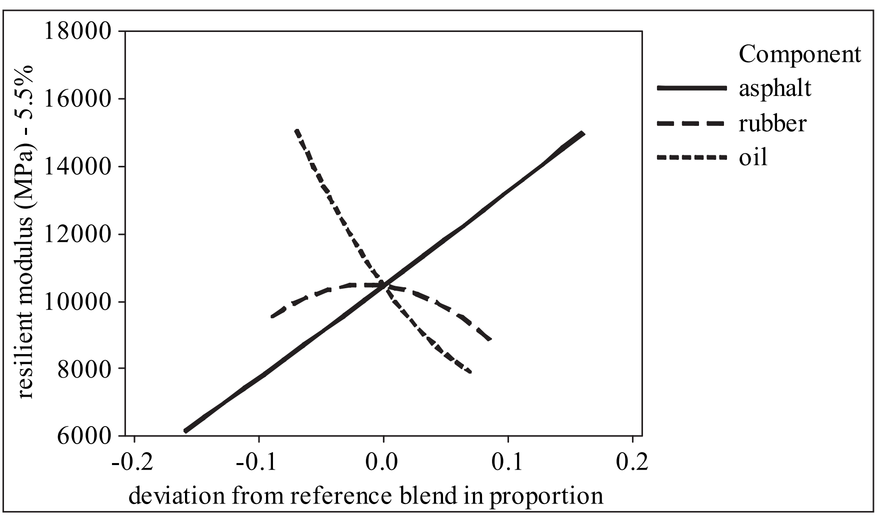

Figure 4. *Response trace plot of resilient modulus for binder content of 5.5%*

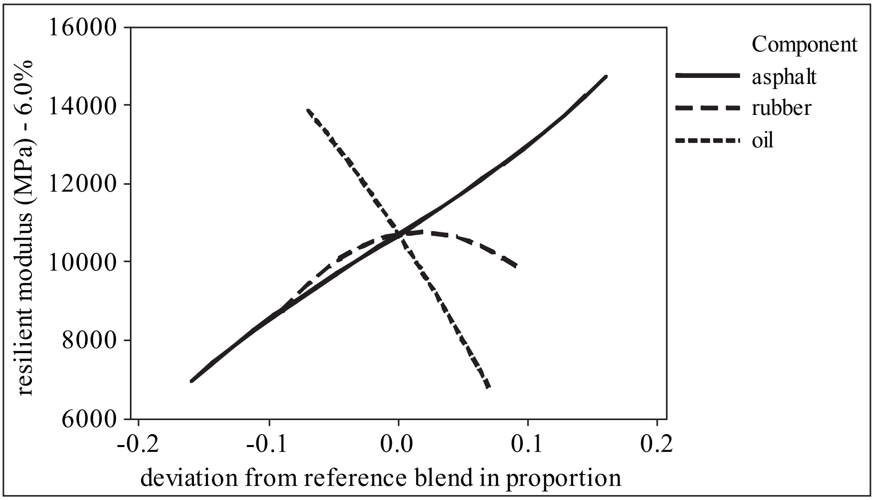

Figure 5. *Response trace plot of resilient modulus for binder content of 6.0%*

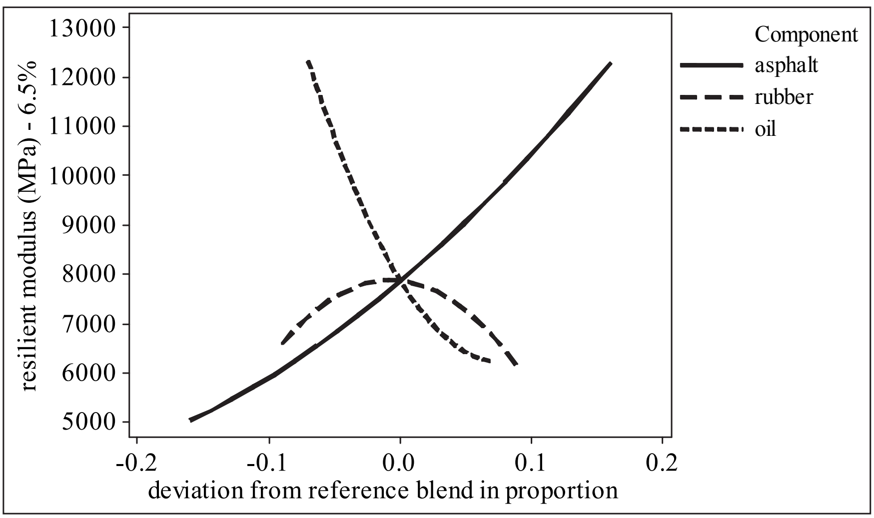

Figure 6. *Response trace plot of resilient modulus for binder content of 6.5%*

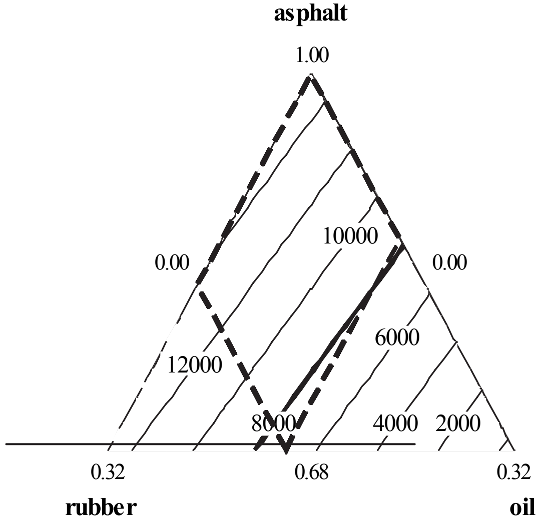

Figure 7. *Contour plot of resilient modulus for binder content of 4.5%*

The linear model obtained for resilient modulus at 4.5% binder content generates linear level curves (Figure 7). This contour plot indicates a small rubber effect. Higher resilient modulus values are obtained when small proportions of rubber and oil are used (mixtures at the top of the experimental region) and smaller values are obtained when high proportions of rubber and oil are used (mixtures at the bottom of the experimental region).

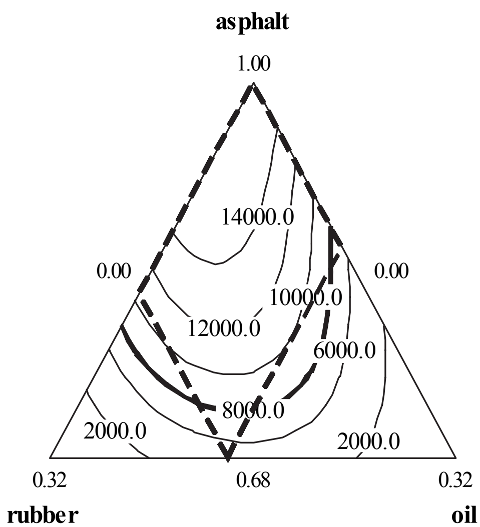

Figure 8. *Contour plot of resilient modulus for binder content of 5.0%*

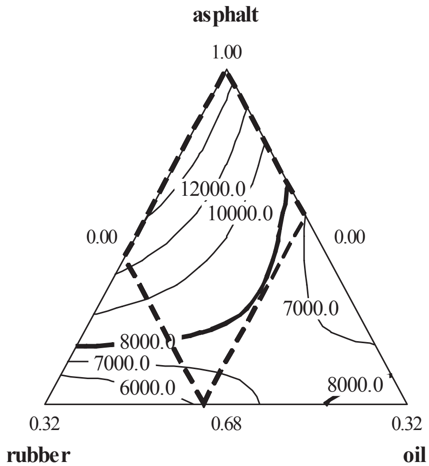

Figure 9. *Contour plot of resilient modulus for binder content of 5.5%*

Figure 8 (contour plot for the 5.0% binder content) points out that higher resilient modulus values are obtained with low to intermediate rubber concentrations and with low oil concentrations (mixtures at the top of the experimental region). This pattern repeats for the 5.5, 6.0 and 6.5% asphalt contents (Figures 9 to 11). Asphalt mixtures with lower resilient modulus values are obtained with high concentrations of rubber and oil for the 5.0, 5.5 and 6.5% asphalt contents (mixtures at the bottom of the experimental region). Lower resilient modulus values are also obtained for low rubber proportions and high oil proportions for the 6.5% asphalt content (mixtures at the right of the experimental region). For the 6.0% asphalt content, smaller resilient modulus values are obtained only with small rubber proportions and high oil proportions.

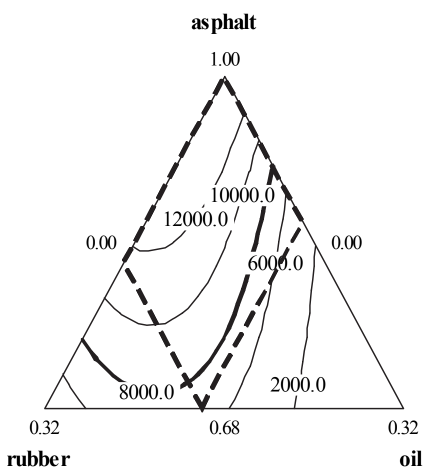

Figure 10. *Contour plot of resilient modulus for binder content of 6.0%*

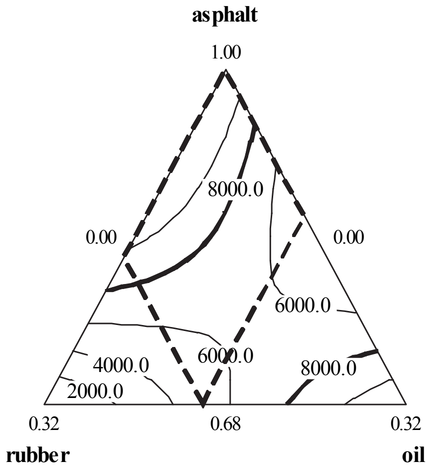

Figure 11. *Contour plot of resilient modulus for binder content of 6.5%*

It seems natural to associate the different patterns depicted by component effects plots to the influence of binder content. On the other hand, it is important to keep in mind that these plots are generated from models whose coefficients are fitted using available samples. Variability inherent to data determinates if a certain coefficient will be statistically significant or not, at a 95% confident level, leading to models in which not all terms are present. The different patterns depicted by these plots are influenced not only by asphalt content but also by model terms. Hence, asphalt content can not be pointed out as the unique factor influencing resilient modulus behavior.

As an example, the maximum resilient modulus value of 8,000MPa was chosen in order to find which compositions meet this requirement. Table 4 indicates rubber and oil proportions, for the five binder contents, in order to guarantee that HMA mixtures show resilient modulus higher than 8,000MPa. A visual inspection of response surfaces reveals that it is not easy to decide which these ranges are. Then, ranges reported here are an approximation. The overall results indicate that, in order to meet a resilient modulus value of 8,000 MPa, only asphalt binders containing high rubber and oil proportions are acceptable, for binder contents of 4.5, 5.0 and 5.5%, and only asphalt binders with any rubber proportion and intermediate to high oil proportions are acceptable, for binder contents of 6.0 and 6.5%. It is obvious that these ranges are tied to the arbitrated value for resilient modulus. Other limits for resilient modulus would imply in other ranges.

Table 4. *Proportion ranges for resilient modulus superior to 8,000MPa*

binder content	**proportions**
4.5	rubber from 10 to 18% and oil from 12 to 14%
5.0	rubber from 16 to 18% and oil from 12 to 14%
5.5	rubber from 0 to 3% and oil from 12 to 14% and rubber from 13 to 18% and oil from 11 to 14%
6.0	rubber between 0 e 18% and oil from 12 to 14%
6.5	rubber between 0 e 18% and oil from 6 to 14%

Figure 12 depicts the component effects on tensile strength for the 4.5% binder content. Both crumb rubber and shale-oil residue contribute to reduce tensile strength, what can be seen as a disadvantage. The increase of rubber proportion implies in the reduction of asphalt available to bind the mineral particles. With less binding material available, tensile strength reduces naturally. Crumb rubber does not play the role of binding mineral particles, but it can affect consistency of available asphalt, once rubber particles swells during processing using light fractions from asphalt binder.

Another aspect to be taken into account is the susceptibility of tensile strength to binder content: generally, the highest tensile strength values lie around optimum binder content and smaller values are obtained for binder contents above and below the optimum binder content. In fact, tensile strength is related to film thickness formed at aggregate surface and so thicker films would imply in higher tensile strength values until a certain binder content, as asphalt binder in excess would damage tensile strength.

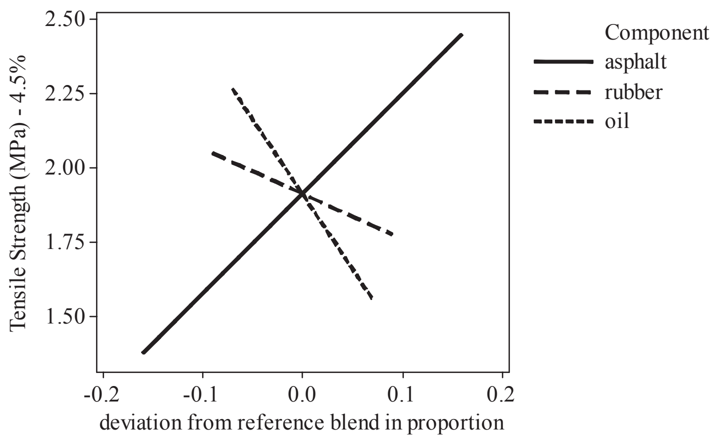

Figure 12. *Response trace plot of tensile strength for binder content of 4.5%*

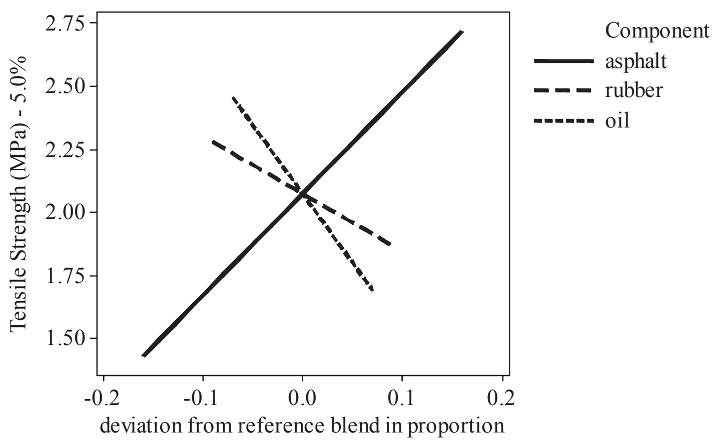

Figure 13. *Response trace plot of tensile strength for binder content of 5.0%*

As 4.5% is a small binder content, taking into account the particularities of the materials used here, the quantity of binding material available is small, what may lead to smaller tensile strength values. By its time, the increase of oil proportion also reduces the asphalt available, reducing the quantity of binding material. But oil plays another undesirable role: it makes the available asphalt binder less consistent, reducing tensile strength as a consequence.

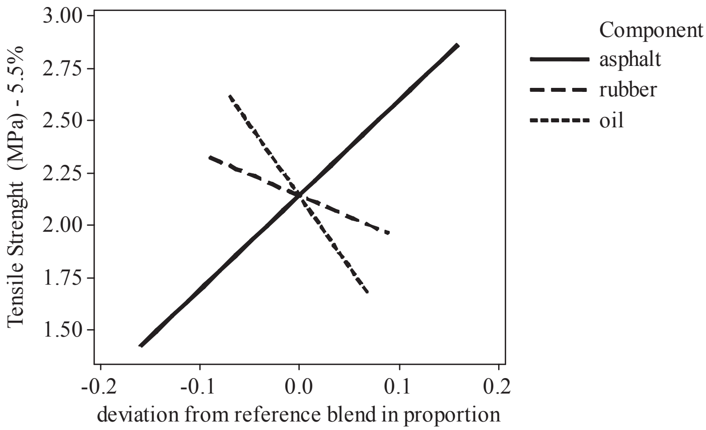

Figure 14. *Response trace plot of tensile strength for binder content of 5.5%*

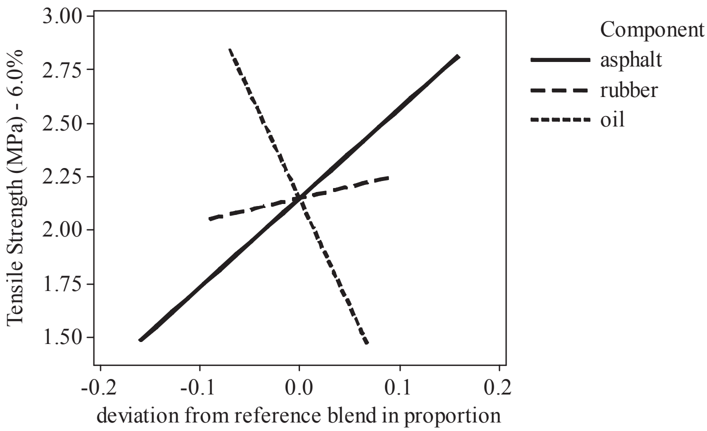

Figure 15. *Response trace plot of resilient modulus for binder content of 6.0%*

Figure 12 also shows that rubber and oil reduce tensile strength with different intensities. Oil has a more intense effect than rubber: oil reduces tensile strength at a rate of 0.05MPa per 1% oil added and rubber reduces it at a rate of 0.02MPa/1%. The same pattern is observed in Figures 13 and 14. Both rubber and oil reduce tensile strength and oil preserves its tendency of reducing this property with more intensity than rubber. For the 5.0% binder content (Figure

13), rubber reduces tensile strength at a rate of 0.02MPa/1% and oil at a rate of 0.05MPa/1%. For the 5.5% binder content (Figure 14), rubber reduces tensile strength at a rate of 0.02MPa/1% and oil at a rate of 0.07MPa/1%.

Rates obtained for binder contents from 4.5 to 5.5% seem to indicate that asphalt content does not interfere either in the component effect or the component intensity. But it is worthy to mention that the quality of the fitted models can interfere in these trends in a certain extent. Models for 4.5 and 5.0% binder content showed a coefficient of determination relatively low, denoting a relatively poor fitting. Looking at the coefficients of determination of the five models, one can observe that there is a high correlation between quality of regression and binder content, although it would be dangerous to conclude that this correlation has a true cause-effect relationship.

A slightly different pattern is observed for the 6.0% binder content, as depicted by Figure 15. The linearity of effects is kept, as seen in the previous graphs, but two main changes are observed: crumb rubber increases tensile strength slightly, instead of reducing it as shown in other binder contents, and oil effect is more intense. Rubber increases tensile strength at a rate of 0.01MPa/1% and oil reduces it at a rate of 0.09MPa/1%. This could be an explanation to the controversy of literature about the effect rubber has on tensile strength: in fact rubber may increase tensile strength but it depends on binder content.

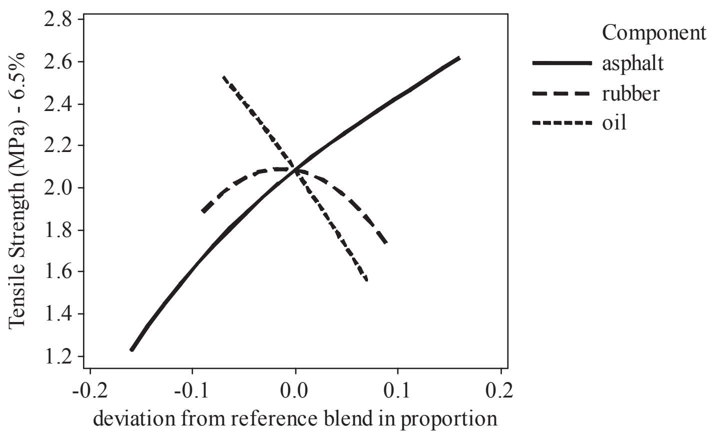

Figure 16. *Response trace plot of tensile strength for binder content of 6.5%*

A completely different pattern is observed in Figure 16 for tensile strength. In this case, the regression is parabolic, instead of linear as in the previous binder contents. Rubber has a parabolic effect, with maximum point near the reference blend (84-9-7), what means that rubber increases tensile strength from 0 to 9% and decreases it from 9 to 18%. This helps to understand that rubber can increase tensile strength depending not only on binder content but also on rubber proportion. For small proportions, rubber increases tensile strength, but for intermediate to high proportions, rubber decreases it. But this conclusion is valid only for the 6.5% binder content. For the 6.5% binder content, oil maintains its linear effect on tensile strength, reducing it at a rate of 0.07MPa/1%.

Although response trace plots clearly describe component effects, they are generated from a statistical model that tries to interpretate nature. But one question still remains: is crumb rubber really capable of increasing tensile strength taking into account that it can not contribute in terms of binding potential? One would have to assume that probably there is another factor influencing tensile strength increase rather than rubber proportion.

Optimum binder content may be this factor. As tensile strength generally decreases when binder content is above the optimum point due to the excessive quantity of asphalt binder, however much rubber is added to the composition the smaller the quantity of asphalt binder available. Thus, it is not a question of rubber increasing tensile strength for a binder content above the optimum point. But, rather, it is a question of a smaller quantity of asphalt binder presented in a formulation with a high concentration of rubber. Higher rubber proportion, and consequently smaller asphalt proportions, imply in higher tensile strength values.

Table 5 shows the intensity of the effects of rubber and oil of reducing tensile strength at a rate of MPa per 1% modifier added. Negative indicate reduction and positive indicate increase. Results indicate that binder content does not affect the intensity of rubber effect on tensile strength expressively and that binder content has a slight effect on oil effect. Oil effect increases with binder content for binder contents from 4.5 to 6.0%.

Table 5. *Intensity of component effects on tensile strength (MPa/1% modifier added)*

	binder content (%)				
component	4.5	5.0	5.5	6.0	6.5
rubber	- 0.02	- 0.02	- 0.02	+ 0.01	-
oil	- 0.05	- 0.05	- 0.07	- 0.09	- 0.07

Figures from 17 to 20 are generated from linear models, so level curves are also linear. Figures from 17 to 19 show a similar pattern: HMA mixtures using asphalt binders with high proportions of rubber and oil present smaller tensile strength values and HMA mixtures using asphalt binders with small proportions of rubber and oil show higher tensile strength values. Figure 20 also shows linear level curves, but its pattern is slightly different from the previous figures. Level curves are parallel to the side asphalt-rubber, indicating that oil effect is predominant and rubber effect is unexpressive. Figure 21 shows nonlinear level curves, but the same pattern is kept: HMA mixtures using asphalt binders with high proportions of rubber and oil present smaller tensile strength values and HMA mixtures using asphalt binders with small proportions of rubber and oil show higher tensile strength values.

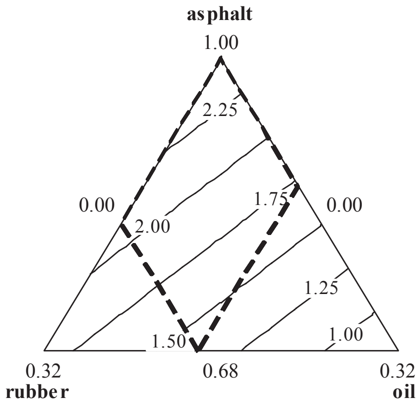

Figure 17. *Contour plot of tensile strength for binder content of 4.5%*

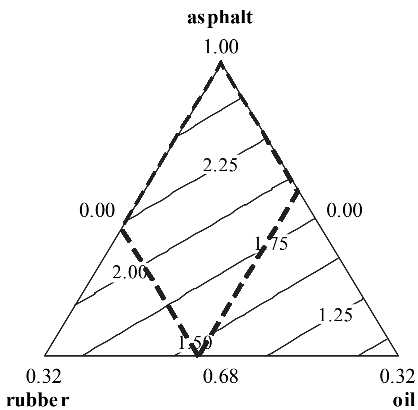

Figure 18. *Contour plot of tensile strength for binder content of 5.0%*

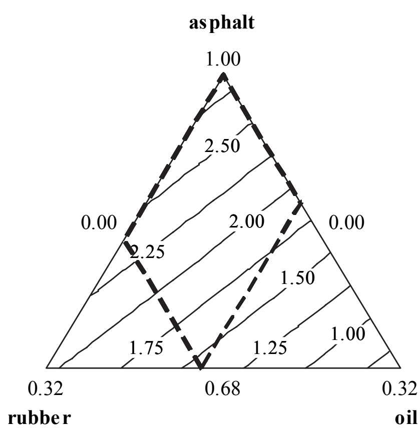

Figure 19. *Contour plot of tensile strength for binder content of 5.5%*

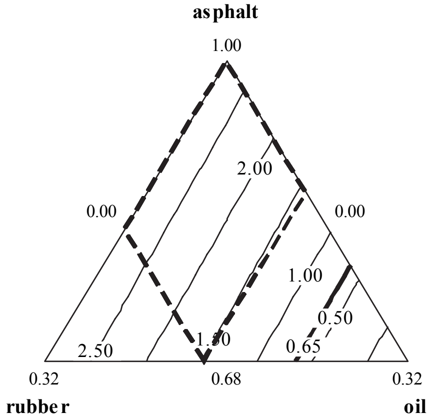

Figure 20. *Contour plot of tensile strength for binder content of 6.0%*

Brazilian specifications require a minimum tensile strength value of 0.65 MPa for dense-graded asphalt mixes. As response surfaces point out, all formulations within the dashed region lead to HMA mixtures with tensile strength values superior to that required by specification. These results drive to the conclusion that asphalt binders formulated with any rubber proportions from 0 to 18% and any oil proportion from 0 to 14% are acceptable in terms of tensile strength if the 0.65 MPa limit is chosen.

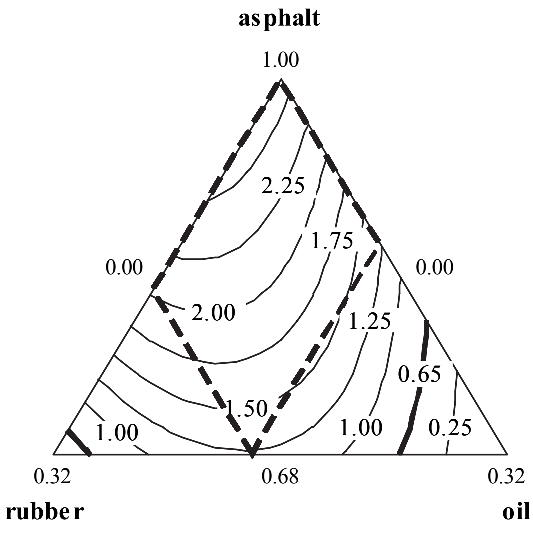

Figure 21. *Contour plot of tensile strength for binder content of 6.5%*

8. Conclusions

This research aimed at evaluating the effect of crumb rubber and shale-oil residue on resilient modulus and tensile strength values obtained at 25℃. Eight HMA mixtures were produced using asphalt binders compounded with different proportions of crumb rubber and shale-oil residue. Properties were monitored at five binder contents (4.5, 5.0, 5.5, 6.0 and 6.5%) and data were used to fit mixture models, based on which response trace plots and contour plots were generated.

From the response trace plot analysis, the following conclusions can be drawn:

- the overall effect of rubber on resilient modulus is parabolic, with maximum point around the reference mixture (84-9-7);

- when binder content is 5.0, 5.5 or 6.5%, rubber increased resilient modulus for proportions from 0 to 8% and reduced it for proportions above 8%; when binder content is 6.0%, rubber increased resilient modulus for proportions from 0 to 11% and reduced it for higher proportions; for 4.5% binder content rubber effect is linear, acting to reduce resilient modulus at an approximate rate of 35MPa per each 1% rubber added;

- rubber tends to reduce resilient modulus when intermediate to high proportions are used.

- rubber tends to increase resilient modulus when small to intermediate proportions are used;

- oil reduces resilient modulus linearly, at an approximate rate of 400 to 500 MPa per each 1% oil added, depending on binder content;

- oil effect is inexpressive on resilient modulus when small concentrations (for 5.0% asphalt binder) or high concentrations (for 5.5 and 6.5% binder content) are used;

- both rubber and oil reduce tensile strength for binder contents of 4.5, 5.0 and 5.5%; for 6.0% binder content, rubber increases tensile strength slightly and oil reduces it; for 6.5% binder content, oil reduces tensile strength and rubber shows a parabolic effect, increasing it for proportions from 0 to 9% and reducing it for proportions from 9 to 18%;

- for small binder contents (from 4.5 to 5.5%), rubber reduces tensile strength at an average rate of 0,02MPa/1%; oil reduces tensile strength at an average rate of 0.07 MPa/1%;

- binder content does not affect the intensity of rubber effect on tensile strength expressively and has a slight effect on oil effect: oil effect increases for binder contents from 4.5 to 6.0%;

- although literature reports that, in general, rubber reduces tensile strength, this research indicates that rubber can increase tensile strength, depending on binder content and rubber proportion: small to intermediate proportions of rubber combined with high binder contents lead to higher tensile strength values.

From the response surface analysis, the following conclusion can be drawn:

- in order to meet a resilient modulus value of 8,000 MPa, only asphalt binders containing high rubber and oil proportions are acceptable, for binder contents of 4.5, 5.0 and 5.5%, and only asphalt binders with any rubber proportion and intermediate to high oil proportions are acceptable, for binder contents of 6.0 and 6.5%;

- HMA mixtures using asphalt binders with high proportions of rubber and oil present smaller tensile strength values and HMA mixtures using asphalt binders with small proportions of rubber and oil show higher tensile strength values;

- all formulations within the experimental region (dashed region) lead to HMA mixtures with tensile strength values superior to that required by Brazilian specification (0.65 MPa minimum), then asphalt binders formulated with any rubber proportions from 0 to 18% and any oil proportion from 0 to 14% are acceptable in terms of tensile strength if this limit is taken into account.

Aknowlegments

The authors would like to thank SIX-Petrobras for supplying the shale-oil residue, Artgoma S. A. for supplying the crumb rubber, Capes for supplying a master-degree scholarship for the first author and Fapesp for supplying a doctoral scholarship to the second author.

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Fatigue-Related Properties of Crumb Rubber Modified Asphalt Binders

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ABSTRACT. The experimental investigation presented in this paper focuses on the characterization of the fatigue behavior in crumb rubber modified (CRM) asphalt binders. The main purpose was to identify and to describe the fatigue processes in CRM binders rather than quantify the expected improvement in fatigue life due to crumb rubber modification. The development of the complex modulus (G) in cyclic shear tests (time sweeps) was recorded under different loading conditions and the obtained trends were evaluated according to the available failure criteria. Using both stress-controlled and strain-controlled testing mode, the relations between the stress/strain amplitude and the number of cycle to failure (S-N curves) were established. The results allowed to define some dynamics related to the contribute of the crumb rubber modification in binder damage resistance with regard to the considered loading conditions. The role of the rheometrical parameters was also investigated and the influence of the loading mode on the damage characteristics of CRM binders was established.*

KEYWORDS: Fatigue, Damage Behaviour, Performance-Related Properties.

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1. Introduction

Background

With low and intermediate service temperatures, fracture propagation in asphalt pavements frequently derives from the cyclic accumulation of fatigue damage related to stresses lower than the amount required for the immediate failure. Different methods to evaluate fatigue in bitumen have been thus investigated since rheological analysis was introduced in pavement engineering. In the early ’90s, the development of the SHRP research program led to the definition of fatigue resistance in bitumen by measuring $G^{*}\cdot\sin\delta$ at 10 rad/s (Bahia and Anderson, 1995). The derived criteria were also consequently used to qualify the role of crumb rubber modifiers (CRM) in asphalt binders mechanical performances (Bahia and Davies, 1994). However, although the SHRP parameter is still used for technical specification, it was found inadequate for describing the real fatigue performance of modified bitumen in the field. At this regard, the study of Stuart and Mogawer (2002), dealing with the relation between the fatigue resistance of asphalt pavements and the SHRP fatigue parameter of asphalt binders, is representative.

At present, different approaches have been proposed to improve fatigue testing of bitumen and the concept of damage behavior was introduced by Bahia *et al*. (2001-a) to define specific properties and test procedures related to the conditions of incipient failure in bitumen. Moreover, the use of dynamic mechanical analysis (DMA) to evaluate fatigue properties of bitumen was also proposed as a part of the NCHRP project 9-10 (Bahia *et al*., 2001-b) for the characterization of modified asphalts in the Superpave mix design and a similar approach, still based on the DMA, was also successfully developed by Kim et al. (2002) to study both fatigue and healing potential in sand-asphalt mixtures and mastics. On the basis of these results it was well known and accepted that repeated shear loading of asphalt binders in DSR causes a loss in complex modulus. However, two different modes of failure have been observed: internal fatigue micro-cracking and plastic edge fracture (Anderson et al., 2001). This topic was then investigated by Bonnetti et al. (2002) and by Planche et al. (2004) and it was found that “real” or “true” fatigue actually occurs in time sweep tests only when the initial material stiffness exceeds a certain value, otherwise edge fracture phenomena is preponderant. Furthermore, the application of time sweep testing to investigate the effects of different fillers and external factors on fatigue behavior of bitumen and mastics was also recently done, yielding to an improvement of the reliability of the method (Soenen et al., 2000, Airey et al., 2006, Bocci et al., 2006, Santagata et al., 2008). As a consequence, the analysis of the contribution of crumb rubber and other modifiers on bitumen fatigue behavior can be done by using time sweep testing (Frantzis, 2003, Ajideh et al., 2006). On the other hand, the influence of the rheometrical parameters on the final result must be carefully outlined in order to attain to a reliable assessment of binder fatigue life in presence of non-conventional modifiers. In this context the present experience is developed, focusing on the application of the different testing procedures and the proposed failure criteria to the case of CRM asphalt binders in order to attain to the assessment of their behavior with regard to the considered loading conditions.

2. Research approach

2.1 Materials

The binders used in the experiment consist of traditional bitumen B50/70 (PG 64-22, pen = 76 dmm, TR&B = 49.5℃) subsequently modified in the laboratory at 16% and 20% CRM content referred to the weight of the bitumen. The two CRM binders were prepared using crumb rubber obtained by cryogenic process with maximun particles' diameter of 0.6 mm. Digestion time and mixing methodology are referred to as ASTM D-6114. All samples were not subjected to artificial ageing.

In table 1 the conventional properties of binders are reported. The ring and ball softening point procedure was done according to EN 1427 while the penetration at 25℃ was measured according to EN 1426.

Table 1. *Materials*

Binder	**Type**	**CRM content [%]**	**Penetration [dmm]**	**Softening point – TR&B [°C]**
CRM-0	Unmodified	0.0	77	49.0
CRM-16	CRM modified	16.0	26	79.9
CRM-20	CRM modified	20.0	24	88.9

The samples preparation for rheological measurements was referred to EN 12594. CRM binders were treated as modified binders, thus reheating and homogenization were carefully carried out at a controlled temperature (160℃ ± 5℃) in order to obtain reproducible results (Anderson et al., 2000). Special attention was then paid to the specimen thermal history and storage conditions (1 hour at 25℃ ± 0.5℃) because of their influence on rheological measurements (Soenen et al., 2005).

2.2 Experimental methods

The tests were carried out by means of a Dynamic Shear Rheometer (DSR). The temperature during the tests was controlled by means of a Peltier conditioning cell. An air-operating suspension system guaranteed a significant reduction in the friction between the moving parts of the rheometer. The selected measurement system is represented by the double plate configuration with 2.0 mm gap and 8.0 mm diameter according to expected materials' stiffness. The samples were placed on the bottom plate, squeezed out between the two plates and trimmed off from the edge of the plates using an hot spatula. Once these operations were carried out, the gap was set as required in order to guarantee the correct geometry of each samples. The test temperature was set with a maximum admitted deviation of ± 0.01℃ from the selected temperature during the whole experiment. Before each test the samples were subjected to a 30-minute thermal conditioning period. In order to avoid errors due to the instrument's sensitivity, the torque applied was higher than the minimum suggested by the instrument producer (min. torque = 0.5 μNm).

As mentioned above, the initial binders stiffness strongly influences the failure mode in time sweep tests. In order to ensure that true fatigue occurs during the tests, the initial binders stiffness should be in the region of 15 MPa and in general it must be greater than 5.0 MPa (Planche et al., 2004). Moreover it was also demonstrated that viscoelastic behavior of CRM binders is highly dependent on strain (Kim et al., 2001). A preliminary investigation was thus carried out in order to evaluate the binders stiffness and to check the extension of the linear viscoelastic region. According to the SHRP procedure and to the results of Marasteanu et al. (2000) and Airey et al. (2003) the extension of the linear viscoelastic domain was assessed by performing strain sweeps tests and the limit of the LVE response was identify as the strain amplitude where G* is decreased to 95% of its initial value.

Rheological testing for fatigue are performed with regard to different conditions. Strain-controlled tests were conducted for nominal mean strain amplitude from 1.0% to 4.0%. The stress-controlled tests were conducted for nominal mean stress amplitude from 250 kPa to 475 kPa. The loading frequency was equal to 10 Hz and the test temperature was kept constantly equal to 15.00℃ ± 0.01℃. Defining fatigue failure during laboratory testing is quite difficult and different conventional limit and criteria were proposed to define the number of cycle to failure (Ghuzlan and Carpenter, 2000). During this study, fatigue life is measured with regard to two different proposed criteria:

- fatigue life is the number of cycle (Nf) to reach 50% reduction in complex modulus (G*).

- fatigue life is the number of cycles (Np20) to achieve 20% deviation from the initial linear trend of DER (Dissipated Energy Ratio) and calculated as follows:

$$DER = \frac{\sum_{i=1}^{n} W_i}{W_n} \qquad \text{(Eq. 1)}$$

where Wi is the dissipated energy per cycle:

$$W_i = \pi \cdot \sigma_i \cdot \varepsilon_i \cdot \sin \delta_i \qquad \text{(Eq. 2)}$$

With regard to the DER criteria, the Relative Crack Propagation Amplitude (RCPA) parameter, recently proposed by Santagata et al. (2008) and defined as reported in equation 3, was also evaluated in this study to highlight the peculiar effects caused by the CRM.

$$RCPA = \frac{N_{MAX} - N_{p20}}{N_{MAX}} \qquad \text{(Eq. 3)}$$

In equation 3, NMAX is the number of cycle corresponding to the maximum value of DER (evaluated in stress-controlled mode) and Np20 is the conventional binder fatigue life measured as referred above.

3. Results and analysis

3.1 Linear viscoelastic properties

Results of the viscoelastic characterization are summarized in table 2, where the parameters G*·sinδ calculated at 10 rad/s and 15℃ are presented in order to identify the SHRP fatigue qualification of the binders. The equi-stiffness temperature relative to the value of 5.0 MPa for G* are also presented in order to verify the stiffness condition for 'true fatigue' measurement according to Anderson et al. (2001). Further data in table 2 regard the definition of the linear viscoelastic limits expressed in terms of stress and strain amplitude.

Table 2. *Linear viscoelastic limits, SHRP fatigue parameters at 15℃ and equi-stiffness temperatures (G* = 5.0 MPa).*

Binder	γLVE (10 Hz) [%]	τLVE (10 Hz) [kPa]	G*·sinδ @ 15°C (10 rad/s) [MPa]	T(G*=5.0 MPa) @ 10 Hz [°C]
CRM-0	1.76	423	3.69	21.7
CRM-16	1.24	225	6.28	24.6
CRM-20	0.98	133	5.04	23.0

The equi-stiffness temperatures are higher than 15℃ and very similar for all binders tested. Therefore, according to the literature findings reported above, it can be considered that the fatigue test performed at 15℃ are internal to the 'true fatigue' region. However, for the stress/strain amplitudes greater than the material relative τLVE and γLVE, specific effects related to non-linearity must be taken into consideration.

3.2 Fatigue analysis

3.2.1. Stress-controlled approach

Results of stress-controlled fatigue tests are reported in table 3 as the number of cycles to failure evaluated from both the 50% G* criterion and the DER criterion. Table 3 also includes the initial complex modulus G* and the initial dissipated energy per cycle Wi evaluated after 100 loading cycles from the start of the test. All results are means of almost two independent determinations.

Table 3. *Results of stress-controlled fatigue analysis (T = 15℃)*

Binder	Frequency [Hz]	Stress amplitude [kPa]	Initial complex modulus [MPa]	Initial dissipated energy [Pa]	N_f (50%G*) [-]	N_{p20} (DER) [-]
CRM-0	10	250	25.63	5837	$1.22 \cdot 10^5$	$1.06 \cdot 10^5$
	10	325	26.16	9837	$6.59 \cdot 10^4$	$5.82 \cdot 10^4$
	10	400	24.53	15702	$2.53 \cdot 10^4$	$2.23 \cdot 10^4$
	10	475	24.60	22050	$1.72 \cdot 10^4$	$1.52 \cdot 10^4$
CRM-16	10	250	19.32	5526	$7.06 \cdot 10^5$	$5.05 \cdot 10^5$
	10	325	19.07	9582	$3.14 \cdot 10^4$	$2.42 \cdot 10^4$
	10	400	16.70	17179	$6.32 \cdot 10^3$	$4.27 \cdot 10^3$
	10	475	15.83	29153	$2.10 \cdot 10^3$	$1.43 \cdot 10^3$
CRM-20	10	250	13.23	8071	$1.41 \cdot 10^5$	$9.23 \cdot 10^4$
	10	325	13.08	15116	$3.13 \cdot 10^4$	$2.19 \cdot 10^4$
	10	400	12.40	21961	$8.76 \cdot 10^3$	$6.22 \cdot 10^3$
	10	475	11.92	37856	$2.98 \cdot 10^3$	$2.08 \cdot 10^3$

According to the results of previous experiences (Frantzis, 2003, Ajideh et al., 2006), binder fatigue resistance as measured by stress-controlled rheological testing depends on the presence of CRM. In particular a reduced fatigue life is generally recorded for CRM-16 and CRM-20, especially for the higher stress amplitude used. This evidence can be firstly explained by considering that the presence of CRM softens the binder response in the temperature/frequency range investigated. As a consequence, for stress-controlled loading mode, CRM binders are subjected to more extensive deformations than the unmodified binder (Wi at 400 kPa change from 15702 Pa recorded for the unmodified binder to 21961 recorded for the CRM-20). Moreover, a higher sensitivity to the strain level is recorded, and generally observed (Kim et al., 2001), for the CRM binders. The consequent higher deformation levels, together with the higher strain dependency (non-linearity in shear deformation and consequent modulus reduction), can be firstly explain the reduced fatigue life measured on CRM binders at $\tau0 = 475$ kPa. However, a different assessment of the problem can be outlined by analysing data in a graphical form. At this regard, figure 1 depicts the DER trends obtained for binder CRM-0 and CRM-20 for a shear stress amplitude of 250 kPa. For the sake of brevity, data of CRM-16 are not presented, however very similar tends were obtained for binders CRM-16 and CRM-20.

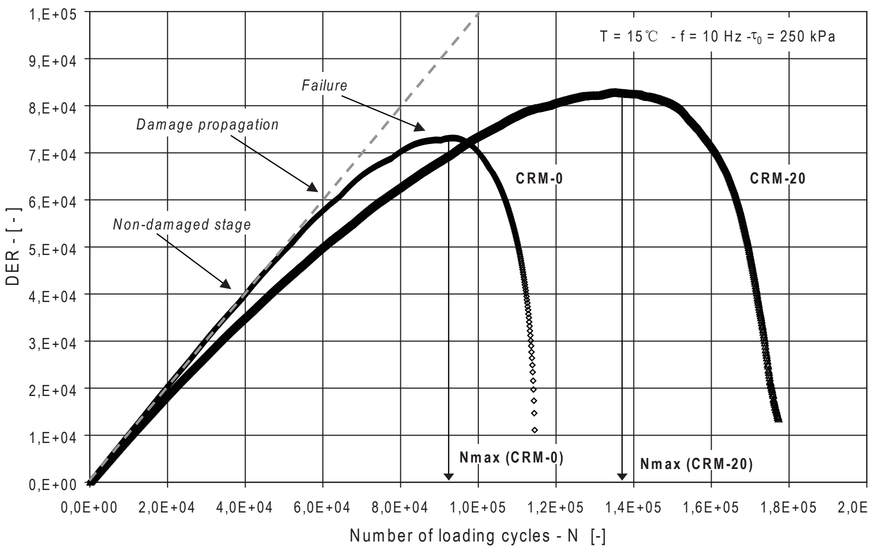

Figure 1. *Fatigue data from stress-controlled time sweep, 50%G* failure criterion. Binders CRM-0 and CRM-20 (σ0= 250 kPa).*

It is now immediately clear as the fatigue resistance of CRM binder can not be summarized by the parameter Np20 only. On the contrary, a more correct approach can be pointed out by involving the analysis of the whole fatigue behaviour. On the basis of the recorded trend, we can consider that for CRM binders, the damage propagation starts as the test begins, but its more 'ductile' behaviour leads to locate the complete failure threshold in correspondence of a very higher number of cycle. Differently, for the CRM-0, the region of the damage propagation cover a reduced number of cycle (less ductile behavior) and the failure occurs after a few cycles after the damage starts to propagate.

Substantially, for CRM-20 the detachment of the DER line from the DER=N line occurs after a few loading cycle. As a consequence, the 20% of DER variation from the linear trend, which define the cycle number Np20, is located in correspondence of a number of cycle very far from the complete failure (Nmax). For the unmodified binder CRM-0 the situation is different and Nmax is similar to Np20. Hence, if Nmax is used in stead of Np20, fatigue life of binders CRM-16 and CRM-20 increases up to fatigue life higher than those of CRM-0. A quantitative evaluation of the observation reported above can be offered by the parameter RCPA, which approaches zero for the unmodified binder only (table 4).

Table 4. *RCPA (T = 15℃)*

Binder	Frequency [Hz]	Stress amplitude [kPa]	RCPA [-]
CRM-0	10	250	0.01
	10	325	-0.03
	10	400	-0.02
	10	475	-0.03
CRM-16	10	250	0.21
	10	325	0.18
	10	400	0.41
	10	475	0.41
CRM-20	10	250	0.42
	10	325	0.49
	10	400	0.47
	10	475	0.49

3.2.2. Strain-controlled approach

Results of strain-controlled fatigue tests are reported in table 5. In addition to the number of cycles to failure evaluated with regard to both the considered failure criteria, the initial complex modulus and the initial dissipated energy are also reported.

Table 5. *Results of strain-controlled fatigue analysis (T = 15℃)*

Binder	Frequency [Hz]	Strain amplitude [%]	Initial complex modulus [MPa]	Initial dissipated energy [Pa]	N_f (50%G*) [-]	N_{p20} (DER) [-]
CRM-0	10	1.0	23.27	5688	194350	127152
	10	2.0	19.91	19980	28940	19756
	10	3.0	21.95	51028	9570	6980
	10	4.0	19.32	82579	5980	4194
CRM-16	10	1.0	17.73	3096	-	-
	10	2.0	17.01	12497	-	-
	10	3.0	13.02	23341	22760	39745
	10	4.0	15.46	50486	13300	8630
CRM-20	10	1.0	-	-	-	-
	10	2.0	12.13	8935	-	-
	10	3.0	10.45	18405	223300	558685
	10	4.0	10.93	35721	43820	53121

The first main observation is that crumb rubber modification strongly influences the fatigue life of the base bitumen. For the strain-controlled loading mode, CRM binders shown a great enhancement in fatigue resistance. The improvement of the fatigue life was recorded in the entire range of strain amplitude investigated but it was mainly evident at the lower strain amplitudes. In particular, when the tests were conducted at 1.0% and 2.0% strain amplitude, the CRM binders did not reach the reference conventional values (50%G* or 20% DER) of the failure criteria even after 1·106 cycles. In these cases we can hypothesize that the conditions are approaching a sort of endurance limit, as commonly defined in general fatigue analysis (Dowling, 1999).

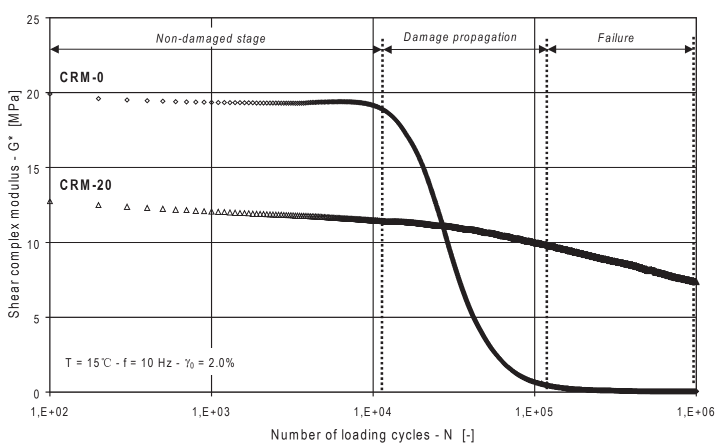

Figure 2. *Fatigue data from strain-controlled time sweep – G* analysis. Binders CRM-0, CRM-20 (γ0 = 2.0%).*

As figure 2 depicts, the G*(N) curve of binder CRM-0 is characterized by an initial horizontal plateau and by a subsequent increase in G* trend, identifying a non-damaged stage and a steric hardening respectively (Anderson et al., 2001). Differently, the absence of such initial characteristics in the trend of CRM-20 indicates that CRM binders have a specific fatigue behavior. As previously discussed for the stress-controlled analysis, in this case the region of damage progression begins soon at the start of the test and an effective non-damaged stage can not be well identified. However, as figure 3 depicts, the rate of the decrease in G*, extremely lower than those recorded for CRM-0, still leads to a very higher fatigue resistance.

The longer fatigue life measured for CRM-20 and CRM-16 can be explained as a consequence of the specific loading conditions. In a strain-controlled loading mode, the amount of stress and the dissipated energy per cycle, Wi decreases according to the increasing damage (increasing number of cycle, N). In presence of binders with low stiffness, i.e. CRM binders, this fact leads to reduce the values of Wi up to a threshold where damage can not further propagate up to the complete failure. This observation can be better outlined with regard to the DER analysis (figure 3).

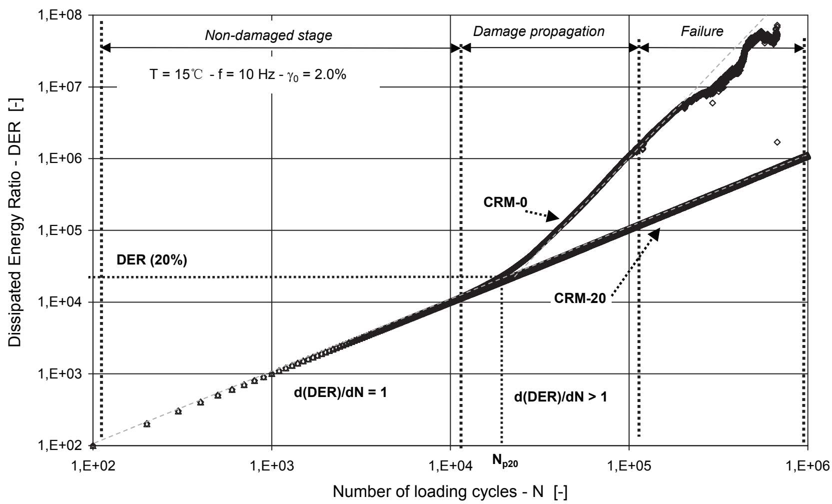

Figure 3. *Fatigue data from strain-controlled time sweep, DER failure criterion. Binders CRM-0 and CRM-20 (γ0 = 2.0%).*

3.2.3. Fatigue curves

Fatigue curves realized using the Np20 derived from the DER failure criteria are presented in figure 5 and 6. For the binders CRM-16 and CRM-20 very similar fatigue curves were found. As a consequence, only the data obtained for CRM-20 are reported. The fatigue curves are realized using the initial dissipated energy in order to plot both strain-controlled data (CR in the figures) and stress-controlled data (CS in the figures) in a same diagram. In figure 5, only two points compose the strain-controlled data series since for the lower strain amplitude no failure was recorded before 106 loading cycles (table 5).

In this representation, the previous observations on CRM binder fatigue behaviour are more evident and important differences arise between the modified binders and the unmodified one. In fact, a unique fatigue curve, that can be fitted with a unique power law in the W-N plot can be obtained for the unmodified binder only (figure 4). Contrary, for the binder CRM-20, a univocal fatigue curve can not be identified and data derived from strain-controlled analysis (white points in figure 5) are located on a different line. In this case the poor value of the coefficient R2, equal to 0.16, quantifies this observation.

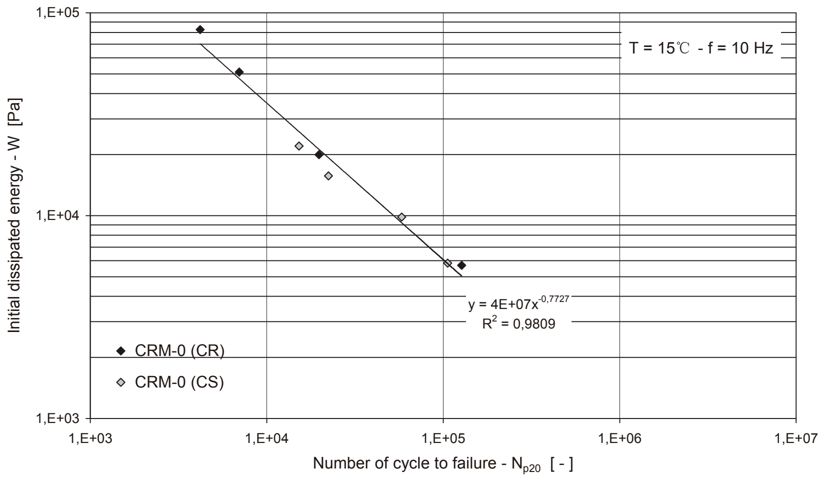

Figure 4. *Fatigue curve of CRM-0 from DSR time sweep, DER failure criterion (CR = strain-controlled data, CS = stress-controlled data).*

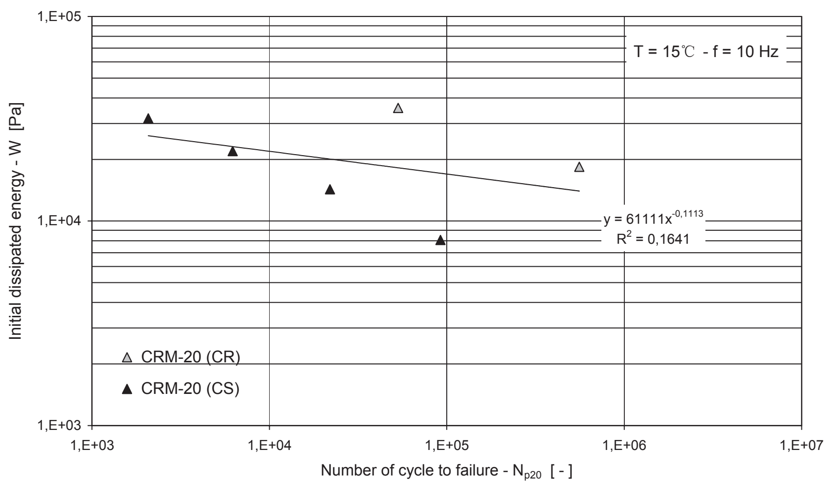

Figure 5. *Fatigue curve of CRM-20 from DSR time sweep, DER failure criterion (CR = strain-controlled data, CS = stress-controlled data).*

4. Conclusions

The fatigue behavior of CRM binders was studied using DSR time sweeps. The binders relative fatigue life was consequently evaluated according to different failure criteria. Based on the experimental results, it was shown as CRM asphalt binders have a specific fatigue behavior, strongly dependent on the loading mode, and characterized by its own fundamental characteristics.

By analyzing the DER trends obtained from stress-controlled time sweeps it was shown as in the case of CRM binders, the damage propagation starts as the test begins, but the presence of a more ‘ductile’ behavior leads to locate the complete failure threshold in correspondence of a very higher number of cycle. Differently, for the unmodified base bitumen, the region of the damage propagation cover a reduced number of cycle (less ductile behavior) and the failure occurs soon as the damage starts to propagate. However, in case of stress-controlled analysis, the differences in fatigue life due to the modification become less important when the stress amplitude is external to the linear viscoelastic region.

If the loading mode is strain-controlled, a very improved fatigue resistance was recorded for the CRM modified binders. In particular, when the tests were conducted for low strain amplitude, the recorded data did not meet the failure criteria even after 106 loading cycles. Finally, it was shown as, in presence of CRM, stress-controlled data and strain-controlled data must be described by different curves in the W-N plot.

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Influence of the Bitumen Properties on the Functional and Rheological Behaviour of Asphalt Rubber Binders

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ABSTRACT. It is estimated that about ten kilograms of tires are discarded per inhabitant annually. The negative impact of this residue can be reduced, since rubber can be reused as a constituent of asphalt rubber (AR) binder in road pavements. However, the materials which constitute the AR binders and their interaction are not sufficiently characterized. In this work several base bitumens interacted with crumb rubber, in order to produce AR binders, which were subsequently separated, by using a modified "Basket drainage method " to recover the residual bitumen and rubber. Additionally, a new "Sphere AR production simulator" method was developed to reproduce the bitumen aging without the contribution of the rubber. EN 12591 and EN 14023 standard tests and the Dynamic Shear Rheometer (DSR) were used to evaluate the changes in the properties of the binders during AR production. It was observed that the characteristics of the base bitumen significantly affect the AR binder properties (mainly for interactions with softer bitumens).

KEYWORDS: "Sphere AR production simulator", DSR, Rheology, Swelling, "Basket drainage method".

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1. Introduction

Approximately 9 to 10 kg of rubber tires per inhabitant and year are currently discarded in the industrialized societies. Although tires are not regarded as a dangerous residue, their hollow shape usually brings sanitary problems and difficulties in their final deposition. When ignited, the resulting fire is impossible to extinguish, releasing hazardous smoke. Finally, the vulcanized rubber of tires cannot be recycled or used for the same proposes as the unvulcanized polymer.

Furthermore, the society is wasting materials from the tire with a high intrinsic value, namely its main constituent: vulcanized natural and synthetic rubber. Thus, the introduction of crumb rubber in the production of Asphalt Rubber (AR) mixtures for road pavements should be considered as a sustainable technology which transforms an unwanted residue into a new mixture with a high resistance to fracture. However, the addition of rubber in a bituminous mixture increases its complexity, hence being essential to carry out a study to understand the interaction between its constituents in order to optimize the performance of AR mixtures.

The main objective of this work is to evaluate the influence of AR components, i.e. bitumen and rubber, on its final properties. Another aim of this research is to assess the functional and rheological changes that occur in the bitumen in the process of the AR binder production.

Different types of bitumen from the same source and obtained in the same distillation column were used in this study. Initially, the bitumen properties were evaluated using conventional EN standard tests and rheological tests carried out in the Dynamic Shear Rheometer (DSR). After that, the bitumen was left to interact with 17.5% of crumb rubber by mass of AR. Then, a modified “Basket Drainage Method” was used to separate again the constituents of the AR binder (residual bitumen and depolymerised rubber). The AR binder and the residual bitumen properties were then evaluated by replicating the same tests used for bitumen characterization, so as to assess the changes in the functional and rheological behaviour of the binder.

A new “sphere AR production simulator” test was developed during this work to simulate the changes that occur in the base bitumen during the production of AR that are independent from the interaction between bitumen and rubber (such as aging). This way, it would be possible to measure the changes in the bitumen caused exclusively by the interaction with crumb rubber.

The main results of this work are: (i) the establishment of a relationship between the functional and rheological properties of different bitumen and the resultant AR binders; (ii) the assessment of the rheological changes in the properties of the AR binder and of the base bitumen as a consequence of bitumen-rubber interaction.

2. Literature review on bitumen-rubber interaction

Presently, the pavement technology and the evaluation of bituminous mixtures are essentially based on empirical-mechanistic studies. Frequently, the materials that compose asphalt pavements are not sufficiently characterized, more specifically concerning their physicochemical constitution, their rheological properties and their mutual interaction at a microscopic scale.

Therefore, it is essential to develop knowledge about the interaction between the constituents of bituminous materials used in flexible pavements by carrying out their characterization at a microscopic level, so as to understand their macroscopic structural and functional behaviour. This study becomes more significant for modified or unconventional mixtures due to their complexity (e.g. Asphalt Rubber). Actually, beyond the traditional constituents of the bituminous layers of the pavements, the use of crumb rubber (recycled from used tires) should be studied as a form of environmental protection and as a pavement performance enhancer.

Absorption of bitumen components by the rubber inevitably depletes the bitumen of the absorbed components and, consequently, modifies its properties, by making it stiffer and brittler (Singleton et al., 2000; Artamendi et al., 2002; Airey et al., 2003). Furthermore, the rubber particles may also suffer some form of degradation (mainly devulcanization and depolymerisation) when they are mixed with bitumen at high temperatures for prolonged periods of time (Billiter *et al*., 1997; Zanzotto *et al*., 1996). The extent of swelling and degradation depends on the nature of the rubbers, the chemical composition of bitumen and the mixing conditions of time, temperature and degree of agitation. In addition, these processes will determine the mechanical properties of the crumb rubber modified binders (Abdelrahman *et al*., 1999). Asphalt rubber binders are also very dependent on asphalt characteristics: lower saturates and asphaltenes contents improve the asphalt capacity to dissolve rubber (Billiter *et al*., 1996).

Blending crumb rubber into asphalt is believed to improve its elastic and energy absorption properties, which are directly related to the resistance of the binder to cracking and rutting failures. The addition of crumb rubber also improves the resistance of asphalt binders to low temperature cracking (Gopal et al., 2002). Bahia *et al*. (1995) concluded that the impact of crumb rubber content on the reduction of low temperature cracking is a linear function of the rubber content and that the effect of the rubber is less significant for low viscosity asphalts.

The use of low penetration grade bitumens in asphalt-rubber mixtures reduces the rate and the amount of swelling of the crumb rubber particles. However, any changes in the rheological properties of the binder following rubber-bitumen interaction could result in the binder becoming embrittled (losing flexibility and capacity of resisting cracking and fretting). The use of high penetration grade bitumen will increase the rate and the amount of rubber swelling and therefore the shape and rigidity of the rubber, while the binder should still have sufficient flexibility, following the rubber-bitumen interaction, to resist cracking and fretting (Airey *et al*., 2003).

The basket drainage bitumen absorption method (Rahman, 2004) was found to be a simple and effective way to monitor rubber-bitumen interaction, when the crumb rubber particles absorb bitumen and swell when added together at mixing temperatures. The rate and amount of absorption is mainly dependent on the mixing temperature and on the complex chemical nature of bitumen, but only marginally dependent on the bitumen type and grade. In addition to normal oxidation, the residual bitumen experienced further changes in mechanical and rheological properties in terms of increase in stiffness, elasticity, viscosity and reduction in penetration.

Rheology is the science concerned with the aspects of deformation of real bodies under the influence of external stresses (Ferguson *et al.*, 1991). The concepts of rheology should be used to define the characteristics of bituminous binders, due to the importance of the time-dependent deformation or creep response that occurs when binders are subjected to loads (Tia *et al.*, 1987).

Asphalt rubber (produced by the wet method) is a very special binder if compared to others. Extremely low phase angles at high temperatures/low frequencies and relatively high phase angles and low stiffness at very low temperatures/high frequencies make it a very interesting binder (van de Ven *et al.*, 2003). The addition of crumb rubber to bitumen decreases the elastic and viscous moduli at low temperatures and, therefore, it causes an increase in binder flexibility. On the contrary, at high temperatures a significant increase in both moduli and a notable drop in the loss tangent values result in a more elastic binder. Furthermore, it can be deduced that the thermal susceptibility of the binder is clearly reduced as a consequence of rubber addition. Consequently, enhanced resistance to permanent deformation, low-temperature and fatigue cracking should be expected in the resulting asphalt rubber mixtures (Navarro *et al.*, 2005).

3. Asphalt rubber production, separation and characterisation

3.1. Bitumen selection, sample preparation and crumb rubber selection

3.1.1. Bitumen samples

In order to control the variables to be studied, all the bitumen samples were obtained from the same batch of bitumen production on the refinery (the same distillation column), since bitumen characteristics are different depending on the petroleum sources and their processing.

Asphalt rubber properties are very dependent on the used bitumen. Thus, this work was begun by choosing several grade bitumens in order to evaluate their influence on the AR characteristics. Due to the difficulty of the refinery to supply other bitumens than the commercial line ones (10/20, 40/50, 60/70 and 150/200), these commercial bitumens were combined in different proportions in order to obtain sixteen base bitumen samples (Table 1).

Table 1. *Proportions of commercial bitumen used to obtain the sixteen base bitumen samples*

Sample labelling		A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
Commercial bitumen	10/20	1.00	0.75	0.50	0.25						0.25	0.50	0.75				
	40/50		0.25	0.50	0.75	1.00	0.75	0.50	0.25								
	60/70									1.00	0.75	0.50	0.25		0.25	0.50	0.75
	150/200						0.25	0.50	0.75					1.00	0.75	0.50	0.25

The laboratorial characterisation of these sixteen base bitumen samples was carried out in order to evaluate the following properties:

- Functional properties, namely the penetration at 25 ºC (EN 1426), ring and ball softening point (EN 1427), the dynamic viscosity (EN 13302) and the elastic recovery (ASTM 5329);

- Rheological properties, using the Dynamic Shear Rheometer (only for eight of the studied bitumens) to determine the viscosity, the modulus and the phase angle.

In addition, the four commercial bitumens were also subjected to a “sphere AR production simulation” test developed in this work to simulate the changes that occur in the base bitumen during the production of AR that are independent from the interaction between bitumen and rubber. After these procedures, the changes in the functional and rheological properties of the residual bitumens were assessed, in comparison with the AR binder residual bitumens. This way, it was possible to measure the changes in the bitumen caused exclusively by the rubber.

3.1.2. Crumb rubber sample

The crumb rubber used in this work was produced by the cryogenic process and, according to the supplier, was obtained by the cryogenic grinding of 30% of truck tires and 70% of car tires. The supplied crumb rubber was sieved in order to select and use only the fraction passed in sieve ASTM #20 (0.850 mm) and retained in sieve ASTM #40 (0.425 mm).

3.2. Production and separation of the asphalt-rubber binder

The method used to produce AR and collect the desired samples of AR, of recovered swelled rubber and of residual bitumen was the “basket drainage method” (Rahman, 2004). The asphalt rubber production facility is constituted by several equipments, presented in Figure 1, assembled in a laboratory ventilation chamber.

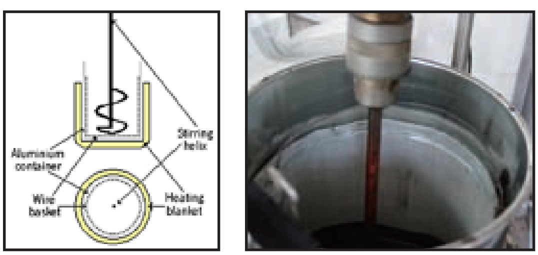

Figure 1. *Scheme and photo of the asphalt rubber production facility*

The wire basket used in the AR production facility was manufactured with a wire mesh, which was previously observed in an optical microscope (Figure 2) to measure its average opening dimension (# 0.470 mm). The microscope was able to draw an idealized picture of the mesh in order to assess, automatically, the referred opening dimension.

The process used to produce and collect the several samples of AR, of recovered swelled rubber and of residual bitumen was the following one:

- Heating of 1 kg of each base bitumen (Bb) at 180 ºC and collecting of a sample;

- Introduction of 17.5% of crumb rubber by mass of asphalt rubber;

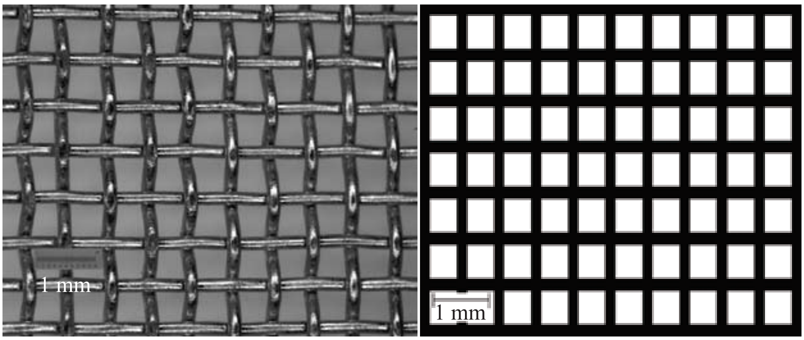

Figure 2. *Microscopic photo and drawing of the wire mesh used to manufacture the basket*

- Continuous heating of the asphalt rubber binder at 180 ºC for 60 minutes (digestion time) while stirring the binder at a velocity of 230 rot/min (Figure 1);

- Collection of a sample of the produced asphalt rubber (AR);

- Suspension of the wire basket with the AR binder in an oven at 180 ºC for 15 min, so as to separate its constituents (Figure 3) and collect a sample of the residual bitumen (Res) (Figure 4).

Figure 3. Separation between the crumb rubber and the residual bitumen of the AR

Figure 4. *Appearance of the residual bitumen and of the recovered rubber after separation*

3.3. "Sphere AR production simulator" method

One of the objectives of this research is to determine the changes in the bitumen characteristics caused by the interaction with crumb rubber.

When the bitumen is heated, smaller molecules are volatilized (aromatics), while other molecules can associate themselves with bigger molecules (asphaltenes). Furthermore, when bitumen interacts with rubber, the rubber particles absorb the light parts of bitumen and swell. Since those phenomena simultaneously occur during the AR production, it seems unfeasible to evaluate the changes in the bitumen structure caused by every previous phenomenon. Hence, a new method was developed during this research in order to determine the ratio of the changes exclusively induced by heating and stirring the bitumen during the AR production, entitled "sphere AR production simulator". The aim of this method is to simulate the AR production, by replacing the rubber particles with metallic spheres (roughly with the same dimensions of the rubber particles) in order to reproduce the evaporation and association processes occurring in the bitumen during the heating period for AR production, independently from the interaction with rubber. Before the experimental procedure, the metallic spheres were measured ($ø_{eq} \approx 0.825$ mm) using an optical microscope and their volumetric weight was evaluated (7.75 kg/dm^3).

The "Sphere AR production simulator" was carried out in a reduced scale AR production facility (similar to the one used for AR production), and consisted in the following procedures:

- Heating of approximately 100 g of each commercial bitumen at 180 ºC;

- Introduction of approximately 240 g of metallic spheres (these spheres have a volume equivalent to 17.5% of crumb rubber by mass of asphalt rubber);

- Continuous heating of the binder (bitumen and metallic spheres) at 180 ºC for 60 minutes (digestion time) while stirring the binder at a velocity of 230 rot/min;

- Suspension of the wire basket with the binder (bitumen and metallic spheres) in an oven at 180 ºC for 15 min, so as to separate its constituents;

- Collection of a sample of the resulting bitumen (Sb) for evaluation of its functional properties and future comparison with the base (Bb) and residual (Res) bitumen properties.

3.4. Analytical determinations

During this work a series of analytical determinations were carried out in several samples of binders obtained during the AR production, namely by evaluating the penetration at 25 ºC (Pen), the ring and ball softening point (R&B), the dynamic viscosity (DV), the elastic recovery (ER) and the rheological properties using the Dynamic Shear Rheometer (DSR). Table 2 systematizes the laboratorial research analysis performed in this work with the base bitumens (Bb), asphalt rubbers (AR), residual bitumens (Res) and "Sphere simulator" recovered bitumen (Sb).

3.5. Determination of the linear viscoelasticity range of stresses to be used in the rheological characterization of the binders

From the sixteen initial samples of base bitumen, only eight were selected (based on their functional properties) to evaluate their rheology. A DSR was used to assess the rheological properties of the base bitumen, AR and residual bitumen of those eight bitumens.

Table 2. *Analytical determinations carried out in this work*

<table>
<tr><th colspan="2">Sample labelling</th><th>A</th><th>B</th><th>C</th><th>D</th><th>E</th><th>F</th><th>G</th><th>H</th><th>I</th><th>J</th><th>K</th><th>L</th><th>M</th><th>N</th><th>O</th><th>P</th></tr>
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The rheological tests were carried out at different temperatures (25, 35, 50, 80, 110, 140, 160 and 180 ºC) in order to determine the rheological variation of the binder properties at all in service, laying down and mixing stages. For each temperature, frequency sweep tests between 0.1 and 10 Hz were performed in the linear viscoelasticity range, using parallel plate geometry with 40 mm of diameter and 1.0 mm of gap between plates.

Initially, it was necessary to establish the input stress at each DSR test temperature. Therefore, previous stress sweep tests were performed at 1.0 Hz to determine the linear viscoelasticity region (at each test temperature). The linear viscoelasticity range can be identified in Figure 5 as being the range of stresses where no change is observed in the values of the rheological characteristics measured at each test temperature.

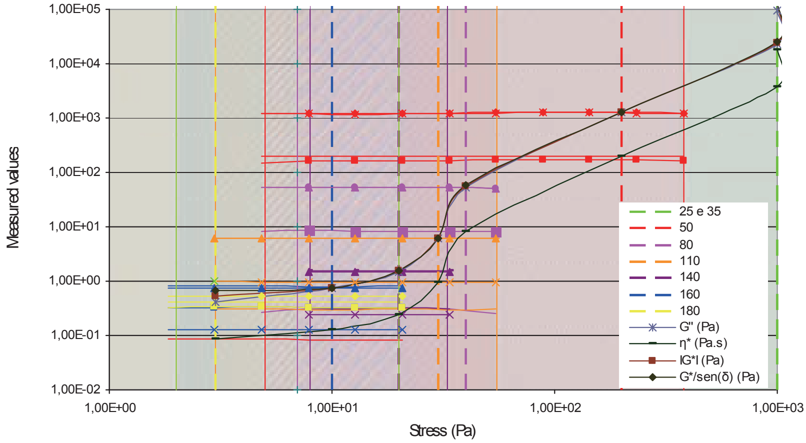


Figure 5. *Summary of the results of the rheological stress sweep tests at 1 Hz*

The selection of the stress values to apply on the rheological tests at each temperature was made to guarantee that the frequency sweep tests will be carried out in the linear viscoelasticity region for all tested binders, thus being possible to compare their rheological behaviour.

Rheological time sweep tests were also performed, at a constant temperature, stress and frequency, to confirm that no structural modifications occurred in the binders throughout the tests (60 minutes were used to evaluate the changes during AR production). The rheological characteristics assessed in the DSR were the complex modulus (G*), storage or elastic modulus (G'), loss or viscous modulus (G''), complex viscosity (η*) and phase angle (δ).

4. Results and discussion

4.1. Functional characterization of the studied binders

The functional characteristics of different samples of binder collected throughout the AR production, namely the base bitumen (base) and the asphalt rubber binder (AR), were assessed in this part of the work in order to evaluate the changes in the binder properties caused by the addition of rubber to the bitumen. Thus, Figure 6 presents the functional characteristics (penetration, R&B and dynamic viscosity) of the sixteen different base bitumens (A to P) and of the resultant AR binders produced in this study, as well as the differentials (Δ) between the functional properties of each matching pair of base bitumen and AR binder.

Based on the values of the functional properties, it was possible to classify and sort out the sixteen base bitumens. The results of the different tests were very consistent with the proportions of the used commercial bitumens. Actually, the changes in the constitution of the samples (from harder to softer bitumens) originated an ordered sequence for the evaluated functional properties, represented by the potential trending lines presented on the left side of Figure 6.

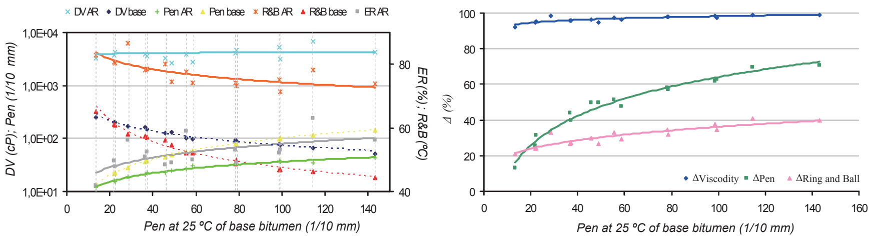


Figure 6. *Functional properties of the ARs compared to the ones of the base bitumens*

For every functional property evaluated, the AR binder results presented greater dispersions, mainly because this material is more heterogeneous than the bitumen. As it was expected, the AR binders presented much lower values of penetration and higher values of ring and ball temperatures and dynamic viscosity than the corresponding base bitumens. The dynamic viscosity test is the most effective to find variations in the behaviour of the different materials, since it distinguishes the several base bitumens (A to P) and the corresponding AR binders very well.

The influence of the type of base bitumen in the functional characteristics is clearer in the evaluation of the base bitumens. In opposition, the functional characteristics of the AR binders produced with the different bitumens are very similar, mainly the dynamic viscosity and the elastic recovery which are almost constant. The comparison between the several types of bitumen used to produce AR also showed that the highest variations in the functional characteristics of the binder (AR comparison with base bitumen) are obtained with the softer bitumens. Thus, even the 150/200 bitumen (M), which is not generally used to produce asphalt rubber and hot mix asphalts, revealed to be a good alternative to produce AR binders with a final penetration similar to the one of the 60/70 base bitumen (I).

One of the aims of this research is to determine the changes in the bitumen characteristics caused exclusively by the interaction with crumb rubber. The "sphere AR production simulator" method was applied to obtain recovered bitumens (Sb) that were influenced by the processes occurring in the bitumen during the AR production, independently of the interaction with rubber.

The AR is a heterogeneous material, which can be separated in two different phases, the swelled rubber and the residual bitumen. The swelled rubber, by itself, does not promote the adhesion between the aggregates in the production of the AR mixtures. Thus, it is reasonable to consider the swelled rubber as a special category of aggregates, being the residual bitumen the final binder of the AR. Therefore, it is essential to evaluate the behaviour of this residual bitumen of the AR binder in order to optimize the performance of the AR mixtures.

The measured values of the functional properties of the "sphere" and residual bitumens are presented in Figure 7, in comparison with the corresponding base bitumens. The graph on the left side of Figure 7 represents the evolution of the functional properties of the residual, "sphere" and base bitumen. The right side of Figure 7 shows the proportional changes in the functional properties of the base bitumen (differential percentage in comparison with the base bitumen) due to the "sphere AR production simulator" method and to the interaction with rubber.

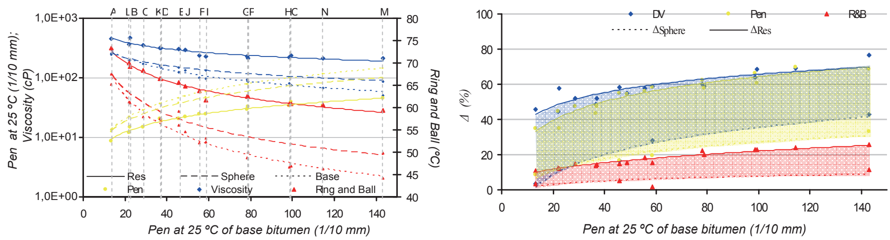


Figure 7. *Functional properties of the "sphere" and residual bitumens in comparison with the corresponding base bitumens*

The changes on the bitumen constitution, after the "sphere" method and the interaction with rubber, can be evaluated by measuring the variation in the functional properties of binders. These changes are more perceptible in the DV test, while the R&B test is the less sensitive one.

The results obtained for the "sphere" recovered bitumen are caused by the changes occurred during the production of the AR that are independent from the bitumen-rubber interaction, being mainly caused by the volatilization of light parts of the bitumen. The variation on the functional properties of the recovered bitumen after the "spheres" method is superior for softer bitumens and has an average value of 10% in the R&B test, 20% in the pen test and 35% in DV test. The results obtained for the residual bitumen show that the changes in the bitumen caused by rubber are superior to the ones observed for the "sphere" bitumen, which indicates a significant absorption of light bitumen parts by the rubber particles.

For the conditions adopted in this work, it was noticed that the variation in the functional properties caused by the bitumen-rubber interaction remains constant for binders obtained from base bitumens with penetration values superior to 30 dmm (as presented in the shaded band for each property on the right side of Figure 7). For these binders produced with base bitumen with pen values superior to 30 dmm, the changes in the functional properties for the AR residual bitumen doubles or triples the variation obtained for the "sphere" bitumen. Thus, when the bitumen has some concentration of light molecules able to be absorbed by the rubber and/or volatilized due to the temperature and agitation, these two phenomena coexist almost equally.

However, the use of harder bitumens (pen < 30 dmm) in the AR originates binders with ratios absorption/volatilization greatly superior to 1. For these binders the changes caused by the evaporation of the volatile molecules of bitumen are considerably inferior to the ones caused by the rubber-bitumen interaction. Thus, the process of diffusion of the smaller molecules of the bitumen into the rubber is more efficient than the bitumen volatilization, occurring initially during the AR production. The quantification of the bitumen changes during the production of AR is difficult to evaluate, since several inter-dependent changes simultaneously occur in the rubber and in the bitumen, such as diffusion and volatilization.

However, the presented results allowed a better understanding of the extension of these two phenomena.

4.2. Rheological characterization of the studied binders

The viscoelasticity of bitumen can be described by plotting the shear stress – τ (Pa) vs. shear rate – γ (s^{-1}), as presented in Figure 8 for the base bitumen 150/200 at different temperatures.

The bitumen M (150/200) behaves as a non-Newtonian linear viscoelastic material, or a Bingham solid, for temperatures below 150 ºC. Above the referred temperature it behaves as a Newtonian fluid. The slope of the several lines presented in Figure 8 increases as temperature reduces, as a consequence of the rise in the stiffness and viscosity of the bitumen.

Only eight bitumens were selected to carry out rheological tests. In order to exemplify the typical behaviour of the bitumen in the DSR tests, some results are only presented for the base bitumen I (60/70) since it is the commercial bitumen typically used to produce AR binders.

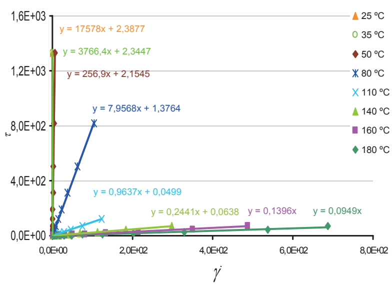


Figure 8. *Shear stress vs. shear rate curves for bitumen 150/200, at different temperatures*

One of the most effective ways of representing the data obtained in the DSR tests is by drawing master curves. These curves can be plotted because it is generally accepted that the bitumen has a simple rheological behaviour, being applicable the time-temperature superposition principle (TTS), by using the Arrhenius equation [1] or the Williams-Landel-Ferry (WLF) equation [2]. The result of the superposition of curves at different times (frequencies) and temperatures are the master curves, that cover a large range of time or frequency domains.

$$\eta^* = k.\exp[E_a/(R.T)] \qquad \text{for } T < T_g \qquad [1]$$

$$\text{Log } a_T = -C_1.(T\text{-}Tg)/(C_2+T\text{-}T_g) \qquad \text{for } T > T_g \qquad [2]$$

With the initial data it is not possible to determine the glass transition temperature (Tg) value, and consequently it is not possible to identify the equation that better approaches the data used to draw the master curves. Thus, the easiest way to choose the best equation to fit the data is to draw a graphic of the logarithm of the stiffness parameters versus the inverted temperature. The Arrhenius equation should be used for a linear relation between the plotted data, or else the best approach is obtained with the WLF equation. The first graph of Figure 9 (plot of η* vs. 1/T) shows a linear relation between the plotted parameters only for temperatures above 100 ºC (equation [1] must be used to draw the master curve). Below 100 ºC the master curve must be drawn with equation [2]. The application of equation [2] has three output values: C_1, C_2 and T_g. This last parameter is a characteristic value of each material. Its determination depends on the geometry used to assess the rheological parameters and on the material behaviour. In this case the Tg parameter could only be achieved by using a bending beam geometry. However, the master curve shows a point of interception (nearly at 30 ºC) of G' and G'' lines (tg δ = 1) for temperatures between 25 ºC and 100 ºC, indicating some change in the rheology of the bitumen.

Above 100 ºC the graphic of η* vs 1/T can be approached to a straight line, and thus the Arrhenius equation was used to plot the master curve. The Arrhenius equation also provides a characteristic value of each material, the activation energy (E_a). In this case, the E_a value should not be considered, since it would be necessary to use other geometry (rotating spindle) to assess more significant values. However, this master curve also shows a point of interception of G' and G'' lines (nearly at 170 ºC) that corresponds to a new alteration in the rheology of the bitumen.

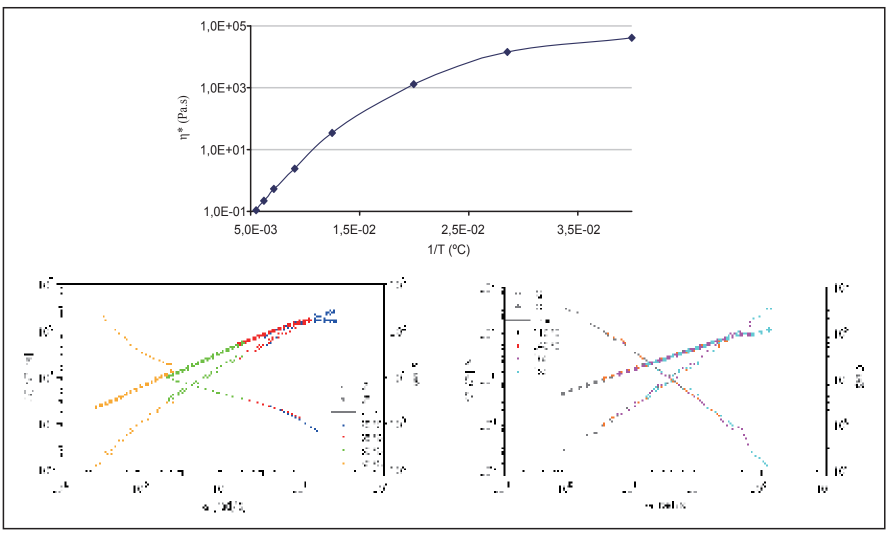


Figure 9. *Master curves of the base bitumen I (60/70) at two different ranges of temperatures*

With the rheological results obtained in this work it is not possible to draw a unique master curve (Figure 9) covering the entire range of tested temperatures. It will be necessary to use a variety of diverse geometries in different rheometers (for each range of temperatures), and more temperatures should be tested to guarantee a good superposition of the obtained data.

In order to evaluate the differences in the rheological behaviour of the base bitumen, AR binder and residual bitumen at different temperatures, the Figure 10 summarizes the main rheological results obtained at 1.0 Hz for bitumen I (60/70). For example, the separation of the G'' and G*/sen(δ) lines for temperatures below 80 ºC only occurs in the AR binder, because of the increase in the values of G' (the rubber particles are essentially elastic) in relation to G'', thus reducing the values of the phase angle (δ) for similar values of G*.

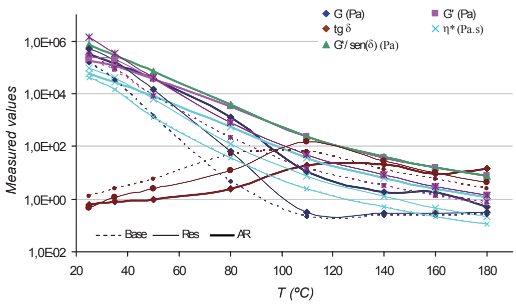


Figure 10. *Rheological characteristics of the base bitumen 60/70 and the corresponding AR binder and residual bitumen*

The inclusion of the crumb rubber in the bitumen not only rises the stiffness of the new binder (AR), but also changes the development of the storage (G') and loss (G'') modulus with the temperature. The G' value greatly rises due to the presence of an elastic solid, the rubber, that increases the elasticity of the AR. Actually, for temperatures below 50 ºC, the AR presents G' values superior to those of G'', which is consistent with the better related resistance of the AR to rutting, fatigue and reflective cracking distresses of the pavement at in service temperatures.

Between 80 and 160 ºC, the rheological behaviour of the AR binder, at the chosen frequency (1 Hz), is similar to the one of the base bitumen, even if some differences are evident in the values of G', G'' and G*/sen(δ) that are higher but proportional to the ones of the base bitumen. Thus, the AR binder is stiffer than the base bitumen, but its rheological behaviour at production and applying temperatures is somewhat similar.

On one hand, for temperatures inferior to 100 ºC, the differences in the rheological properties between the residual and the base bitumen are evident, presenting the maximum discrepancy between 50 and 80 ºC. In this range of temperatures, the residual bitumen is clearly stiffer than the base bitumen, being even stiffer than the AR binder for temperatures inferior to 35 ºC. On the other hand, for temperatures superior to 110 ºC, the differences between the rheological properties of the residual and the base bitumen are minor, reducing as the temperature increases and trending to the same value at 180 ºC. However, at these higher temperatures the AR binder still presents rheological properties clearly superior to the base and residual bitumens.

Figure 11 shows the isochronal plots of the rheological parameters of the base bitumen and of the resulting AR binder at a frequency of 1.0 Hz and temperatures between 25 and 180 ºC for eight bitumens selected to carry out the DSR tests (A, B, E, I, J, M, O and P).

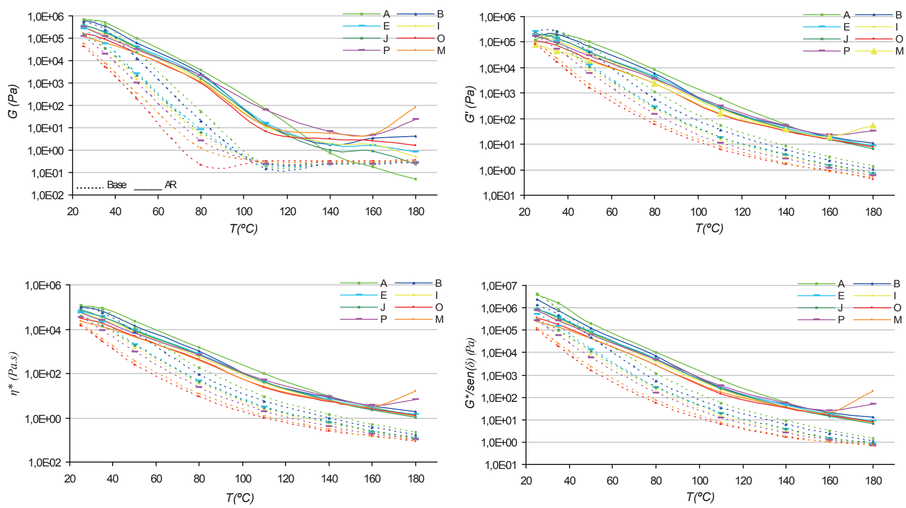


Figure 11. *Isochronal plots of the rheological properties of the base bitumens and ARs at 1.0 Hz*

The AR binder is the stiffest material, being the less susceptible to the change of the type of base bitumen (plots are very similar) and to the variation of temperature. In fact, the high increase in the elasticity of the AR binder for all studied temperatures, caused by the presence of rubber particles, totally changes the rheology of the binder, by reducing the influence of the temperature in the behaviour of the binder (AR does not become a viscous liquid above 100 ºC).

Asphalt binders must have a set of characteristics in order to allow their adequate use in road pavement construction at different temperatures. Binders should not be very rigid at low and medium operating temperatures (to avoid cracking distresses), but they should be sufficiently stiff at high operating temperatures to improve their resistance to permanent deformation. The AR binders fulfil these demands for every used base bitumen, even for the softer bitumen 150/200 (M), since the values of the rheological parameters at 25-35 ºC are similar to the base bitumen, being much higher at 60-80 ºC. However, the several AR binders present elevated viscosities at high temperatures, making the mixing and laying down of AR mixes more difficult.

The changes on the viscoelastic nature of the bitumen after the interaction with crumb rubber are also presented in Figure 12, which shows the evident differences between the tgδ plots of the base bitumen and the AR binder for the tested range of temperatures at a frequency of 1.0 Hz.

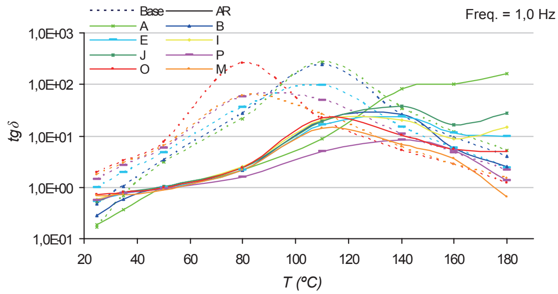


Figure 12. *Isochronal plots of tg δ for the studied binders (base bitumens and ARs) at 1.0 Hz*

The maximum value of tgδ observed in Figure 12 for the several base bitumens occurred for temperatures within 80 and 110 ºC. This means that a reversible phenomenon of relaxation takes place in this range of temperatures for all the bitumens, because the same results were obtained after repeating the DSR test in the same samples of bitumen. In comparison with the base bitumen, the maximum value of tgδ and the relaxation phenomenon observed in Figure 12 for all AR binders occurred for higher temperatures within 120 and 150 ºC. Moreover, at temperatures below 110 ºC the values of tgδ of the AR binders are inferior to the ones of the base bitumen (due to the increased elasticity of AR), but above 110 ºC the AR and the base bitumen values of tgδ are similar. Finally, it was observed that the AR values of tgδ are almost independent from the type of bitumen used in the AR for temperatures between 35 and 80 ºC.

The viscoelastic rheology of the several AR binders, base and residual bitumens, can be easily observed for temperatures between 25-50 ºC in the Cole-Cole chart shown in Figure 13.

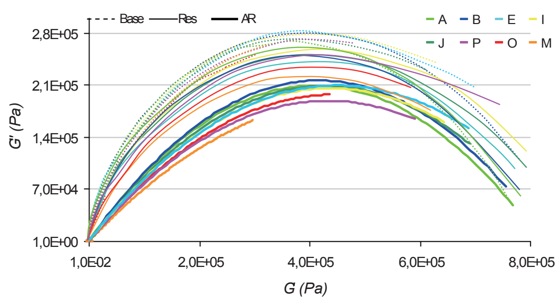


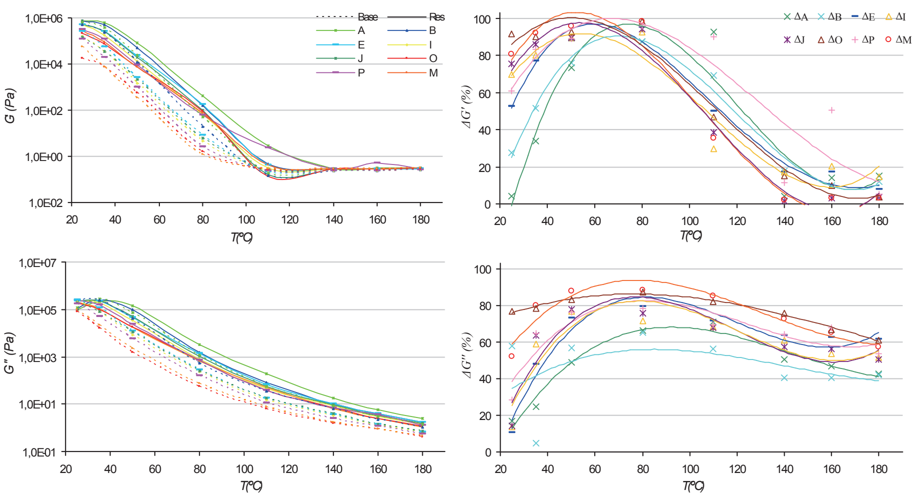
Figure 13. *Comparative Cole-Cole chart for the AR binder, base and residual bitumen*

After analyzing the three sets of curves (AR binder, residual and base bitumen), a generalized hardening was observed for the residual bitumen, that is more obvious for the AR binder, due to the increase of elasticity (G') and decrease of viscosity (G''). The Cole-Cole curves of the several AR binders and residual bitumens have slightly the same form of the corresponding base bitumens and are organized by the same order. Thus, the softer bitumen (M) is always the lower and shorter curve and the harder bitumen (A) is the upper and longer curve of each set, being the other six bitumens organized by the same logic between the previous limits. Besides the stiffness increase, there is no sign of other specific changes in the residual bitumen caused by the interaction with the rubber for temperatures between 25 and 50 ºC.

The set of Cole-Cole curves of the AR binders are wider than the ones of the base and residual bitumens, denoting a major influence of the crumb rubber in the modification of the softer bitumens. In fact, during the production of the AR binder, the softer base bitumen presented the best interaction with the crumb rubber. In summary, the AR binders produced with softer bitumens presented good rheological characteristics at in service temperatures, thus showing the benefits of the strong interaction between that bitumen and the crumb rubber.

The changes induced in the rheology of the base bitumen during the AR production can be evaluated by comparison with the rheological properties of the residual bitumen, as presented in Figure 14 for all studied bitumens at the frequency of 1.0 Hz.

The analysis of the charts on the left side of Figure 14 showed that the isochronal curves of the residual bitumen are less scattered than those of the base bitumen, particularly at higher test temperatures, probably due to the several processes occurred during the AR production. The rheological property that better reflects the described behaviour is the storage modulus. The G' isochrones defining the evolution of the elastic character of the different bitumens became completely independent from the constitution of the different residual and base bitumen and also from temperature, since those isochrones nearly overlap defining a single line parallel to the horizontal axis above 140 ºC. The weak interactions that maintained the "networks" of asphaltenes/resins no longer exist at 140 ºC and the bitumen behaves as a Newtonian fluid, where the asphaltenes are totally peptized by the resins and dispersed in the maltenes.



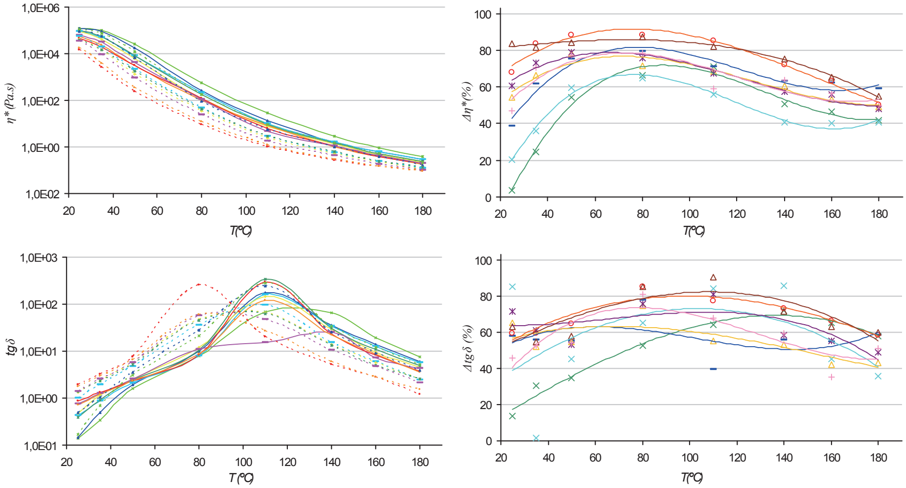


Figure 14. *Isochronal plots of the rheological characteristics of base and residual bitumens and the corresponding differentials at 1.0 Hz*

As it was expected, the graphics on the right column of Figure 14 confirmed the results previously obtained with the comparative analysis of the functional properties of the base and residual bitumens. Therefore, it was observed that the constitution of the base bitumen has a great influence in the rheological and functional properties of the residual bitumen. The softer bitumen presented the greatest changes in its rheological properties after the AR production, mainly for temperatures between 25-35 ºC. However, the maximum variations in the rheological properties of the base and residual bitumen were usually observed between 35 and 110 ºC.

In conclusion, the softer a base bitumen is, the more affected it will be during the AR production process, altering more drastically its initial characteristics. Thus, the soft base bitumens used in AR binders become significantly stiffer and eventually more adequate to be applied in pavements conferring an increased resistance to weather and traffic actions.

4.3. Comparison between the rheological and functional properties of binders

During the pavement life cycle, the asphalt binders used in bituminous materials and pavements should always present an adequate behaviour, initially during the mixing and lay down phases at elevated temperatures, then during the operating phase, in order to resist to the weather and traffic actions at low and medium temperatures and, finally, in the removal and recycling of the distressed mixtures (this last phase was not studied in this work). The results of the two different approaches (functional and rheological, for a reference frequency of 1 Hz) used to assess the behaviour of the different studied binders (A, B, E, I, J, M, O and P) will be analysed and compared in this part of the work.

At medium operating temperatures causing fatigue cracking distresses, the functional parameter measured in this work was the penetration at 25 ºC. Thus, Figure 15 relates the penetration value of the several binders to the rheological properties measured at 25 ºC. The range of values of the penetration results is visibly inferior to the rheological classification for the studied binders, and thus the vertical axis (rheology) is presented in a logarithmic scale. Actually, the penetration values of AR binders and residual bitumens only show little variations, being always inferior to 50 dmm.

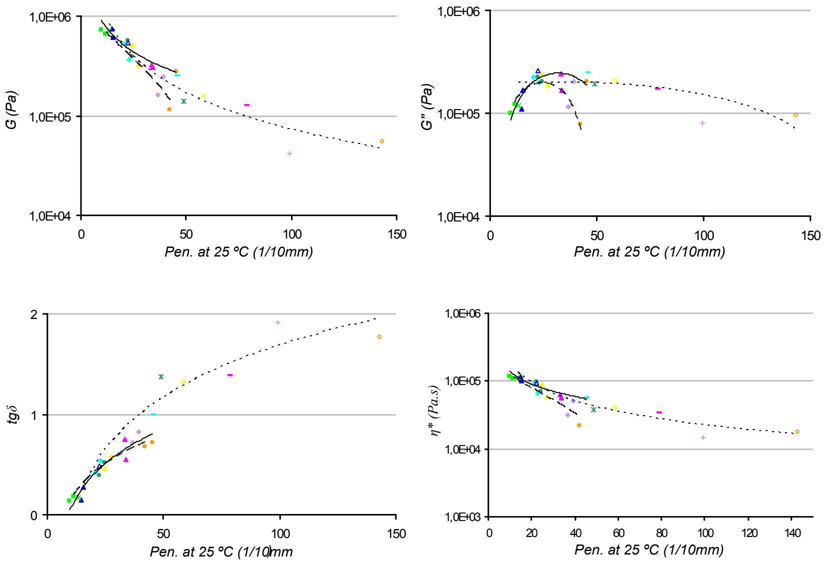


Figure 15. *Rheological vs. functional properties of the studied binders at operating temperatures causing fatigue cracking*

The best fit to the results that relate the functional and rheological properties of the binders at 25 ºC was obtained with potential trending lines. The exceptions were tgδ and G'', which presented polynomial best fit lines with significant dispersion (probably the relation between these properties and the penetration value is not very good at 25 ºC). Comparing the rheology of the different binders (base and residual bitumen and AR) for the same penetration value, the AR binder is the softest material. This result is consistent with the best fatigue cracking resistance of this material. The most considerable changes in the characteristics of the binder, before and after the interaction with crumb rubber, were observed for bitumen 150/200 (M). The penetration of the base bitumen M (145.3 dmm) decreased considerably for the resulting AR binder (42.1 dmm) and residual bitumen (45.4 dmm), and the penetration becomes similar to the one of the base bitumen 40/50 (E). However, the changes in the rheology of bitumen M were not so severe. By opposition, the functional and rheological characteristics of base bitumen A are nearly the same of the corresponding AR binder and residual bitumen. Thus, the presence of rubber in the binder A does not affect the AR binder behaviour at this temperature (25 ºC).

After that, the evolution of the rheological properties of the different AR binders was compared with their elastic recovery in Figure 16. These results are important since the changes occurring in harder binders (bitumen A) after the interaction with crumb rubber could be only evaluated through the analysis of the elastic recovery tests of AR binders.

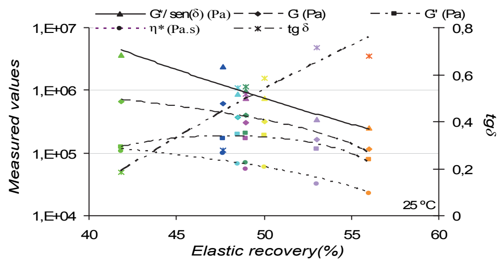


Figure 16. *Relation between the elastic recovery at 25 ºC and the rheological parameters of the different AR binders*

The elastic recovery of AR binders increases as the base bitumen used for their production is softer. The reduction of the stiffness of the residual bitumen surrounding the rubber particles in the AR allows an easier and faster recovery of their form after deformation, while the increase of the swelling of the rubber particles also confer them an improved volume and elasticity. The trending lines shown in Figure 16 (except for tg δ) show a reduction in the rheological properties of the AR with the increase in the elastic recovery. Thus, the elastic recovery of the AR binders could be eventually increased by reducing the stiffness of the AR binders.

At high operating temperatures causing rutting distresses, the functional parameter measured in this work was the ring and ball softening temperature. Thus, Figure 17 relates the ring and ball softening temperature of the several binders to the rheological properties measured at 50 ºC.

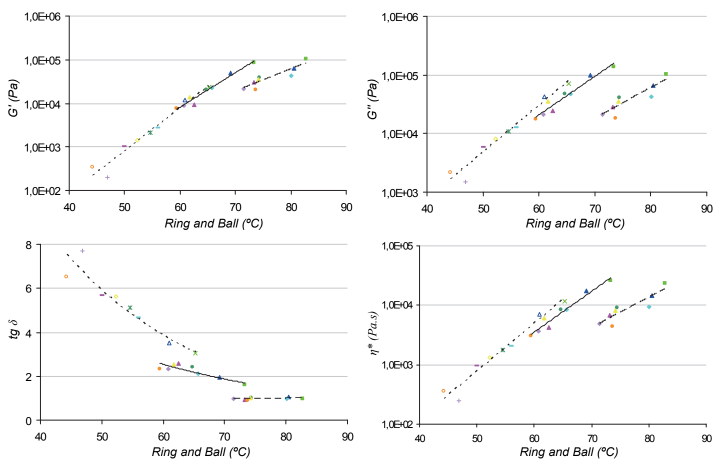


Figure 17. *Rheological vs. functional properties of the studied binders at high operating temperatures causing rutting*

The range of values of the ring and ball softening temperature is visibly inferior to the rheological classification for the studied binders, and thus the vertical axis (rheology) is presented in a logarithmic scale. The best fit to the results that relate the functional and rheological properties of the binders at 50 ºC was obtained with exponential trending lines. The ring and ball temperature presented a very good relation with the rheological properties. The relations between the functional and rheological properties for the several types of binders (base, AR and residual) are clearly different, since the trending lines obtained for every binder are visibly detached. This occurs because the softening temperatures of the several types of binders are dissimilar, especially the AR binder that melts above 70 ºC. In fact, the softer AR binder is stiffer than all the base bitumens at these higher temperatures. The ring and ball values and the rheological properties of the residual bitumen and AR binder show a significant stiffening of the base bitumen at high operating temperatures after AR production. Unlike the penetration results, the AR binders and residual bitumens presented a considerable variation of their ring and ball and rheological properties in relation to the base bitumen, and the materials produced with softer bitumens were more affected.

At mixing and laying down temperatures, the functional parameter measured in this work was the dynamic viscosity at 180 ºC. Thus, Figure 18 relates the dynamic viscosity at 180 ºC to the rheological properties also measured at 180 ºC.

The dynamic viscosity at 180 ºC appears to be more accurate to describe the bitumen behaviour at this temperature than the rheological parameters, since the range of the DV results is visibly superior to the rheological classification. Thus, both horizontal (functional) and vertical (rheology) axes are presented in a logarithmic scale.

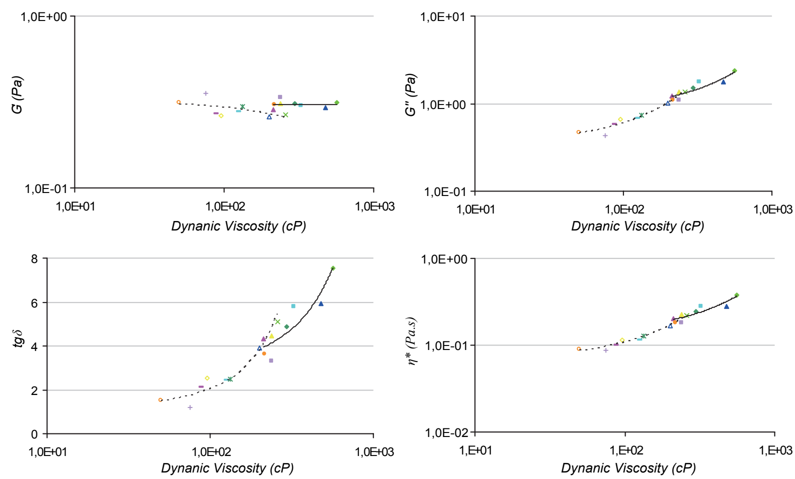


Figure 18. *Rheological vs. functional properties of the base bitumen at production and laying down temperatures*

The residual bitumens are clearly stiffer than the base bitumens at 180 ºC (with a great increase in the dynamic viscosity and rheological properties). Finally, it was observed that the relation between the dynamic viscosity (functional test) and the complex viscosity (DSR test) is very good, nearly with the same results obtained in both test methods (since 1 Pa.s = 1000 cP).

5. Conclusions

The main conclusions that can be drawn from the results of this work are the following:

- during the AR binder production, the interaction of the crumb rubber particles with softer bitumens is clearly superior than with harder bitumens;

- the functional characterization of binders is more effective for bitumens than for AR modified binders, because the AR functional properties are poorly influenced by the type of base bitumen used (especially at higher temperatures);

- the AR binder results present greater dispersions in some tests, mainly because of the heterogeneity of this material;

- the 150/200 bitumen is rarely used to produce AR binders, but it greatly interacts with the crumb rubber, being a good alternative to produce AR with very good characteristics (similar to those obtained with 60/70 bitumens) during the life cycle of the pavement;

- the relation between the functional and rheological properties of the studied binders depends on the test temperature, being this relation very good for all the studied binders (base, AR and Res) at higher temperatures (dynamic viscosity and ring and ball temperature), but not so good at medium operating temperatures (penetration).

Acknowledgements

The authors would like to thank CEPSA and RECIPAV for their support and for the supply of bitumen and crumb rubber, respectively.

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Rheological Changes in Bitumen Caused by Aging and by the Interaction with Rubber

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ABSTRACT. The use of bitumen modified with crumb rubber contributes for a sustainable development of road infrastructures. However, the increasing demands on quality and durability of pavements lead to the need of a profound knowledge in the physicochemical changes of the materials which constitute these AR binders. In this work several base bitumens interacted with crumb rubber, in order to produce AR binders, which were subsequently separated, by using a modified "Basket drainage method" to recover the residual bitumen. The effect of aging during the laying procedures was also studied by using the RTFOT test. Additionally, a new "Sphere AR production simulator" method was developed to simulate the bitumen aging without the contribution of the rubber. The DSR was used to evaluate the chemical and rheological changes in the properties of AR binders. In this work, reduced changes were observed in the binder due to the interaction with crumb rubber.

KEYWORDS: Rheological changes, DSR, Molecular weight, Molecular weight distribution, "AR sphere simulator", "Basket drainage method".

1. Introduction

The use of bitumen modified with crumb rubber contributes for a sustainable development of road infrastructures through: (i) the definition of an efficient final destination for the used tires of cars and trucks; (ii) the improved performance of the resulting material used in the pavement.

However, the materials that constitute these asphalt rubber (AR) binders and the physicochemical changes in the binder during the production of the AR are not sufficiently characterized. Thus, the main objective of this study is to characterize the chemical changes in the constitution of the AR binder components (bitumen and rubber) and to evaluate their interaction at a molecular scale, which will be related to the functional properties of the binders.

Initially, the functional properties of a variety of bitumen types from the same source and obtained in the same distillation column were assessed using conventional EN standard tests, and their rheological behaviour was evaluated with a Dynamic Shear Rheometer (DSR). After being characterized, the bitumen was left to interact with 17.5% of crumb rubber by mass of AR binder. Then, a modified "Basket Drainage Method" was used to separate the residual bitumen and the rubber constituting the AR binder. The aging of AR mixtures during the mixing and laying down procedures were also studied by using the RTFOT method and a new "AR sphere simulator", which was developed to reproduce the aging of the bitumen during AR production without the contribution of the crumb rubber interaction. Finally, the changes in the chemical composition of the base bitumen, the AR binder and the residual bitumen, before and after aging, were evaluated by replicating the same test used for bitumen characterization.

The main results of this work are: (i) the identification of the chemical changes in the bitumen as a consequence of the bitumen-rubber interaction, and (ii) a better understanding of the aging influence in the AR binder and mixture production phases.

2. Literature review on bitumen-rubber interaction and aging – chemistry and rheology

The construction and maintenance of asphalt-surfaced roads has a long term and significant impact on the economic vitality of a nation. The primary reasons for the deteriorating conditions of roads include increases in overall traffic, poor asphalt binder quality coming from high-tech refining processes, and changes in climate. Better construction processes are needed to face these challenges. It is necessary to understand the fundamental behaviour and properties of the roads before beginning to develop advanced construction processes (Glover, 2007).

Presently, the pavement technology and the evaluation of bituminous mixtures are essentially based on empirical-mechanistic studies. Frequently, the materials that compose asphalt pavements are not sufficiently characterized, more specifically concerning their physicochemical constitution, their rheological properties and their mutual interaction at a microscopic scale. Despite of bitumen use since biblical times, knowledge of the structure that controls its properties and its viscoelastic nature in particular, remains incomplete (Masson *et al.*, 2006).

Bitumen is a single-phase homogeneous mixture of many different molecules, which may be differentiated into two broad classes: polar and non-polar. The non-polar molecules serve as a matrix or solvent for the polar molecules, which form weak “networks” of polar-polar associations that give bitumen its elastic properties. The polar materials are uniformly distributed throughout the bitumen, and upon heating the weak interactions are broken to yield a Newtonian fluid. When perturbed, in response to temperature changes and physical stresses, these interactions break and reform to produce a new combination of interactions (Figure 1).

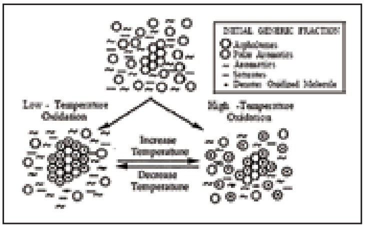


Figure 1. *The Bitumen Model (Jones et al., 1992)*

The polar molecules interact and primarily give bitumen its elastic characteristics. The non-polar molecules primarily contribute to the viscous behaviour of the bitumen and control the low temperature properties of the bitumen (Youtcheff *et al*., 1994). Pfeiffer *et al*. (1940) observed that asphaltenes, considered as the core constituents of dispersed phases, have a marked tendency to absorb aromatic hydrocarbon solvents, approaching bitumen to a colloidal system.

During its life cycle, bitumen hardens (aging) due to chemical reactions, including oxidation, polymerization and condensation and/or due to physical processes, including loss of volatiles and structural morphological changes. However, oxidative hardening has been shown to be the principal factor responsible for a more brittle structure and an increase in susceptibility to cracking. Oxidative hardening happens at a relatively slow rate and varies seasonally, occurring faster at higher temperatures as diffusion of oxygen increases (Glover, 2007).

The main ageing mechanism is an irreversible one, characterised by chemical changes of the binder, which in turn has an impact on the rheological properties. The processes contributing to this type of ageing include oxidation (Branthaver *et al*., 1993), loss of volatile components (Traxler, 1961) and exudation or migration of oily components from the bitumen into the aggregate or modifier (Curtis *et al*., 1993). The second mechanism is a reversible process called physical hardening (Bahia *et al*., 1993) and may be attributed to molecular structuring, i.e. the reorganisation of bitumen molecules or microstructures to approach an optimum thermodynamic state under a specific set of conditions (Petersen, 1984). The particles shown in the ESEM images of Figure 2 (Rozeveld *et al*., 1997) may consist of clusters of ten or more individual networks, and illustrate the relative increase in size of the asphaltenes which occurs during the aging mechanism in the bitumen.

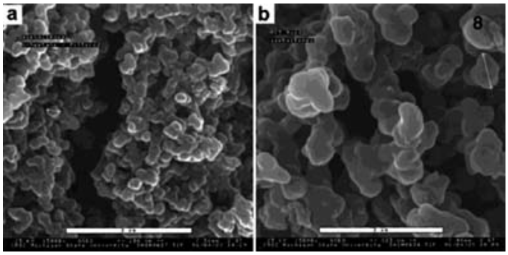


Figure 2. *Environmental scanning electron microscopy (ESEM) images of asphaltene fraction for (a) an AC-5 bitumen with 0.2-0.3 μm and (b) the aged AC-5 bitumen with 0.8-1.2 μm*

There are several available methods to investigate the physical properties of modified asphalts under aging conditions. One of the most used methods is the assessment of the binder rheology with the DSR (Lima *et al*., 2006). The rheology of bitumen at a particular temperature is determined by the constitution (chemical composition) and structure (physical arrangement) of the molecules. Thus, to understand changes in the bitumen rheology, it is essential to realize how the structure and constitution of bitumen interact to influence its rheology (Read *et al*., 2003).

When crumb rubber is blended at high temperatures with bitumen to produce a modified binder (i.e. wet process), the two materials interact once bitumen components migrate into the rubber causing it to swell. Initially, the bitumen-rubber interaction is a non-chemical reaction, where the rubber particles are swollen by the absorption of the aromatic oils of bitumen (Bahia *et al*., 1995). The impact of the crumb rubber modification improved the aging susceptibility, decreasing the binder aging ratio (Martinez *et al*., 2006). However, the base bitumen should be selected carefully to ensure that the content of light fractions of bitumen is large enough at the end of the curing process (Ould-Henia *et al*., 2006). Generally, a higher rate of mixing and curing temperature worsened the aging index, while higher rubber contents and asphalt cements with lower molecular weight improved the aging characteristics of the binder (Leite *et al*., 1999).

3. Asphalt rubber production, separation and characterisation

3.1. Bitumen selection, sample preparation and crumb rubber selection

3.1.1. Bitumen samples

Special attention was taken during the selection of the bitumens to be used in this work in order to control the variables to be studied. Therefore, all the bitumen samples were obtained from the same batch of bitumen production on the refinery (the same distillation column), since bitumen characteristics are different depending on the petroleum sources and their processing.

Asphalt rubber properties are very dependent on the used bitumen. Thus, this work was begun by choosing several grade bitumens in order to evaluate their influence on the AR characteristics. The commercial line bitumens selected to carry out this study were pen grades

10/20, 40/50, 60/70 and 150/200.

The laboratorial characterisation of sixteen base bitumen samples was carried out in order to evaluate the following properties:

- Functional properties, namely the penetration at 25 ºC (EN 1426), ring and ball softening point (EN 1427), the dynamic viscosity (EN 13302) and the elastic recovery (ASTM 5329);

- Rheological properties, using the Dynamic Shear Rheometer (only for eight of the studied bitumens) to determine the viscosity, the modulus and the phase angle;

The commercial bitumens were also subjected to other procedures in order to assess their changes during the AR binder production, and in the mixing and laying down processes, namely:

- The standard aging test rotating thin film oven test or RTFOT (EN 12607-1);

- The "Sphere AR production simulator" method developed in this work.

After these procedures, the functional and rheological properties of the obtained bitumen were assessed, and the changes in the binder characteristics were measured.

3.1.2. Crumb rubber sample

The crumb rubber used in this work was produced by the cryogenic process and, according to the supplier, was obtained by the cryogenic grinding of 30% of truck tires and 70% of car tires. The supplied crumb rubber was sieved in order to select and use only the fraction passed in sieve ASTM #20 (0.850 mm) and retained in sieve ASTM #40 (0.425 mm).

3.2. Production and separation of the asphalt-rubber binder

The method used to produce AR and collect the samples of AR, of recovered swelled rubber and of residual bitumen was the "basket drainage method" (Rahman, 2004). The AR production facility is constituted by several equipments (Figure 3) assembled in a lab ventilation chamber.

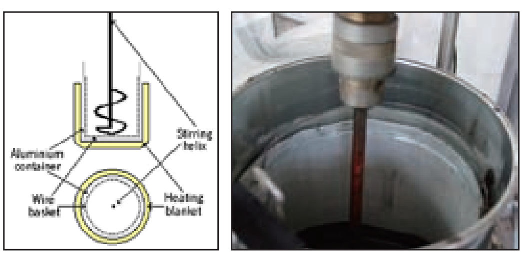


Figure 3. *Scheme and photo of the asphalt rubber production facility*

The wire basket used in the AR production facility was manufactured with a wire mesh, which was previously observed in an optical microscope to measure its average opening dimension (# 0,470 mm). The microscope was able to draw an idealized picture of the mesh in order to assess, automatically, the referred opening dimension.

The process used to produce and collect the several samples of AR, of recovered swelled rubber and of residual bitumen was the following one:

- Heating of 1 kg of each base bitumen (Bb) at 180 ºC and collecting of a sample;
- Introduction of 17.5% of crumb rubber by mass of asphalt rubber;
- Continuous heating of the asphalt rubber binder at 180 ºC for 60 minutes (digestion time) while stirring the binder at a velocity of 230 rot/min (Figure 4);
- Collection of a sample of the produced asphalt rubber (AR);
- Suspension of the wire basket with the AR binder in an oven at 180 ºC for 15 min, so as to separate its constituents and collection of a sample of the residual (Res) bitumen (Figure 4);

Figure 4. *Separation between the crumb rubber and the residual bitumen of the AR*

3.3. Reproduction of the aging during the AR production using the "Sphere AR production simulator" method

When the bitumen is heated, smaller molecules are volatilized (aromatics), while other molecules can associate themselves with bigger molecules (asphaltenes). Furthermore, when bitumen interacts with rubber, the rubber particles absorb the light parts of bitumen and swell. Since those phenomena simultaneously occur during the AR production, it seems unfeasible to evaluate the changes in the bitumen structure caused by every previous phenomenon. Hence, a new method was developed during this research in order to determine the ratio of the changes exclusively induced by heating and stirring the bitumen during the AR production, entitled "Sphere AR production simulator".

The aim of this method is to simulate the AR production, by replacing the rubber particles with metallic spheres (roughly with the same dimensions of the crumb rubber particles) in order to reproduce the evaporation and association processes occurring in the bitumen during the heating period for AR production, independently from the interaction with rubber. Before the experimental procedure, the metallic spheres were observed and measured (øeq ≈ 0.825 mm) using an optical microscope (Figure 5) and their volumetric weight was evaluated (7.75 kg/dm^3).

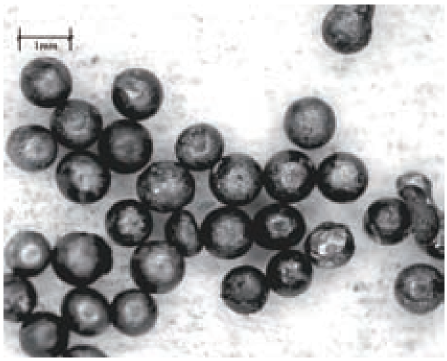


Figure 5. *Microscopic photo of the metallic spheres used to simulate the crumb rubber particles*

The “Sphere AR production simulator” was carried out in a reduced scale AR production facility (similar to the one used for AR production), and consisted in the following procedures:

- Heating of approximately 100 g of each commercial bitumen at 180 ºC;
- Introduction of approximately 240 g of metallic spheres (these spheres have a volume equivalent to 17,5% of crumb rubber by mass of asphalt rubber);
- Continuous heating of the binder (bitumen and metallic spheres) at 180 ºC for 60 minutes (digestion time) while stirring the binder at a velocity of 230 rot/min;
- Suspension of the wire basket with the binder (bitumen and metallic spheres) in an oven at 180 ºC for 15 min, so as to separate its constituents;
- Collection of a sample of the resulting bitumen (Sb) for evaluation of its functional properties and future comparison with the base (Bb) and residual (Res) bitumen properties.

3.4. Analytical determinations

During this work a series of analytical determinations were carried out in several samples of binders obtained during the AR production, namely by evaluating the penetration at 25 ºC (Pen), the ring and ball softening point (R&B), the dynamic viscosity (DV), the elastic recovery (ER) and the rheological properties using the Dynamic Shear Rheometer (DSR).

The laboratorial research carried out in this work was performed in base bitumens (Bb), asphalt rubbers (AR) and residual bitumens (Res), before and after being aged using the RTFOT (Bb, AR, Res) procedure, and with the "Sphere simulator" recovered bitumen (Sb).

3.5. Determination of the linear viscoelasticity range of stresses to be used in the rheological characterization of the binders

A DSR was used to assess the rheological properties of the base bitumens, ARs and residual bitumens. The rheological tests were carried out at different temperatures (25, 35, 50, 80, 110, 140, 160 and 180 ºC) in order to determine the rheological variation of the binder properties at all in service, laying down and mixing stages. For each temperature, frequency sweep tests between 0.1 and 10 Hz were performed in the linear viscoelasticity range, using parallel plate geometry with 40 mm of diameter and 1.0 mm of gap between plates.

Initially, it was necessary to establish the input stress at each DSR test temperature. Therefore, previous stress sweep tests were performed at 1.0 Hz to determine the linear viscoelasticity region (at each test temperature). The linear viscoelasticity range is the range of stresses where no change is observed in the values of the rheological characteristics measured at each test temperature. The selection of the stress values to apply on the rheological tests at each temperature (1000 Pa at 25 and 35 ºC, 200 Pa at 50 ºC, 40 Pa at 80 ºC, 30 Pa at 110 ºC, 20 Pa at 140 ºC, 10 Pa at 160 ºC and 3 Pa at 180 ºC) was made to guarantee that the frequency sweep tests will be carried out in the linear viscoelasticity region for all tested binders, thus being possible to compare their rheological behaviour.

Rheological time sweep tests were also performed, at a constant temperature, stress and frequency, to confirm that no structural modifications occurred in the binders throughout the tests (60 minutes were used to evaluate the changes during AR production). The rheological characteristics assessed in the DSR were the complex modulus (G*), storage or elastic modulus (G'), loss or viscous modulus (G''), complex viscosity (η*) and phase angle (δ).

4. Results and discussion

4.1. Functional characterization of the studied bituminous materials

To understand the changes induced by the aging process in the base bitumens and in the AR binders, these materials were tested with the RTFOT method. The variations of the different parameters measured before and after the RTFOT are presented in Figure 6.

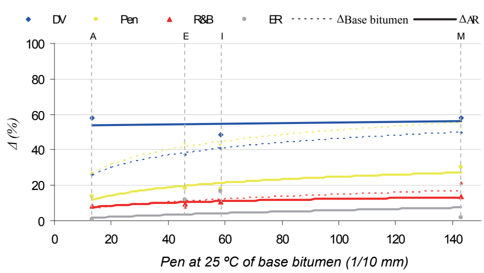


Figure 6. *Comparative plot of the functional properties of the base bitumen and AR binder before and after the RTFOT procedure*

The viscosity was the main property altered during the RTFOT aging test of the base bitumens and AR binders. Furthermore, all the functional properties showed superior variations for the softer base bitumen 150/220 (M), suggesting that the aging process produces more alterations on the softer base bitumens than in the harder ones. These results confirm that during the aging of bitumens, the smaller molecules that confer them softness are volatilized or associate in bigger molecules that also are oxidized, and as result bitumens become stiffer.

The characterization of the AR binder with the dynamic viscosity test (EN 13302) presented some dispersion of the results, mainly due to the heterogeneity of the AR binder and the different density of its components. The AR binder property that was less affected by the RTFOT aging was the elastic recovery, being possible to observe an improvement in this characteristic after the aging process. Moreover, it was noticed that the elastic recovery of the AR produced with the harder and the softer bitumens are almost constant after the RTFOT aging.

The changes of the functional properties after the RTFOT aging are smaller in the AR binder than in the base bitumen, except for the dynamic viscosity. These observations suggest that the AR binder is less sensitive to aging, during the mixing and compaction phases, than the base bitumen. However, during the AR production the base bitumen is already “aged” and the RTFOT favours the continuation of the rubber swelling. Since the residual bitumen surrounding the rubber particles has characteristics different from the base bitumen, with the present data it is difficult to conclude if the rubber protects the AR binder from aging.

The penetration and ring and ball tests are carried out at temperatures below 100 ºC, at which occurs a relaxation process in the bitumen. Thus, after RTFOT aging the referred properties are mainly affected by the bitumen behaviour. In contrast, the dynamic viscosity values are obtained for high temperatures, at which the bitumen behaves as liquid oil. In these conditions, the changes in the behaviour of AR binders after RTFOT aging are mainly controlled by the rubber.

The functional properties of several bitumen samples were assessed in different stages of the AR binder production process, as well as after the application of the developed “Sphere AR production simulator” method. The differences between the functional properties of the processed and base bitumens are plotted in Figure 7.

The graphic on the left side of Figure 7 presents the percentage differences in the functional properties caused in the base bitumens by the “Sphere AR production simulator” method, the AR production process (residual bitumen) and by the RTFOT aging procedure. The process that caused fewer changes in the functional properties of the base bitumens was the “AR Sphere simulator” method, being the most influential the AR production. In order to evaluate the changes caused exclusively by the rubber in the base bitumen, it was necessary to deduct the changes strictly caused by the solubilisation, association and oxidation phenomena that occur during the AR production, which are simulated by the “Sphere AR production simulator” method. Thus, the shadow areas of Figure 7 show the percentage alterations caused by the rubber, which have a considerable impact in the final residual bitumen.

Furthermore, to evaluate the aging process in the AR mixtures during the mixing and laying down phases, in comparison to the conventional mixtures, the changes in the base bitumen should be compared with the changes in the residual bitumen (the shadow areas in the graphic on the left side of Figure 7), since this is the effective binder of the AR mixtures. It was observed that the aging influence on the AR residual bitumen is visibly minor than on the base bitumen.

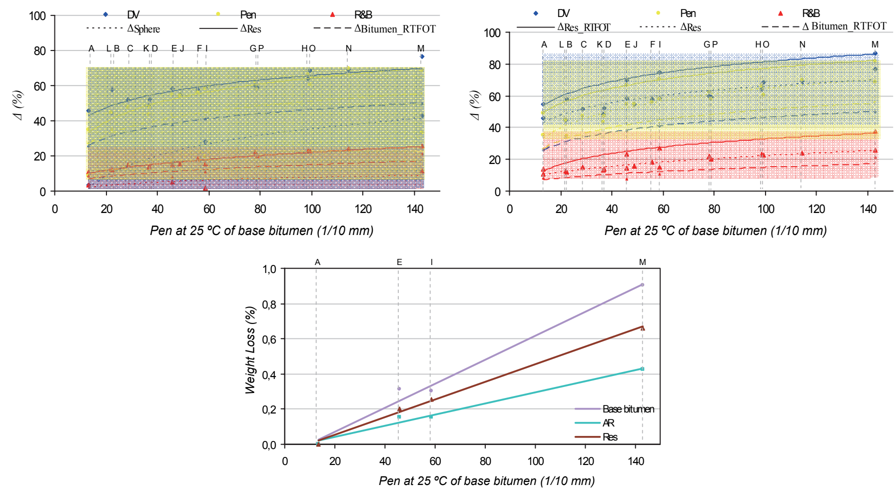


Figure 7. *Comparative plot of the functional properties and weight loss for the "sphere", base and residual bitumens before and after RTFOT aging*

One of the results obtained in the RTFOT aging method is the percentage of bitumen weight loss (graph below in Figure 7). The variation of this parameter with the penetration of the base bitumen is a linear trending line with positive slope. The mass loss increased for softer bitumens, being the base bitumens the most influenced by aging, followed by the residual bitumens and finally by the AR binders. It was concluded that the rubber protects the AR binder from aging changes, namely because the AR is less affected by RTFOT aging than the residual bitumen.

4.2. Rheological characterization of the studied binders

Initially, based on the rheological characteristics of the base bitumen I at 1.0 Hz, Figure 8 presents the changes caused in the bitumen during the AR production and by the RTFOT aging.

Below 100 ºC, and except for tgδ, all rheological parameters have higher values after the RTFOT aging due to the increase of the binder stiffness. For this range of temperatures the elastic modulus of the bitumen increases more rapidly after aging, showing that the asphaltenes are responsible for the elastic characteristics of the bitumen. Above 100 ºC, after the relaxation of the asphaltenes, the rheological properties of the base and aged bitumens are similar. The aged AR binder presented an unusual evolution of G'' and G*/sen(δ), mainly for temperatures above 80 ºC. The referred behaviour is caused by the great increase of the elastic modulus of this material, which is even higher than the viscous modulus. This phenomenon is not usual for the other studied materials at temperatures above 80 ºC, indicating that the aged AR binder is clearly the stiffer and most elastic material evaluated in this work. At these temperatures the rheology of the AR binder is largely controlled by the behaviour of the crumb rubber particles.

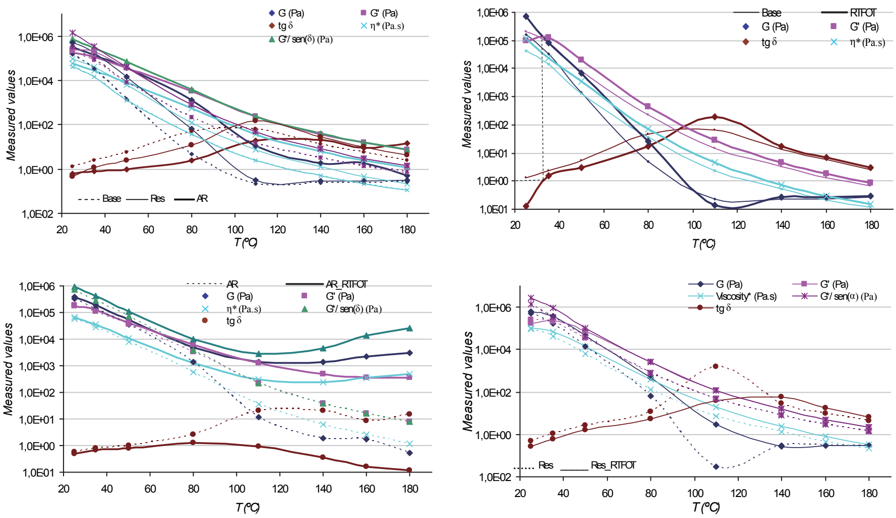


Figure 8. *Comparative plot of the rheological properties at 1.0 Hz for the base bitumen I and the corresponding AR binder, residual bitumen and RTFOT resulting materials*

However, for temperatures inferior to 80 ºC the rheological properties of the aged AR binder are only slightly higher than for the initial AR binder. At these temperatures the rheology of the AR binder is dominated by its bitumen component.

For temperatures above 110 ºC the G' values of the aged AR binder increase fairly with the temperature, while the G'' values present a moderate decrease, originating very small values of tgδ and the increase of the complex viscosity values. Since the bitumen is totally liquefied at these temperatures, the referred only occurs because the rubber particles create an attrition force over the plates opposing their movement.

In order to assess the effect of RTFOT aging in the four commercial bitumens, rheological tests were performed in these bitumens after RTFOT aging and the results were compared with the corresponding base bitumens. Figure 9 presents the isochronal curves obtained for the different rheological parameters and the differentials between each base and aged bitumen.

The most significant rheological variations between the base and aged bitumen occurred for the softer bitumen M (150/220). In opposition, the harder bitumen A showed the inverse tendency, especially at lower temperatures. The rheological parameter that was more influenced by the RTFOT aging was the storage modulus, especially below 100 ºC. At these temperatures the variations of G' were superior to 80 % for all studied bitumens. This change is also reflected on the change of tgδ, which also presents significant variations (around 60 %) after aging.

The shape of the curves representing the rheology of the studied bitumens is nearly the same after aging, but is moved up and to the right (due to the higher viscosity after aging).

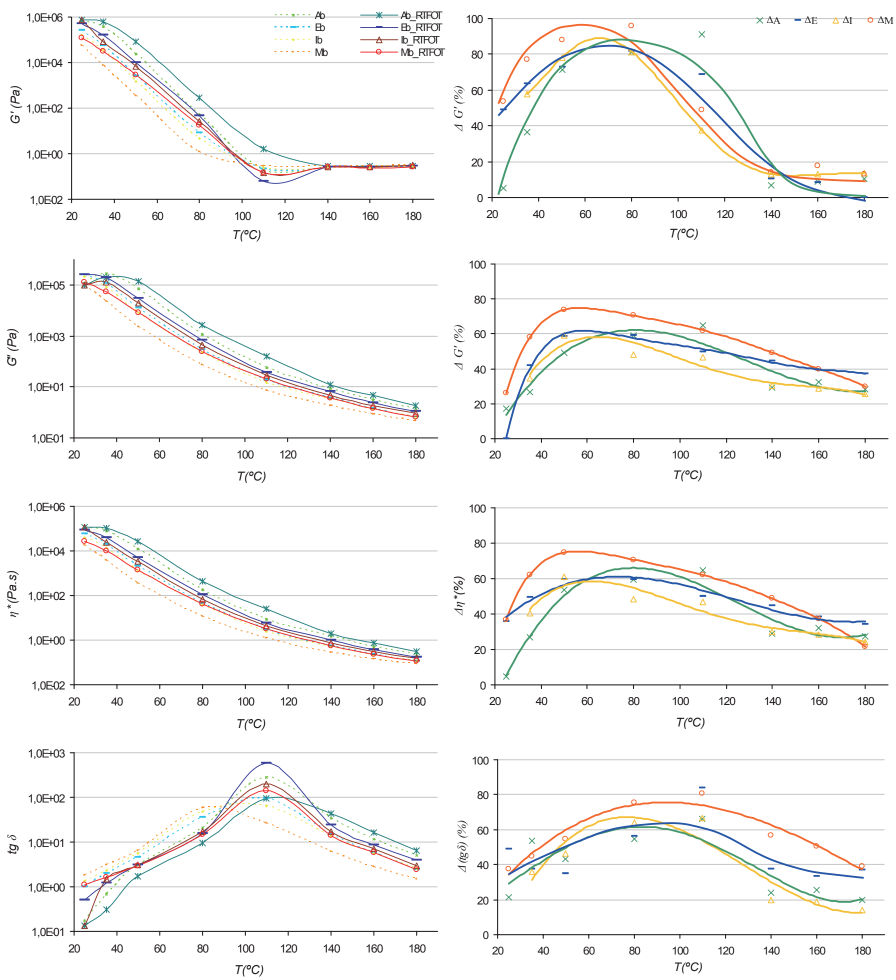


Figure 9. *Comparison between the rheological properties of the base and aged bitumens*

The temperatures at which the maximum values of tgδ occurred after aging have also moved to the right (higher temperatures), for higher values of tgδ, reflecting the presence of bigger molecules in a superior concentration after the aging of the base bitumens.

In order to assess the effect of the RTFOT aging in the initial AR binder produced with the four commercial bitumens, rheological tests were performed in these AR binders after RTFOT aging and the results where compared with the corresponding initial AR binders. Figure 10 presents the isochronal curves obtained for the different rheological parameters and the differentials between each initial and aged AR binder.

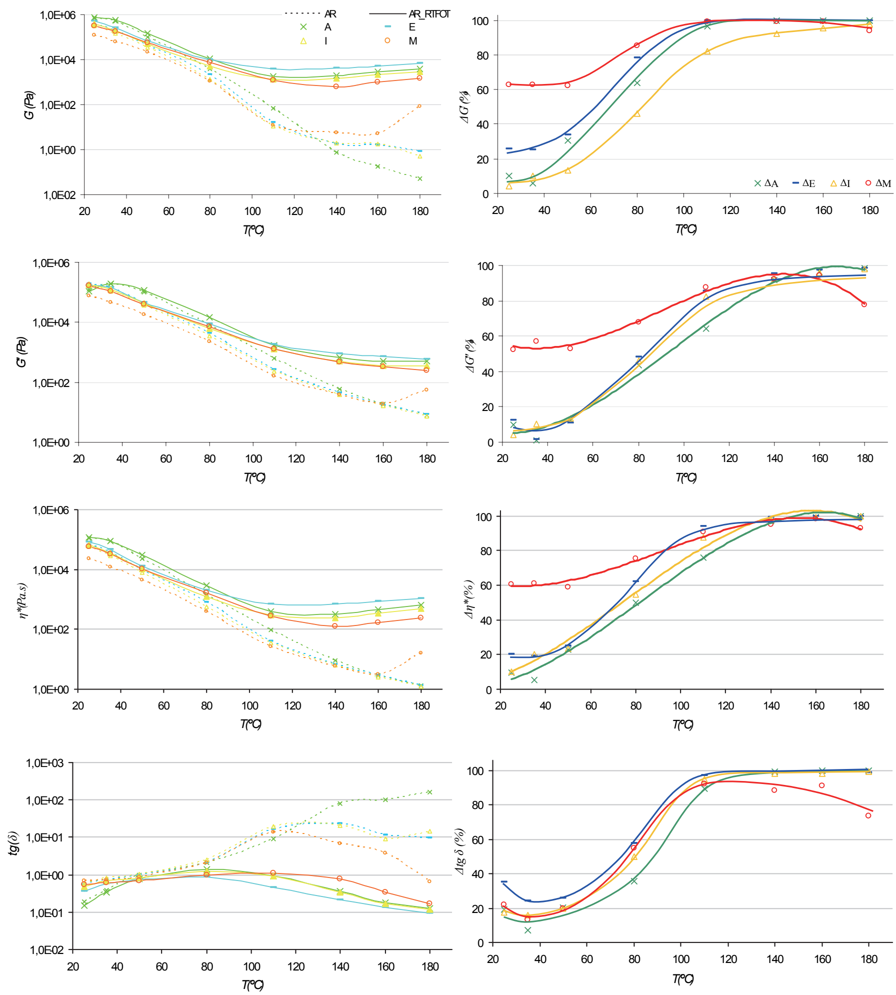


Figure 10. *Comparison between the rheological properties of the initial and aged AR binders*

The evaluation of the changes in the rheology of the aged AR binder should be divided in two phases, above and below 100 ºC. From 25 to 80 ºC there are few changes in the behaviour of the AR binder due to aging, since the shape of the curves describing the rheological behaviour of the initial AR and the aged AR binders is similar (even if some stiffening was noticed). At these temperatures the behaviour of the AR is controlled by the hardened bitumen that encloses the rubber particles in its semi-rigid structure, restraining their movements. Moreover, the behaviour of the swelled rubber particles is influenced by the bitumen within its reticulated structure.

Between 80 and 110 ºC, an alteration is noticed in the aged AR binder behaviour, which coincides with the beginning of the relaxation process of the bitumen structure. After this temperature, it is possible to observe a clearly different behaviour affected by the rubber. During the aging of the AR binder using the RTFOT several processes took place, such as:

- The crumb rubber particles continue to swell since during the AR binder production they were not totally saturated and thus, the volume fraction of the rubber particles after aging of the AR binder is clearly superior to its value in the initial AR binder;

- The residual bitumen surrounding the rubber particles is subjected to a more intense aging than the base bitumen, since it is exposed to the same test conditions in a thinner layer;

- In the presence of sulphur, vulcanization and polymerisation processes may take place in and within the rubber particles during the aging test, as well as during the AR binder production;

- The rubber particles that have swollen into the saturation point start to depolymerise and break by the action of temperature and pressure of the bitumen molecules held in its reticules;

- At the end of the RTFOT aging, it is almost impossible to drain the AR binder from the containers, since bitumen is entrapped between the rubber swelled particles.

The referred processes mainly occur due to the chemical characteristics of the rubber. In fact, during the AR binder production and later during the AR mixing and laying down phases, the rubber particles are exposed approximately to the same conditions of tire rubber production. In the presence of vulcanization and polymerization inducers, which were not totally consumed in the primary reaction, the broke and un-vulcanized rubber chains can be polymerized/vulcanized.

The results for temperatures superior to 100 ºC confirm the previous analysis. The presence of the bitumen in the aged AR binder is imperceptible, because at these temperatures the bitumen is softer than the swelled rubber. Furthermore, the interaction between the rubber and the bitumen and the reactions among the swelled rubber particles, initiated during the AR binder production and continued in the RTFOT aging, resulted in a material controlled by the rheological behaviour of the rubber, with values of G' visibly superior to the ones of G''.

Evaluating the graphs of G' and η^* between 140 and 180 ºC, it is perceptible an increase of the rheological parameters with the temperature, mainly due to the reduction of viscosity of the residual bitumen (allowing the rubber particles to control the rheology or the AR binder) and to the dilatation of the links in the rubber structure and increased amplitude of vibration of the atoms induced by the temperature.

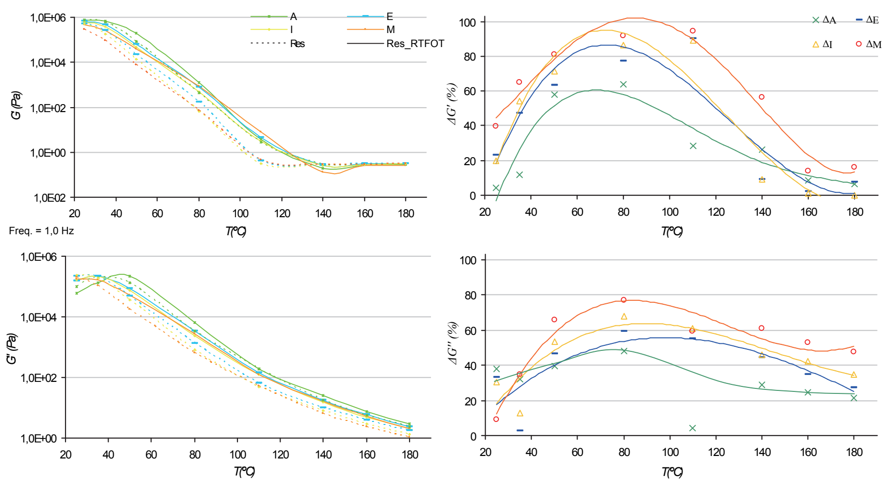
Some differences in the rheological behaviour of the aged AR binder were noticed, considering the type of base bitumen used to produce the AR binder, namely:

- The AR produced with the softer bitumen (M) showed the highest variation between the rheology of the initial and aged AR binder;

- The smallest variation between the aged and initial AR binder was verified for the AR binder produced with the harder bitumen (A), except for the elastic modulus (G') property.

At temperatures below 80 ºC, the elastic modulus was the AR rheological property with greater variation after the RTFOT aging. The rheological isochrones of the four aged AR binders present the same shape and little divergence. The highest values for the rheological parameters (except for tgδ) were observed for the aged AR produced with bitumen E, while the lowest values were obtained for the aged AR with bitumen M. The greatest increase in the variation of the rheological parameters due to the RTFOT aging of AR binders occurs between 50 and 110 ºC. Above 110 ºC, the variation of the rheological properties upholds a value near 100 %.

During the AR mixing and lay down processes two phenomena occur simultaneously in the bitumen, the aging of the residual binder and the extension of the diffusion of parts of bitumen into the rubber. Therefore, to assess exclusively the effect of aging, RTFOT tests were carried out in the residual bitumen obtained from the initial AR binder by the basket drainage method. Figure 11 presents the rheological properties of the residual bitumen before and after RTFOT aging, as well as the percentage differences between them.



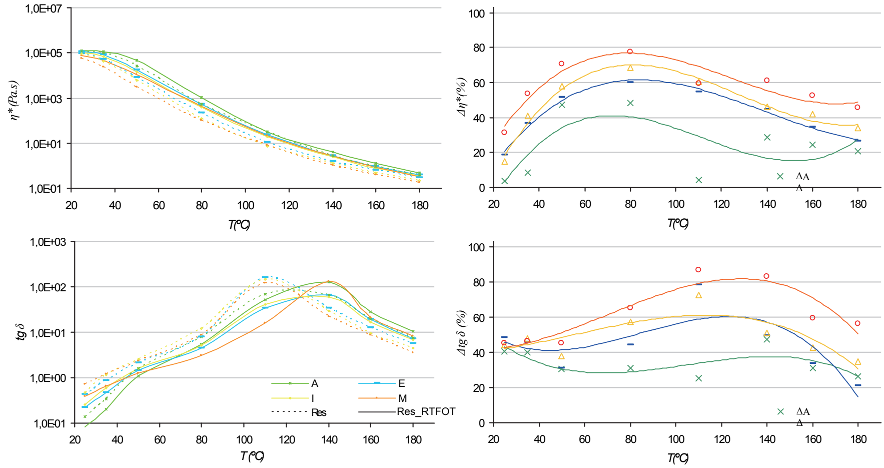


Figure 11. *Comparison between the rheological properties of the residual bitumens before and after RTFOT aging*

According to Figure 11, the rheology of the residual bitumens before and after RTFOT aging is similar to the one of the base and aged bitumens (presented previously), even if the rheological properties of the residual bitumen have presented slightly higher values, except for tgδ.

Finally, it was observed that the rheological properties of the aged residual bitumens are higher than the aged AR binders for temperatures inferior to 80 ºC (when the behaviour is controlled by the bitumen), thus being the residual bitumens stiffer than the corresponding AR binders before and after aging. In addition, at the referred temperatures (< 80 ºC) the variations of the rheological parameters of the residual bitumens before and after RTFOT aging are greater than the observed for the AR binders, confirming the previous observations that the presence of crumb rubber particles in the AR binder reduces the effects of aging in the bitumen.

4.3. Changes of the bitumen during the AR production and caused by aging

The rheological study of the materials is a useful tool to assess their structure. During the AR binder production, AR mixing and lay down phases the bitumen suffer chemical and structural changes that can be monitored using the DSR. In fact, the rheological properties will be plotted differently in this section to better illustrate the chemical changes that occur in the binders. Initially, Figure 12 presents the isochrones and isotherms of tgδ for the base and aged bitumen I.

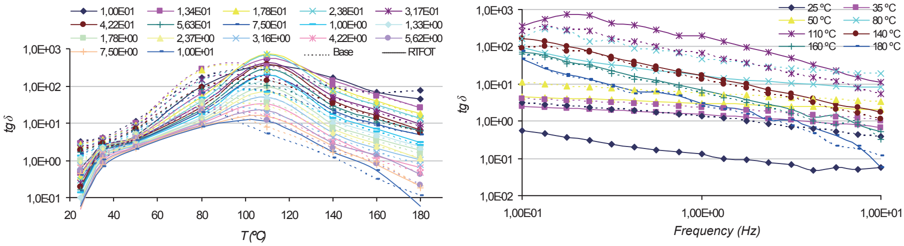


Figure 12. *Isochrones and isotherms of tgδ of the base and aged bitumen I*

In the left graphic of Figure 12 (tgδ vs. temperature) the maximum value of tgδ is obtained at a higher temperature after aging. This occurs because the proportions between the viscous (G'') and the elastic (G') stiffness of the bitumen, at different temperatures, change due to the RTFOT aging. Comparing the tgδ isotherms of the base and aged bitumen I (right side of Figure 13), it is perceptible that the aged bitumen presents lower values of tgδ for temperatures below 100 ºC, due to a more significant increase of G' in comparison with G'' after aging. For temperatures above 100 ºC the values of tgδ are quite similar between the base and aged bitumen. According to the bitumen model (Jones *et al*., 1992), the relaxation of the bitumen shall be inherent to the asphaltenes and resins molecules, since they grow and come together during the RTFOT aging, thus being responsible for the increase in the elastic stiffness of the aged bitumen. This can also be observed in the Cole-Cole diagrams of the base and aged bitumen I (Figure 13).

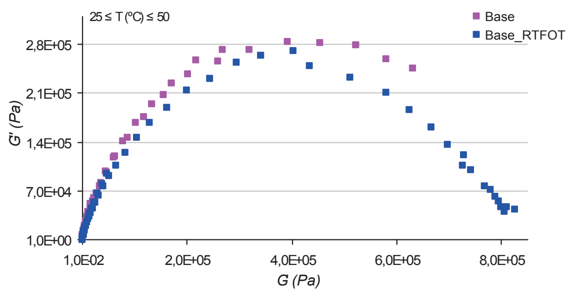


Figure 13. *Cole-Cole diagram for the base and aged bitumen I*

As mentioned before, the Cole-Cole diagrams are a useful tool to identify changes in the bitumen at medium in service temperatures. The more obvious change in the rheological characteristics of the aged bitumen occurred at 25-35ºC, with a great increase in G' values combined with a decrease in G'' values. Thus, at this range of temperatures the behaviour of the aged bitumen is essentially elastic and is controlled by the asphaltenes and resins in the bitumen.

The evolution of the values of tgδ for the aged AR binder in comparison with the initial AR binder is presented in Figure 14 in order to assess the structural changes in this binder.

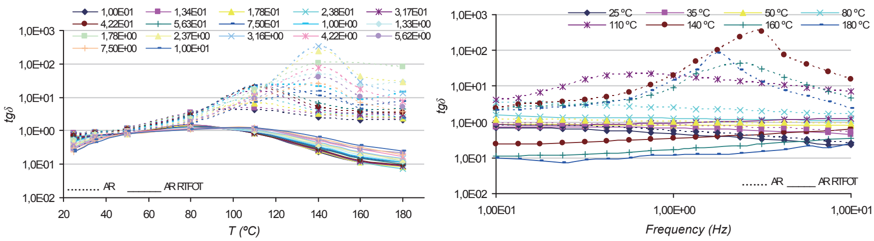


Figure 14. *Isochrones and isotherms of tgδ of the initial and aged AR binder produced with base bitumen I*

In the previous studied materials the isochrones of tgδ present a obvious maximum and the isotherms present a sudden change in slope, for temperatures at which the bitumen changes its behaviour from mainly an elastic material to a viscous material (Tc). However, the isochrones of the aged AR binder merely present a smooth maximum and only a small variation is visible in the slope of its isotherms after RTFOT aging. The values of tgδ for the aged AR binder for temperatures inferior to 80 ºC are similar to the initial AR binder and coherent with behaviour of the base and aged bitumen (because the behaviour is controlled by the bitumen). However, for temperatures above 80 ºC the values of tgδ for the aged AR binder initially stabilize and then decrease due to the increase of G' (because the behaviour is controlled by the rubber).

The relation between the storage and the loss modulus of the aged and initial AR binder, at in service temperatures, is represented by the Cole-Cole diagram shown in Figure 15.

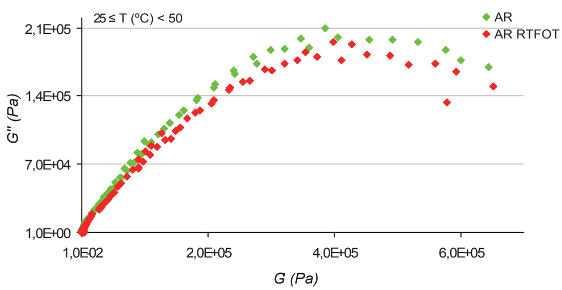


Figure 15. *Cole-Cole diagram for the initial and aged AR produced with base bitumen I*

The initial and aged AR binder have a similar behaviour, even if the line representing the aged AR binder shows that this is a harder material (higher values of G*). Moreover, the ratio between the viscous and the elastic stiffness is inferior in the aged AR binder, pointing to a special increase in the elasticity of the material during the RTFOT aging with impact in this range of temperatures, and consequently in the performance of the pavement in service.

The isochrones and isotherms of tgδ for the residual bitumen I before and after aging are presented in Figure 16.

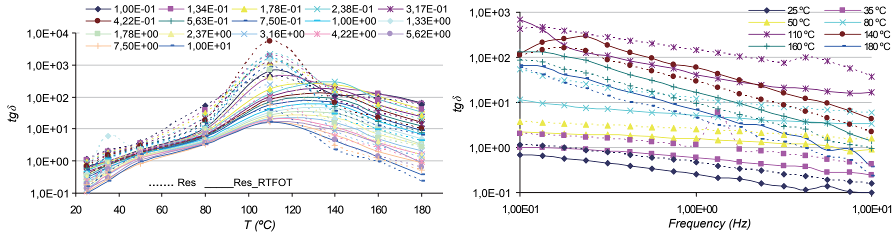


Figure 16. *Isochrones and isotherms of tgδ of the residual bitumen (extracted from the AR binder produced with base bitumen I) before and after aging*

The behaviour of the residual bitumens before and after aging is similar to the one of the base and aged bitumens (presented previously). The upper limit formed by the isochronal lines dislocate to the right, at higher critical temperatures. This phenomenon is reflected in the isotherm lines by the temperature at which the change of slope of the lines occur, between 50 and 80 ºC for the residual bitumen before aging and between 80 and 110 ºC after aging. This change of behaviour is due to the variation of the molecular structure of the residual bitumen. When the critical temperature is achieved, a relaxation of the molecular structure of the bitumen occurs, and the weak interactions that maintain the "networks" formed by the polar molecules of bitumen (asphaltenes and resins) are broken. Thus, the colloidal behaviour becomes a Newtonian fluid behaviour as the asphaltenes are peptized by the resins being totally dispersed in the maltenes phase (non-polar).

Figure 17 presents the changes that occur in the viscoelastic character of the residual bitumen before and after aging at in service temperatures (25 ºC ≤ T < 50 ºC).

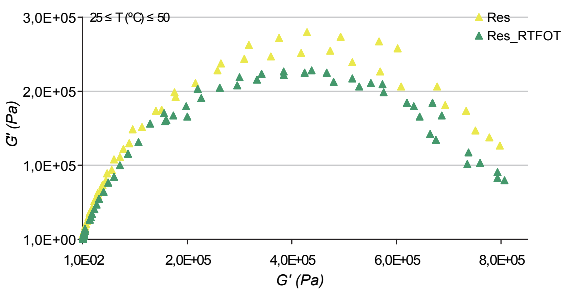


Figure 17. *Cole-Cole diagram for the residual bitumen (extracted from the AR produced with base bitumen I) before and after aging*

The changes in the viscoelastic characteristics of the residual bitumen induced by the RTFOT aging indicate a stiffer aged material due to a reduction in G'' and a slight increase in G', caused by the volatilization of some bitumen molecules and the increase of the particle size of other.

The performance of the studied binders in the pavement can be obtained by analysing the Cole-Cole diagrams of the binders before and after aging (Figure 18), where the changes of behaviour can be clearly assessed at the lowest tested temperatures (25 and 35 ºC).

All aged base bitumens showed an increase of G' and a reduction of G'' at each frequency (upper left side graphic of Figure 18). The lowest variation after aging occurs for bitumen A and the inverse for bitumen M, as expected. The similarity between the form and the evolution of the different curves of the aged bitumens indicates that the RTFOT is a process that approaches the distillation process by which the different grade bitumens are produced.

Aging has a different impact over the diverse initial AR binders (upper right side graphic of Figure 18). The effect of the RTFOT aging over the initial AR binders, between 25 and 50 ºC, caused some changes on the relation between the elastic and viscous stiffness of the tested materials. The Cole-Cole curves of the aged AR binders produced with bitumen E and I show a similar form to the Cole-Cole curve of the corresponding initial AR binders, even if a small translation of the curve towards down and right means that the material gets stiffer. In fact, the interactions between the rubber and bitumen, initiated in the production of the AR binder, will continue during the RTFOT aging, since the rubber particles did not achieved the saturation.

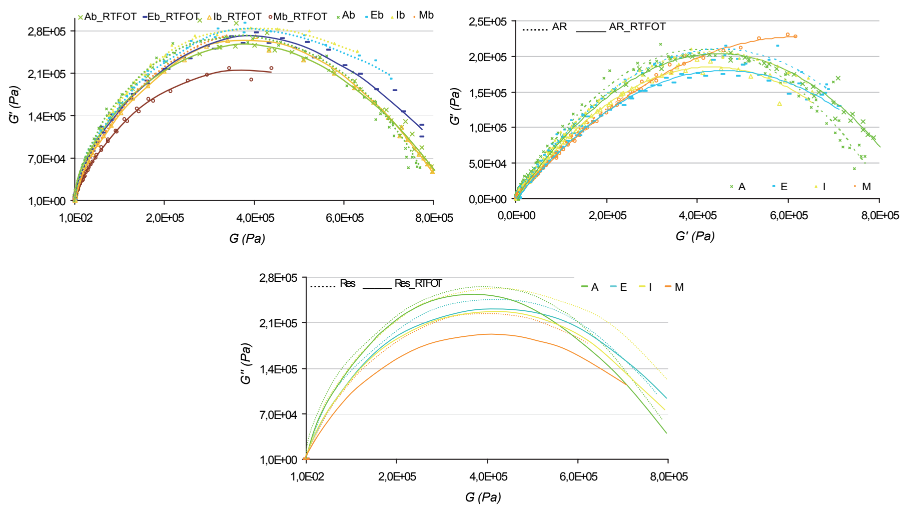


Figure 18. *Comparative Cole-Cole diagrams for the commercial base bitumens, AR binderss and residual bitumens before and after aging*

The Cole-Cole curves of the AR binder produced with bitumen A, before and after the RTFOT aging, cross themselves up and no hardening is detected in the aged material at medium in service temperatures (25 to 50 ºC). This can be explained because during the production of the AR binder almost all the small molecules contained in the base bitumen A (10/20) have interacted with the rubber particles. Therefore, in the RTFOT aging test the bitumen-rubber interaction was very low.

As it would be expected, the properties of the AR binder produced with bitumen M changed significantly during the aging process. Since the residual bitumen of the AR binder produced with bitumen M still have a considerable amount of small and volatile molecules, they will continue to diffuse into the rubber particles during the RTFOT aging until their saturation (gel rubber). Then, the particles of gel rubber fracture, releasing little pieces that increase the volume fraction of the gel rubber in the AR binder, but also reduce the stiffness of the binder at medium in service temperatures (at these temperatures the behaviour is controlled by the bitumen).

Finally, some dispersion can be observed in the Cole-Cole curves of the residual bitumens before aging, due to the different effect of the rubber in the studied bitumens. However, the influence of RTFOT aging in the behaviour of the residual bitumens is consistent with the previous analyses presented for the base bitumens (the aged residual bitumens are stiffer). Despite the average values of G'' for the residual bitumens are superior to the ones of the base bitumens and inferior to the ones of the AR binders, they always present the higher average values of G' (before and after RTFOT aging). Thus, the residual bitumens are the stiffest materials studied in this work, which shows the great changes in the chemical structure and composition of the bitumen during the production of the AR binder.

5. Conclusions

The main conclusions that can be drawn from the results of this work are the following:

- The "Sphere AR production simulator" method was found to be a simple and effective method to determine the changes induced by the heating and stirring of the bitumen during the AR binder production without considering the interaction with the rubber particles;

- The assessment of the functional properties indicates that the aging process causes more alterations on the softer base bitumens than in the harder ones;

- The elastic recovery is the functional property of the AR binder less affected by aging, and this property improves after the RTFOT aging process;

- The functional properties of the AR binder are less affected by aging than to the ones of the base bitumen. These observations suggest that the AR binder is less sensitive to aging than the base bitumen during the mixing and lay down phases;

- The rubber particles continue the swelling process during the AR binder aging, because they are not totally saturated after the AR binder production;

- The interaction between the rubber and the bitumen and the reactions among the swelled rubber particles, initiated during the AR binder production and continued in the RTFOT aging test, result in a material controlled by the rubber rheological behaviour for temperatures superior to 110 ºC (with G' values visibly superior to the values of G'') and by the bitumen rheological behaviour for temperatures below 110 ºC;

- The rheological properties obtained at temperatures below 80 ºC showed that the highest variation of the AR binder characteristics after aging was obtained for the AR binder produced with the softest bitumen M (150/220) showed, while the smallest variation was verified for the AR binder produced with the hardest bitumen A (10/20);

- At temperatures inferior to 80 ºC, the changes in the rheological properties of the residual bitumens after RTFOT aging are greater than the ones observed for the AR binders, indicating that the presence of the rubber particles in the binder reduces the aging processes;

- There is a relaxation phenomenon in the bitumen structure that occurs at a characteristic temperature (between 80 and 140 ºC) and is related to the change in the material behaviour, representing the change between the viscoelastic behaviour of the bitumen (below that temperature) and the behaviour of a Newtonian fluid (above that temperature);

Acknowledgements

The authors would like to thank to CEPSA and to RECIPAV for their support and for the supply of bitumen and crumb rubber, respectively.

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Study on the Reduction of the Viscosity of Asphalt Rubber

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ABSTRACT: It is profound to reduce the viscosity of asphalt rubber (AR) at high temperature for easy processing, transportation and application. The effects of several additives at two typical blending temperatures (175 ℃, 190 ℃) on the viscosity and the softening point of AR were investigated. The results showed that, the viscosity and the softening point of AR were obviously decreased after being mixed at 190 ℃ for 45minutes, compared to those mixed at 175 ℃. Wax with high melting temperature (Tm) decreased the viscosity of AR, but increased the softening point. Reactive agent could efficiently decrease the viscosity of AR, therefore, it can be used to decrease the blending temperature.

KEYWORDS: Reduction, Viscosity, Reactive agent, asphalt rubber (AR)

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1. Introduction

According to the definition of ASTM D8, asphalt rubber (AR) is one kind of blend processed at high temperature (≥180 ℃) (Sun, 2007). AR has a series advantages such as low temperature susceptibility, good elastic recovery and age resistance. The application of AR could reduce the thickness of the road and extend the life of the road building. In the other hand, the use of crumb rubber in the road can reduce the cost of asphalt modification and solve the solid pollution of the used tire, and take effect in environmental protection.

There are a lot of researches and applications about AR, the properties of AR were dependent highly on the size and the content of crumb rubber, the processing technology, and asphalt composition (Huang, 2006; Navarro, 2007). All these studies indicated that the viscosity at high temperature is the key factor influencing the quality of AR (Huang, 2006).

The processing temperature of AR is usually above 180℃, and the viscosity of AR is quite high, about 10 Pa·S at 135℃. For the mixing and laying of the asphalt mixer, the viscosity of modified asphalt should be below 3 Pa·S. The high processing temperature and high viscosity cause a series defects of AR, such as the higher requirement of equipment, the better age-resistance of the base asphalt, the more exposure of pollution gas and higher energy consumption. In the meantime, the high viscosity causes serious problems of transporting and laying. Therefore, the reduction of the viscosity and the processing temperature of AR became an important issue.

In this paper, several additives were used to reduce the viscosity of AR, and the effect of these additives on the viscosity at two temperatures was investigated.

2. Experimental

2.1. Materials

Crumb rubber, tread rubber from waste tires, 30 mesh, was provided by Shanghai Xiaoyou Reclaim Rubber Factory. Asphalt, Liaohe AH-90, was produced by Panjin Liaohe Petrochina Asphalt Co. Ltd. Additive, trans-octane rubber (TOR) (Vestnamer), was produced by Degussa Co. Ltd. PE wax, (melting temperature of 105 ℃) was produced by Shanghai Huayi Agent Company. Reactive agent, activator 420, was provided by Shanghai Xiaoyou Reclaim Rubber Factory.

2.2. Processing Method

Composition: 20 wt % crumb rubber, 80 wt % asphalt. Additive content is weight content of the AR. Processing temperature: the base asphalt was heated to the set point of temperature, and then 20 wt % of crumb rubber was then added in.

Method 1: mixed at 175 ℃ for 45 minutes.

Method 2: mixed at 190 ℃ for 45 minutes.

2.3. Measurements

Conventional properties of softening point and rotational viscosity was tested according to Chinese Road Engineering Asphalt and Mixture Test Procedures (JTJ052-2000), and respectively, guideline of T0606-2000 for softening point and T0625-2000 for rotational viscosity.

3. Results and Discussion

The size of crumb rubber particles increased when it absorbed the maltene from the asphalt. The volume of solid particles increased and the viscosity of the left asphalt also increased, which led to the increase of viscosity of AR (Huang, 2006). Therefore, the viscosity of the AR could be increased by decreasing the viscosity of continuous phase and the friction between the crumb rubber particles. The solid portion can be reduced by the degradation of crumb rubber. The degradation part should be controlled properly, or else, the extreme degradation will cause poor properties of the AR.

PE wax is different with the wax in the asphalt, which has high molecular weight and high melting temperature. It has been used in the asphalt industry (Edwards, 2007).TOR is trans-Octane rubber (Sun, 2007), which has a low crystallinity and low melt temperature (60 ℃), it was used in the rubber industry to reduce the processing viscosity and increase the hardness of the compound. The reactive agent is a kind of desulphurization agent, it can accelerate the degradation speed of the crumb rubber and enhance the interaction between asphalt and crumb rubber. In the following study, the effect of TOR, PE wax and the reactive agent on the properties of AR were investigated.

3.1. Effect of different additives on the viscosity of AR mixed at 175 ℃

As seen from Table 1, AR has an obvious shear thinning behavior (non-Newton behavior).

Table 1. *Effect of different additives on the softening point and viscosity of AR mixed at 175* ℃

additive Content (wt %)	**Softening point/ ℃**	**Viscosity/ Pa•S (177 ℃)**			
		1rpm	5rpm	10rpm	20rpm
0	70.0	55.0	28.0	21.0	11.0
1%TOR	70.5	78.0	29.0	16.0	10.0
1%Wax	71.0	43.8	16.0	11.0	8.0
2%Wax	73.5	44.3	16.0	12.0	9.0
0.5%Reactive agent	65.5	24.0	14.0	9.0	5.0

TOR and PE wax indeed decreased the viscosity of AR. Compared with TOR, PE wax had a greater effect. With the addition of TOR, the viscosity decreased obviously at high speed. In comparison with TOR, 1% wt PE wax decreased the viscosity of AR to a greater extent. The reduction effect changed a little with the increase of PE wax. It can be also seen from Table 1, 0.5 % reactive agent sharply decreased the viscosity of AR, the viscosity reduced to the half of the original viscosity.

Table 2. *Effect of blending time on the softening point and viscosity of AR mixed at175 ℃*

Blending time/min	**Softening point/ ℃**	**Viscosity/ Pa•S (177 ℃)**			
		1 rpm	5 rpm	10 rpm	20 rpm
45	70.0	55.0	28.0	21.0	11.0
60	68.5	44.0	18.3	14.0	9.0
120	68.0	42.0	16.8	13.0	8.6
180	66.5	40.0	15.8	12.2	8.3

Table 2 showed the effect of blending time on the softening point and viscosity of AR mixed at 175 ℃. From Table 2, the viscosity and the softening point decreased slowly, however, the viscosity was still very high (over 8 Pa•S) at 177 ℃, which did not meet the application requirements of modified asphalt (Navarro.F.J, 2007). It indicated that extending blending time at 177 ℃ could not effectively decrease the viscosity of AR.

As seen from Table 3, the effect of reactive additive on the reduction of viscosity was obvious, and the decreasing trend became obvious with blending time, and then became smooth after 2 h. The blend satisfied the standard specification after 1～2 h reaction time. The effect of additive on the softening point of AR indicated that the softening point decreased in the same trend with the viscosity of AR.

Table 3. *Effect of blending time on the softening point and viscosity of AR mixed at 175 ℃ in the presence of 0.5 wt % reactive agent*

Time/min	Softening time/ ℃	Viscosity/ Pa•S (177 ℃)			
		1 rpm	5 rpm	10 rpm	20 rpm
45	65.5	24.0	14.0	9.0	5.0
60	62.5	17.3	7.4	5.4	4.5
120	61.5	11.0	5.7	3.8	2.7
180	60.5	11.0	5.1	3.8	2.4

3.2. Effect of the additive on the reduction of viscosity of AR mixed at 190 ℃

Table 4. *Effect of the kind of additive on the softening point and viscosity of AR mixed at190* ℃

Additive Content (wt %)	Softening point/ ℃	Viscosity/Pa•S (190 ℃)				
		1 rpm	5 rpm	20 rpm	60 rpm	90 rpm
0	60.0	15.0	6.5	3.0	2.6	2.3
1%TOR	60.5	13.5	6.0	3.1	2.7	2.3
1%Wax	60.5	9.9	4.5	2.8	2.0	1.9
2%Wax	61.0	9.8	4.5.	2.7	2.1	1.9.

Table 4 showed the effect of the different kinds of additives on the softening point and viscosity of AR mixed at 190 ℃. It can be seen that the AR still remained shear thinning behavior after 45 min mixing at 190 ℃, however, whose viscosity was only one fourth of AR mixed at 175 ℃. It indicated that the original state of the crumb rubber changed and part of the rubber was desulfurized or degraded; the percent of solid part became less, which resulted in the decrease of the viscosity. The softening point decreased about 10 ℃, which indicated that, the higher temperature processing not only decrease the viscosity but also decreased the softening point of AR.

TOR and PE wax surely both decreased the viscosity of AR. PE wax had more obvious effect. The reduction of the viscosity changed a little with the increase of the wax content. Both TOR and PE wax had the same effect on the softening point, which lightly increased the softening point of AR.

Compared to the viscosity of AR mixed at 175 ℃ for the same time, it was easy to find that the AR mixed at 190 ℃ for 45minutes could meet the requirement of mixing and laying. Compared the line 3 of Table 3 and line 4 of Table 4, it can be found that AR mixed at 175 ℃ for 120 minutes in the presence of reactive agent could be equal to the AR mixed at 190 ℃ for 45 minutes. The desulphurization agent obviously shortened this process, which made it meet the acceptable viscosity requirement at 175 ℃ in less than an hour.

4. Conclusions

1. Mixing temperature had a great effect on the properties of AR, mixing at higher temperature (190 ℃) is quite different with that at 175 ℃ , which greatly reduced the viscosity and the softening point of the AR.

2. The addition of TOR could lightly reduce the viscosity and increased the softening point of AR.

3. The addition of PE wax decreased the viscosity of AR and kept the softening point in the same time.

4. Desulfurizaiton agent obviously decreased the viscosity in short time, however, the softening point decreased. It could decrease the processing temperature and reduce the blending time of AR.

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Mechanism Research of Crumb Rubber Modified Asphalt

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ABSTRACT. This study is to search the mechanism of crumb rubber modified asphalt and to supply some theoretical support for utilizing crumb rubber modified asphalt successfully and reasonably. In the present study, the rule of oil component absorbed by crumb rubber from catalyzed slurry oil, aromatic hydrocarbon extract oil and vacuum residue with temperature and time was investigated. It was found that during the action of crumb rubber and medium, absorbed oil component ratio first increased and then dropped as temperature and time increased in catalyzed slurry oil and vacuum residue. However, it increased in aromatic hydrocarbon extract oil under the same conditions. The ratio of absorbed oil component in vacuum residuum is lower than those in catalyzed slurry oil and aromatic hydrocarbon extract oil. And crumb rubber released some large molecule component and black carbon into medium. At the first time the study find the proof of component exchange between crumb and asphalt but not by presumption.

KEYWORDS:Mechanism Crumber rubber modified asphalt

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1. Introduction

With the rapid development of traffic, the amount of wasted tires is increasing more and more quickly (Yuan 2001). The importance of environmental protection is paid more attention by people today. How to utilize wasted tires and turn them into a useful material is a new subject. The wasted tires are used to modify asphalt to improve its performance is an important means to consume wasted tires. The utilization of the crumb rubber to modify asphalt in China is prevailing. This is to improve the performance of asphalt as a kind of pavement materials and to consume some tire to decrease pollution from traffic. Some experiment roads are being constructed by traffic and research department. Some researchers began to modify asphalt with crumb rubber in China in 1980s (Billiter 1996; Abdelrahman 1996; Zanzotto *et al*. 1996; Zhang *et al*. 2004). The result from experiment roads is satisfactory. And the results tell us that crumb rubber can improve the performance of asphalt as a kind of pavement materials. However, the mechanism of modifying asphalt with crumb rubber is not clear. Several researches have some done work in this field, but direct evidence was scarcely obtained. They found that crumb rubber swelled at low temperature because of absorbing oil content from asphalt. However, at higher temperature crumb rubber desulfurized and depolymerized. Some crossing network was broken and some small molecules entered into asphalt, which induced asphalt to resolve crumb rubber. These findings are from some properties of crumb rubber modified asphalt and few data are found to verify the conclusion. This paper is to find the mechanism of crumb rubber modified asphalt by studying the rule of the ratio of oil content absorbed by rubber particle from wasted tire in catalyzed slurry oil, aromatic hydrocarbon extract oil and residue.

2. Experimental

2.1. Materials

The rubber particles used in the study were from wasted tires in ShenYang which were processed in environment temperature. They have coarse surface and unshaped shape. The size of the rubber particles is about 10mm. The petroleum contents are catalyzed slurry oil, aromatic hydrocarbon extract oil and residue. The reason to select the three medium is that they have different four group and they are the compenents of petroleum which are the same as asphalt .Their basic properties are listed in Table 1.

Table 1. *Basic properties of catalyzed slurry oil, aromatic hydrocarbon extract oil and residue*

Item	catalysized slurry oil, %	aromatic hydrocarbon extract oil, %	Residue, %
saturates	46.3	12.22	20.85
aromatics	26.04	66.61	35.11
Resins	24.78	20.86	40.94
asphaltenes	2.88	0.31	3.1
Softening point，℃	-	-	41.0
penetration，0.1mm	-	--	164

From table 1, we find that comparing with catalyzed slurry oil, aromatic hydrocarbon extract oil and residue, the saturates is highest and aromatics is the lowest, and oil content is 72.34% in catalyzed slurry oil. Comparing with catalyzed slurry oil and residue, the saturates and asphaltenes are the lowest and the highest aromatics is 66%, and oil content is 78.83% in aromatic hydrocarbon extract oil, the asphaltenes is the highest and resins is the highest comparing with catalyzed slurry oil and aromatic hydrocarbon extract oil, and the lowest oil content is 40.91%t in aromatic hydrocarbon extract oil.

2.2. Experimental procedure and the main analytical item

Weigh the mount rubber particle and put it into a tube, in which there is some petroleum content, then cover the tube with aluminium foil. Put the covered tuber into a constant temperature oven for certain period time. Pour the content in the tube and filter the content and wash the rubber particles to remove the oil content on the surface of rubber particle, weigh dried rubber particle to calculate the ratio of oil content absorbed by rubber particles, a oil content left which isn't absorbed by rubber particles is to evaluate the four group.

In the paper blending exist in the procedure of rubber particle absorbing petroleum content in residue.

The four components are determined by thin layer chromatography, the chromatogram column is made of silica gel， the solvents are n-heptane, toluene, tetrahydrofuran. The method divides petroleum content into four component by their polarity and their diffusion ability in different solvent. We gain saturates by n-heptane, aromatics by toluene, resins by tetrahydrofuran, the left is Asphaltenes.

The temperatures were 130℃, 160℃, 190℃, 210℃, 230℃.The constant times were 1h, 3 h, 5 h, 7h in catalyzed slurry oil and aromatic hydrocarbon extract oil. The constant times were 1h, 2 h, 3 h, and 4h in residue. The mass of rubber particle and petroleum content is constant.

3. Results and discussion

The data from the experiment are analyzed as following.

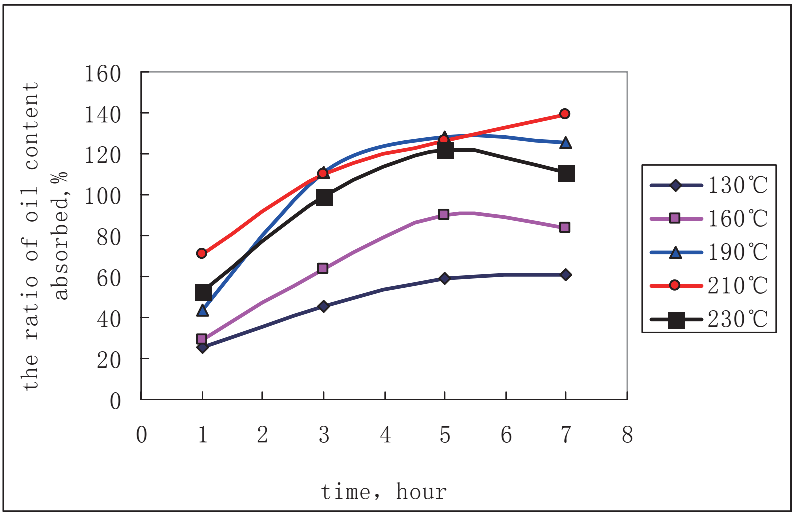

Fig.1 *Relationship between the ratio of oil content absorbed by rubber particle in catalyzed slurry oil with temperature and time*

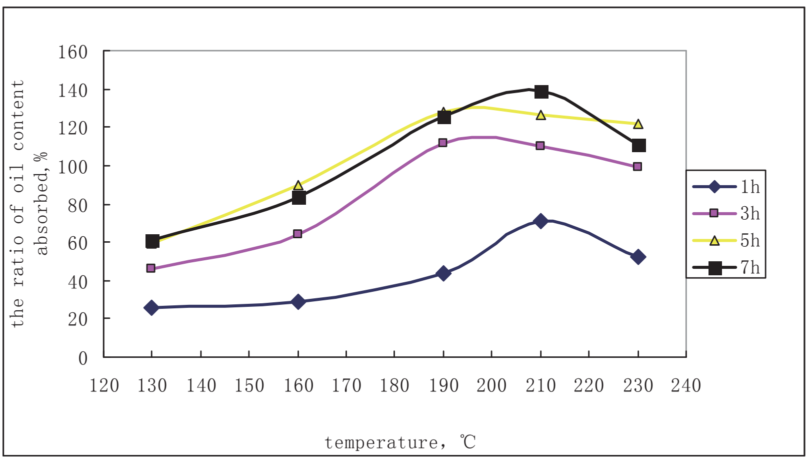

Fig. 2 *Relationship between the ratio of oil content absorbed by rubber particle in catalyzed slurry oil with temperature and time*

From figure 1, we find that the ratio of oil content absorbed by rubber particle in catalyzed slurry oil was increasing with the increase of time at the same temperature, and the time was shorter than five hours., the ratio of oil content absorbed by rubber particle in catalyzed slurry oil was almost the highest when the time is 5h and the ratio of oil content absorbed by rubber particle in catalyzed slurry oil decreased when the time was longer than 5hours. From figure 2, we find that the ratio of oil content absorbed by rubber particle in catalyzed slurry oil increased first when the temperature was lower than 210℃ and then decreased with increase of temperature when the temperature was higher than 210℃ as the time keeps the same in catalyzed slurry oil.

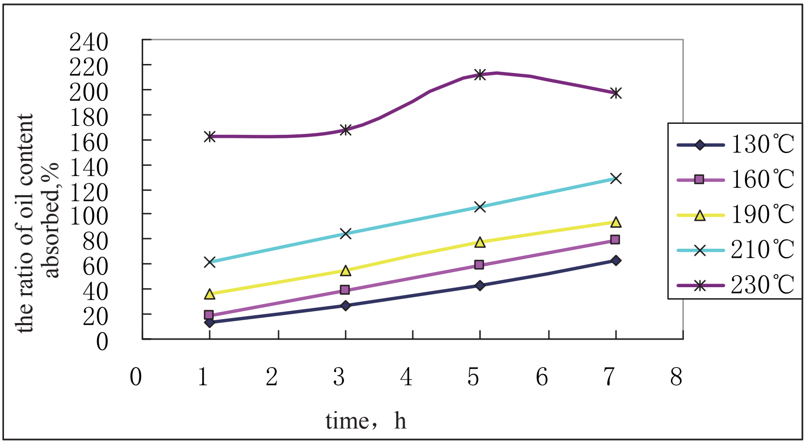

Fig.3 *Relationship between the ratio of oil content absorbed by rubber particle in aromatic hydrocarbon extract oil between temperature and time*

From figure 3 and 4, we find that the ratio of oil content absorbed by rubber particle in aromatic hydrocarbon extract oil increase with the increasing of time and temperature. The ratio of oil content absorbed by rubber particle in aromatic hydrocarbon extract oil at all the temperatures and all the time does not appear. The ratio of oil content absorbed by rubber particle in aromatic hydrocarbon extract oil is higher than that in catalyzed slurry oil when the condition is the same.

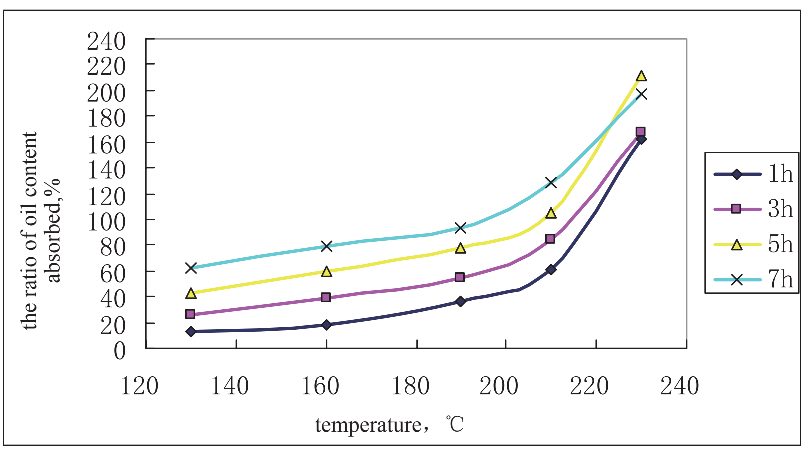

Fig.4 *Relationship between the ratio of oil content absorbed by rubber particle in aromatic hydrocarbon extract oil between temperature and time*

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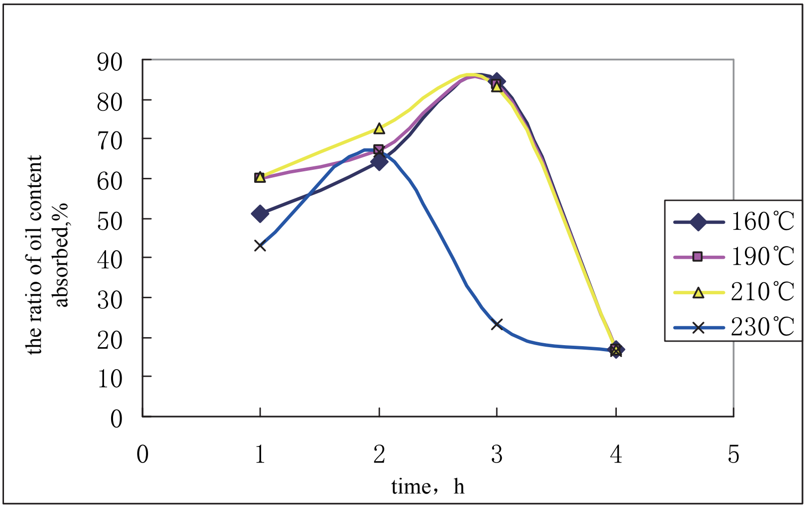

Fig.5 *Relationship between the ratio of oil content absorbed by rubber particle in residue with temperature and time*

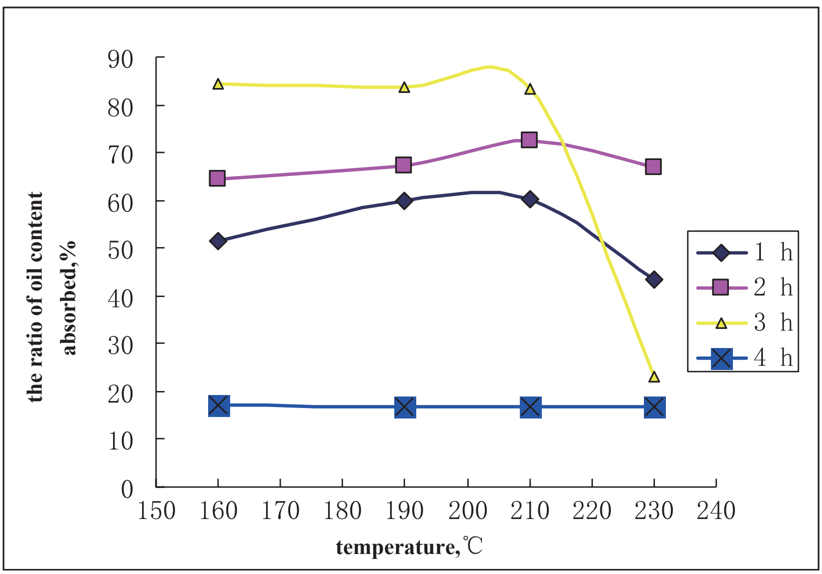

Fig.6 *Relationship between the ratio of oil content absorbed by rubber particle in residue with temperature and time*

From figure 5 and 6, we find，in residue the ratio of oil content absorbed by rubber particle in increase with the increase of time and temperature when time was lower 4h and the temperature is higher than 210℃. The ratio of oil content absorbed by rubber particle in residue is far lower than that of in aromatic hydrocarbon extract oil and in catalyzed slurry oil when the condition is the same. But comparing with ratio of oil content absorbed by rubber particle in catalyzed slurry oil, the phenomenon of decreasing of the ratio of oil content absorbed by rubber particle in residue appear in shorter time and lower temperature. At all temperatures, the ratio of oil content absorbed by rubber particle in residue when time is longer than 4h is far lower than that of in other temperature. This phenomenon does not appear in aromatic hydrocarbon extract oil and catalyzed slurry oil. Stirring accelerates the process of absorbing oil and transferring mass in residue. This makes some components in rubber particle enter residue and make the per se mass of rubber particle reduce. So the mass of rubber particle after rubber particle absorbed oil content in residue is lower than that of aromatic hydrocarbon extract oil and catalyzed slurry oil. But that the mass of rubber particle after rubber particle absorbed oil content in residue is far lower cannot be explained by stirring. From the size of rubber particle after rubber particle absorbed oil in residue in 230℃ become small, we can infer that rubber particle take place the reaction of desulfurizing and polymerizing and the reaction make the per se mass of rubber particle reduce rapidly. From the four groups of three petroleums we can infer the higher content of resins and asphaltenes have the some degree function of inducing the reaction of desulfurizing and polymerizing. This is the reason that the ratio of oil content absorbed by rubber particle in residue and catalyzed slurry oil appear to decrease and that of aromatic hydrocarbon extract oil appear to increase all the while.

4. Conclusions

- The ratio of oil content absorbed by rubber particle increase with time and temperature in aromatic hydrocarbon extract oil.

- The ratio of oil content absorbed by rubber particle increase with time and temperature in catalysized slurry oil when the temperature is lower than 210℃. The ratio of oil content absorbed by rubber particle decreased with time and temperature in catalyzed slurry oil, when the temperature was higher than 210℃.

- The time and temperature of the ratio of oil content absorbed by rubber particle decreasing with time and temperature is far shorter and far lower than that of in catalyzed slurry oil.

- From the study, we infer that the higher content of resins and asphaltenes have the some degree function of inducing the reaction of desulfurizing and polymerizing.

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Chapter 4

Case Studies

The application of Semi-flexible Pavement with Asphalt -Rubber on Heavy Traffic Road

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ABSTRACT: This paper focuses on the performance of semi-flexible pavement used in heavy-traffic road according to the technical characteristics of complex pavement. The high temperature stability, low-temperature cracking resistance and water stability of semi-flexible pavement with asphalt-rubber were tested in the laboratory and compared to asphalt concrete AC-16 with asphalt-rubber (AR-AC-16). In addition, an experimental section was built. The dynamic stability of semi-flexible mixture AC-20 with asphalt-rubber (SFAC-20) was much larger than that of the asphalt concrete AC-16 with asphalt-rubber (RAC-16). The residual stability and freeze-thaw splitting strength ratio of SFAC-20 exceeded 21.2% and 11.7%, respectively. Moreover, SFAC-20 mixture had larger failure tensile strain and stiffness modulus under -10℃ temperature condition. The results showed that the semi-flexible pavement with asphalt-rubber had better properties and was especially suitable for the surface course of heavy-traffic road. Both laboratory tests and engineering practice showed that the semi-flexible pavement with asphalt-rubber had excellent performance and application prospect.

KEY WORDS: Highway engineering, semi-flexible pavement, crumb rubber modified asphalt, brittle point, strain energy density, heavy traffic

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1. Introduction

With the development of industrial production of ground waste tires and its pavement structure research, crumb rubber has become more and more popular in highway engineering as a new road material to improve significantly the performance of asphalt concrete. Modified asphalt preparation has been one of the efficient approaches for using crumb rubber. Furthermore, modified agent of crumb rubber provides a good way to treat waste tires [4][5][6].

Semi-flexible pavement is usually defined a compound one prepared by porous mother asphalt mixture filled with special cement slurry. Therefore, semi-flexible pavement combines the properties of asphalt pavement and cement concrete pavement. It has been widely used to improve the pavement lapse resistance in Japan and other countries. In this paper, according to the research achievements of pouring semi-flexible pavement, the performance and the application of crumb rubber modified semi-flexible asphalt pavement was studied considering its energy saving and environment protection [2][7]. It helps to fill a domestic gap in the semi-flexible field, improve the performance of semi-flexible and its application range.

2. Study on the mix design of semi-flexible pavement mixture with asphalt-rubber

2.1. Properties of raw materials

2.1.1. Crumb rubber

Crumb rubber used was of natural gradation which was ground at room temperature, and its gradation is listed in Table 1.

Table 1. *Gradation of crumb rubber*

Size of sieve (mm)	2.36	1.18	0.6	0.3	0.15	0.075
Passing rate (%)	100.0	97.3	61.4	20.6	6.5	2.7

2.1.2. Asphalt-rubber

Asphalt-rubber was prepared by virgin asphalt of AH-70, and its properties are listed in Table 2.

Table 2. *Properties of asphalt-rubber*

Test items	Design requirements	Asphalt-rubber
Content	-	19%
Penetration (25ºC,0.1mm)	35~55	40
Soft point (ºC)	>60	69.4
Elastic recovery at 25ºC (ºC)	>70	85
Rotary viscosity(Pa·s，180ºC，20r/min)	1.5~5.0	3.9

2.1.3. Cement, fly ash and aggregate

Cement used in the study was Ordinary Portland Cement P.O42.5R from Chongqing. The quality of fly ash was classified as the secondary class. The coarse aggregate used was crushed by a typical limestone. A natural superfine sand from Yangtze River and limestone chip were used as the fine aggregates. The limestone mineral powder was chosen as filler. Properties of all materials met the requirements of standard [1].

2.2. Mix design of cement mortar

The cement mortar investigated in this paper included Ordinary Portland Cement, fly ash, mineral powder, river sand, water, and additive (also toner in type of color mortar). According to the related specification in Japan [2], cement mortar mix proportion was determined and is listed in Table 3.

Table 3. *Mix proportion of cement mortar*

Type	Water/ cement ratio	River sand (%)	Mineral powder (%)	Fly- ash (%)	toner (%)	fluidity (s)	Strength at seventh day (MPa)	
							Flexural strength	Compressive strength
Ordinary type	0.65	14	10	6	-	11.4	4.4	17.2
Color type	0.68	14	10	6	4	11.6	4.9	18.4

2.3. Mix design of mother mixture with asphalt-rubber

The semi-flexible mixture with asphalt-rubber had a skeleton-dense structure. However, mother asphalt mixture with asphalt-rubber had a skeleton-gap structure. Therefore, it could supply sufficient voids for the complete filling of cement mortar based on skeleton formation. This kind of pavement structure possessed good cementing power and its voids were filled with high fluidity cement mortar. The volume characters of mother asphalt mixture played an important role in the mechanical properties of semi-flexible pavement. Provided the volume characters were mainly accounted for in the mix design process, mother asphalt mixtures could perform very well. Designed void ratio of semi-flexible mother mixture with asphalt-rubber was 20%-25%, and its asphalt content was 3.0%-5.0%. All indexes in details are listed in Table 4 [2].

Table 4. *Design requirement s of mother asphalt mixture*

Item	Density (g/cm^3)	Void ratio (%)	Compaction number (times)	Asphalt content (%)	Stability (kN)	Flow value (0.1mm)
Desired value	≥1.9	20~25	Each 50 at double sides	3.0~5.0	≥3.0	20~50

2.3.1. Gradation of mother asphalt mixture

Because the asphalt film of asphalt-rubber was thicker and the void ratio of mixture with asphalt-rubber was less than that of the virgin asphalt mixture, mineral aggregates gradation was inclined to the lower limit in the mix design process so as to ensuring the complete incorporation of cement mortar. Gradations of mineral aggregates are listed in Table 5.

Table 5. *Composite gradation of mineral aggregates*

Size of aggregates	Mix proportion (%)	Size of sieve（mm）								
		26.5	19	13.2	4.75	2.36	0.6	0.3	0.15	0.075
10~20mm	58.0	100	89.5	12.7	5.8	1.4	0.0	0.0	0.0	0.0
5~10mm	28.0	100	100	82.7	10.7	4.5	4.1	1.8	1.5	2.1
chip	10.0	100	100	100	78.7	47.0	16.9	12.4	4.5	2.31
Mineral powder	4.0	100	100	100	100	100	99.8	92.4	81.2	74.4
Composite gradation		100	93.9	44.2	16.9	9.9	6.5	5.2	4.1	3.8
Higher limit of gradation		100	100	60.0	24.0	18.0	14.0	10.0	8.0	6.0
Lower limit of gradation		100	93	40.0	12.0	8.0	6.0	5.0	4.0	2.0

2.3.2. Optimum asphalt- aggregate ratio

Five different asphalt contents were used based on the composite gradation and test results are listed in Table 6. The relationship curve between asphalt-aggregate ratio and scattering loss is shown in Figure 2. As shown in Figure 2, the minimum asphalt-aggregate ratio at inflection point was 3.5%. From the relationship curve between asphalt-aggregate ratio and leak loss (Figure 3), the maximum one at inflection point was 3.7%. Based on the range of 3.5%-3.7%, the optimum asphalt-aggregate ratio of 3.6% was determined according to the designed the void ratio.

Table 6. *Indexes of mixtures at different asphalt contents*

Asphalt-mineral aggregate ratio (%)	Scattering loss (%)	Leak loss (%)	Void ratio (%)	Stability (kN)	Flow value (0.1mm)
3.0	41.7	0.15	23.1	7.35	41.3
3.3	30.3	0.27	22.8	7.23	45.6
3.6	23.8	0.58	22.1	6.80	47.2
3.9	20.4	1.04	21.8	5.79	48.4
4.2	19.1	1.53	21.7	5.38	44.7

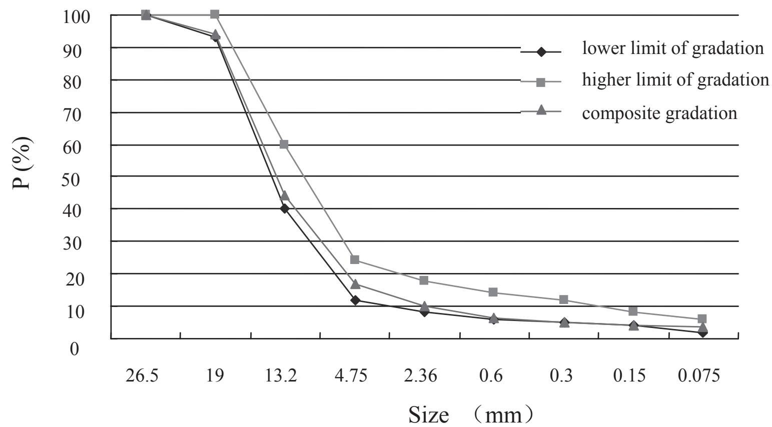

Figure 1. *Composite gradation curve of mineral aggregates*

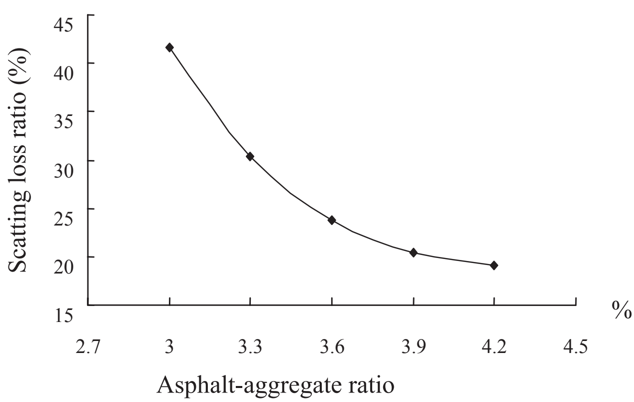

Figure 2. *Relationship curve between asphalt-aggregate ratio and scatting loss ratio*

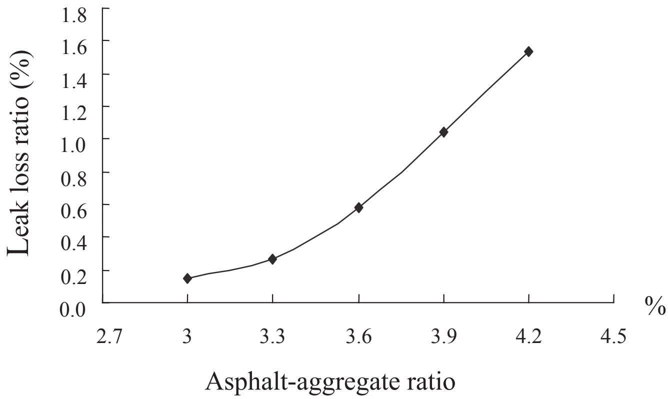

Figure 3. *Relationship curve between asphalt-aggregate and leak loss*

3. Study on the properties of semi-flexible pavement mixture with asphalt-rubber

3.1. High temperature stability

The rutting test results of semi-flexible pavement mixture with asphalt-rubber were listed in Table 7. As can be seen from Table 7, the dynamic stability of AR-AC-16 was about 2000 times/mm whereas that of semi-flexible mixture with asphalt-rubber was up to 15000 times/mm. It was obvious that designed semi-flexible pavement mixture in the study had excellent high temperature performance. It may due to the thinner asphalt film of semi-flexible and the filled void by cement mortar. Therefore, the decreased degree of high temperature performance of semi-flexible was smaller because of the asphalt softening at 60ºC temperature.

Table 7. *Results of the rutting test*

Type	1min	45min	60min	Dynamic stability(times/mm)
Semi-flexible mixture	0.262	0.802	0.842	15750
AR-AC-16	0.635	1.972	2.243	2325

3.2. Water stability

As shown in Figure 4, compared to the AR-AC-16 asphalt mixture, the residual stability and freeze-thaw splitting strength of semi-flexible mixtures with asphalt-rubber exceeded 21.2% and 11.7%, respectively, which showed that water stability of semi-flexible mixture with asphalt-rubber was better than the AR-AR-16 asphalt mixture.

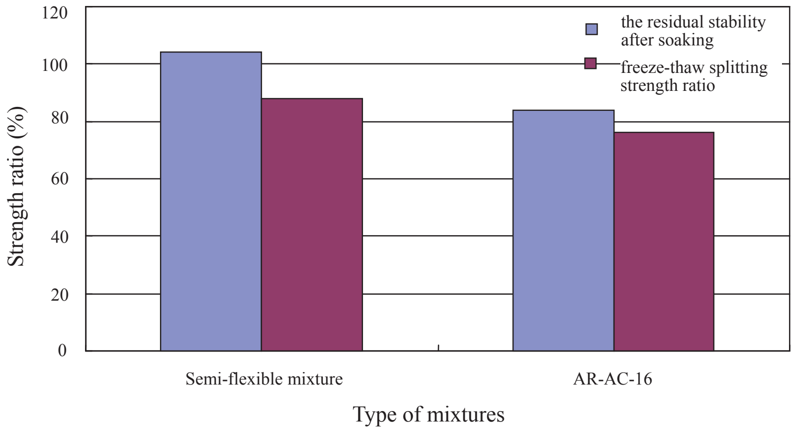

Figure 4. *Comparison of water stability test results between different mixtures*

According to the test results, the stability and residual stability of semi-flexible mixtures with asphalt-rubber were 21.87kN and 21.72kN, respectively. Both of them showed a high strength level. It is thought that the cement used in the semi-flexible mixture obviously improved the rigidity of asphalt mixture and prevented the water into the mixture water. Also, the stability of asphalt mixtures after 48h soaking was better than that of mixtures without soaking. It's because that the strength of cement slurry was not stable but increasing gradually with time. Once ambient temperature and humidity increased, hydration of cement was greatly enhanced and correspondingly the strength increased greatly, which resulted in the better residual stability of mixtures after 48h soaking than that of mixtures without soaking. It was explained from another point of view that this asphalt mixture has good water stability.

3.3. Low temperature crack resistance

Indirect tensile test was conducted to investigate the low temperature crack resistance of semi-flexible mixture with asphalt-rubber. Test results are listed in Table 8. Compared to AR-AC-16 asphalt mixture with asphalt-rubber, semi-flexible pavement mixture with asphalt-rubber had lower splitting tensile strength, larger failure tensile strain, and lower stiffness modulus, which showed that its low temperature crack resistance was better than the AR-AC-16 asphalt. It may be the result of the thinner asphalt film and less temperature influence of semi-flexible.

Table 8. *Results of low temperature crack test*

Test condition	Type of mixtures	Splitting tensile strength (MPa)	Failure tensile strain (με)	Failure stiffness modulus (MPa)
Temperature is -10ºC, and the loading speed is 1mm/min	Semi-flexible mixture	2.18	5136	873.52
	ARAC-16	3.83	3460	1318.74

4. A survey of the practice project construction

4.1. Project Ⅰ

A cement concrete pavement reconstruction project from Xiao nan-hai cement plant in Chongqing was conducted in the study to investigate the construction process. This project was a plant pavement, which had heavy traffic (rear axle load 500kN), large longitudinal slope (the maximum up to 14.3%), and poor horizontal alignment (the minimum of horizontal radius up to 10m). The pavement was damaged quickly during its service period with the soaking of rainwater. Though this pavement was repaired a number of times, the whole service quality of pavement decreased gradually, and some partial sections even became unsafe. To restore its service quality, a design scheme was proposed. First, a 1cm-thick stress absorbing maintenance layer with asphalt-rubber (AR-SAMI) was constructed after the treatment of cement pavement. Then, a 4cm-thick semi-flexible pavement layer with asphalt-rubber was paved on the top of it. The total area of this project was 7200m^2 with a length of 800m and a width of 9m. The used raw materials, the mix design and the cement mortar in the project were similar to the laboratory.

After two service life periods with the loading of rear axle load 500kN, semi-flexible pavement with asphalt-rubber showed an excellent lapse and rutting resistance and kept complete compared with the ordinary concrete pavement with asphalt-rubber.

4.2. Project Ⅱ

The second project was a reconstruction one of Bei-jin road in Bei-bei district in Chongqing. The length of this pavement was 7000m, and the width was 6.5m. The design scheme was “treated aged cement pavement + 1cm AR-SAMI + 5cm ARSF-20”. The surface of pavement was colored or paved with AC-13 of 3.5cm in some partial road sections to meet the heavy traffic and landscape requirement.

Basalt crushed stone was used as aggregate in the project. Other materials were the same as that of project one. The mix proportion of mother mineral aggregates was 85% 10-20mm basalt crushed aggregate, 6% 5-10mm basalt crushed aggregate, 5% limestone artificial sand and 4% limestone mineral powder. The optimum asphalt-aggregate ratio 3.3% was obtained by the Marshall test. The performance of semi-flexible mixture with asphalt-rubber was tested and the results are listed in Table 9. The project was constructed very well and praised completely by the owners.

Table 9. *Test results of semi-flexible mixture with asphalt-rubber*

Index	**Before pouring the cement mortar**		**After pouring the cement mortar**	
	Desired value	Test result	Desired value	Test result
Marshall stability (kN)	>3.0	4.2	>9.0	13.1
Flow value (0.1mm)	20~40	33.9	20~40	32.0
Void ratio (%)	20~28	21.28	<3.5	2.3
Bulk volume relative denstiy	>1.9	2.056	--	2.381
Dynamics stability (times/mm)				17800

4.3. Construction technology

4.3.1. Construction of AR-SAMI

Stress absorbing maintenance layer (SMI), defined as a layer located between the semi-rigid base or cement pavement and asphalt pavement, was prepared by modified asphalt, which has good deformability. It could result in stress relief of crack and avoid the formation of asphalt pavement reflection crack. The construction process included: (1) obtained a complete clean surface of base; (2) determined rubber powder percentage and prepare asphalt-rubber; (3) sprayed asphalt-rubber with a content of 2.0-2.6 kg/m^2 and aggregate with a content of 16±2kg/m^2; (4) compacted pavement using over 25t rubber-wheel roller on a clean of loose aggregate.

4.3.2. Paving of mother mixture with asphalt-rubber

Semi-flexible pavement with asphalt-rubber was a pavement structure prepared by including cement mortar in mother asphalt mixture. Therefore, it was necessary to control accurately porosities of mother mixtures in the paving of mother asphalt mixture. The paving parameters are listed in Table 10.

4.3.3. Preparation of cement mortar

Cement mortar was usually prepared by mixer or manual on site. All raw materials were added in the following order: mineral powder, cement, fly ash, fine sand, early strength agent, and water. Other materials should be completely mixed to be uniform before adding water, then continued to be mixed with water for about two or three minutes to be uniform. The manual mix was used because of the smaller amounts in Project I. The mixer was used in Project II to improve the efficiency and the uniformity of strength.

Table 10. *Construction parameters of mother mixture with asphalt-rubber*

Preparation of asphalt-rubber	Temperature	Virgin asphalt	170℃
		Asphalt-rubber	180~190℃
	Reaction time	Mixing 300r/min	45~60min
mixing	temperature	Asphalt-rubber	170~190℃
		Mixture	180℃
	time	Total mixing time	40s
transportation	temperature	Discharging temperature	190~200℃
		Mixture temperature on site	175~200℃
paving	Paving speed	1~3m/min	
	Loose paving coefficient	1.16~1.18	
	Paving temperature	No less than 160℃, and discarded once lower than 140℃	
compaction	Combination of compaction	double compaction three times by heavy double-drum roller in the way of both high frequency and low frequency, final compaction one time by double-drum roller in the way of static pressure	

4.3.4. Filling of cement mortar

Firstly, the void ratio of mother mixture was determined by conducting core sample to calculate required cement mortar amounts and design some indexes. Then, the fluidity of cement mortar was adjusted to ensure a complete filling. Rubber harrow was first used to pave the cement mortar repeatedly to ensure the natural filling of cement mortar. At the same time, small vibratory roller or flat vibrator was used to enhance the filling effect. When there was a longitudinal slope in the pavement, cement mortar need to be paved in the order of from bottom to top so as to avoid a poor filling due to its quick flow.

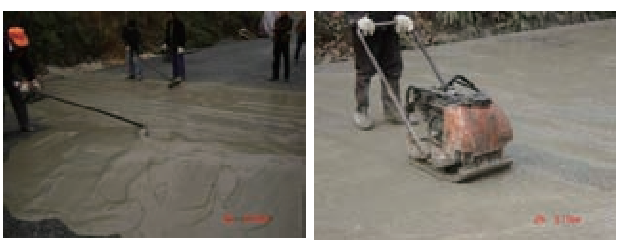

Figure 5. *Pouring construction of cement mortar using flat vibrator*

4.3.5. Appearance treatment of pavement

It was essential that the residual cement mortar on the road surface was cleaned using harrow to bare the irregular surface of mother mixture. Retarder was sprayed on the road surface to obtain a good pavement structure which could be affected due to incorporation of cement mortar in mother mixture (Figure 6). Then, when the final setting time of cement mortar within the mother mixture ended, cement mortar on the road surface was washed before its final setting time to ensure the strength of internal cement mortar, a good pavement structure and the uniform pavement color (Figure 7).

Figure 6. *Spraying of retarder*

Figure 7. *Wash of residual cement mortar*

4.3.6. Curing

Curing time changed with the properties of cement mortar. The traffic was usually open after two or three days curing, even a day provided the incorporation of early strength cement or early strength agent in the cement mortar. The appearance of pavement after curing is shown in Figure 8.

Figure 8. *Appearance of pavement after curing*

5. Conclusions

The results showed that the semi-flexible pavement with asphalt-rubber SFAC-20 had excellent high temperature performance. In addition, its residual stability and freeze-thaw splitting strength ratio exceeded 21.2% and 11.7% respectively compared to RAC-16 mixture. Moreover, its low temperature (-10℃)crack resistance (the failure tensile strain and stiffness modulus) was better than that of RAC-16 mixture. It was indicated that semi-flexible pavement with asphalt-rubber could be used as an economical, feasible in technology and well performed pavement material. It is feasible to construct the semi-flexible pavement using the technology proposed by this study. Also, it is proved that the semi-flexible pavement is very suitable for the pavement structure on heavy traffic road.

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Asphalt Rubber - a New Concept for Asphalt Pavements in Sweden.

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ABSTRACT: 'Asphalt Rubber' is a bitumen, mixed with 15 to 20 % of rubber granules from recycled tyres to improve the functional properties of asphalt pavements. The 'Asphalt Rubber '-concept for asphalt pavements has been successfully implemented in a number of US states. The Swedish Road Administration (SRA) is currently involved in investigating the potential implementation of this concept in asphalt pavements through a three-year development project (2007-09). This paper presents promising test results from trials with asphalt rubber pavement materials from the ongoing projects and evaluation studies.

KEYWORDS: 'Asphalt Rubber', performance, environmental effects, Sweden, Swedish Road Administration

1. Introduction

During the autumn 2006 the Swedish Road Administration (SRA) decided to start a development project for the period of 2007 - 2009 to investigate the potential implementation in Sweden of asphalt pavements, produced according to the 'Asphalt Rubber'-concept. This concept, based on the 'wet technique', was developed in the United States (US) and has been established in a number of states. 'Asphalt Rubber' has been produced in a large scale in US since the end of the 1980's. The international interest for 'Asphalt Rubber'-pavements has grown in recent years. At present the 'Asphalt Rubber'-concept is being established in several countries.

'Asphalt Rubber' can be characterized as a rubber-modified bitumen, where rubber granules have been mixed with a standard bitumen ('wet technique'). The rubber granules originate from used tyres. The 'Asphalt Rubber' is manufactured in a specially developed mixing equipment. The equipment is mobile and placed in connection to the asphalt plant, producing the asphalt mix. The mixing of the components is carefully controlled and followed by a procees of maturity. The content of rubber granules is 15 to 20 % of the mass of the binder or 1,5 to 2,0 % of the mass of the asphalt mixture.

The expected long-term effects and potential benefits of the project are reduced annual and life cycle costs, environmental advantages with respect to noise and emission of particles, and also improved traffic safety in terms of increased friction.

Within the frame-work of the present project the following issues are evaluated:

- Technical conditions and possibilities for production
- Environmental effects and restrictions
- Technical performance of asphalt mix/pavement.

The evaluation is based on field trials/demonstration projects, where the performance (functional properties) and possible influence on work environment as well as environment of surroundings will be monitored.

Special focus is on the environmental consequences (both work environment and environment of surroundings). The Swedish Chemical Agency has recommended a restrictive use of old rubber tyres, in reference to their high contents of polycyclic aromatic hydrocarbonates (PAH).

Asphalt pavements with rubber granules have been previously produced in Sweden, but not according to the above 'wet technique'. Instead a 'dry technique' was used, where the rubber granules were added to the aggregate before the mixing with bitumen at the asphalt plant. The main problem with this type of rubber asphalt was the varying quality. The 'wet technique' according to the 'Asphalt Rubber'-concept makes it possible to produce rubber asphalt pavements of high and uniform quality.

This paper presents a progress report as of February 2009 regarding the Swedish experience in 2007-2008 from asphalt pavements, produced according to the ‘Asphalt Rubber’ -concept..

2. Motives for the project

The Swedish Road Administration (SRA) considers that the ‘Asphalt Rubber’-development project is a project of great economic importance and expects that the introduction of ‘Asphalt Rubber’-pavements on the Swedish road net will result in

1. Reduced annual and lifecycle costs;

2. Environmental advantages, concerning reduction of noise and particle emissions;

3. Increased traffic safety;

Reduced annual and life cycle costs

Based on international experience the service life of ‘Asphalt Rubber’-pavements can be doubled in comparison with corresponding standard pavements. Due to the uncertain influence of studded tyres in Sweden on the service life of the ‘Asphalt Rubber’-pavements, it is assumed that only half of the expected increase in service life can be utilized. The estimated increase in costs for construction of the ‘Asphalt Rubber’-pavement is about 25 %. This means that the annual cost for ‘Asphalt Rubber’-pavements will be about 80% of the annual cost for the standard pavements. The annual maintenance cost for SRA’s asphalt pavements in Sweden amounts to about 250 million Euro. Given that the ‘Asphalt Rubber’-pavements are a marketable alternative, representing 25 % of that volume, the annual maintenance cost could be reduced around 10 million Euro.

Environmental advantages

The tyre noise and the emission of particles, due to abrasion from studded tyres, are expected to decrease and improve the environment.

Improved traffic safety

The traffic safety is expected to be improved on rain wet road surfaces by higher friction between road surface and tyre and reduced formation of ‘water curtains’.

3. Objectives of the project

The project shall demonstrate the prospects for development and implementation of the ‘Asphalt Rubber’ concept for pavements on the Swedish road net. The main objectives of the project are

1. Verification of the possible production of high quality ‘Asphalt Rubber’-pavements in Sweden;

2. Verification of increased service life and reduced annual and life cycle costs;

3. Verification of environmental advantages;

4. Verification of management of possible health and environmental risks;

5. Verification of advantages considering traffic safety.

4. Preparatory actions and studies

The supply of technical expertise has been ascertained by engaging leading equipment suppliers for manufacturing of rubber-modified bitumen. Contracts, covering both machine and personal resources, have been established. In SRA’s own organisation the necessary technical competence will be utilized during the accomplishment of the project.

A comprehensive preliminary study, including technical issues such as production engineering and product quality, has been carried out during 2006.

Environmental aspects have also been studied by literature review and laboratory testing and the preliminary results indicate that the environmental risks should be very limited. Further studies of effects on health and environments form an integral part of the project and shall be carried out in conjunction with full-scale trials.

5. Scope and time schedule of the project

The project activities during 2007-2010 are as follows.

2007 - Construction of experimental ‘Asphalt Rubber’-pavements on roads in the southern and central parts of Sweden. Mixing equipment was supplied by SRA. Evaluation of the hitherto achieved results. Decision was taken to continue the project.

2008 - Extended construction of experimental ‘Asphalt Rubber’-pavements, involving different regional departments of SRA. Mixing equipment was supplied by SRA. Evaluation of the hitherto achieved results. Decision was taken to continue the project.

2009 - Continued construction of experimental ‘Asphalt Rubber’-pavements, involving different regional departments of SRA. Mixing equipment is supplied by SRA. Evaluation of the hitherto achieved results. Preparation of the final report. Draft technical specification for ‘Asphalt Rubber’-pavements will be presented.

2010 - The procurement of the ‘Asphalt Rubber’-pavements will start. Project information will be disseminated to suppliers and contractors to instil a genuine interest in the ‘Asphalt Rubber’-concept and to create a potential market for ‘Asphalt Rubber’-pavements.

6. Manufacture and evaluation of the ‘Asphalt Rubber’-binder

The rubber granules and the manufactured ‘Asphalt Rubber’ are specified according to the Arizona Department of Transportation “Standard specification for Road and Bridge Construction, 2000”.

The rubber modified bitumen is manufactured in a specially designed mixing equipment. It is mobile and placed in connection to the asphalt plant, producing the asphalt mix. See Figure 1.

The rubber granules (see Figure 2) are produced by fragmentation and sieving of recycled tyres, including separation of steel and fibre reinforcements. The rubber granules are mixed with normal standard bitumen in a closed process. When the Asphalt Rubber mix has the designed viscosity (normally 2, 5 – 3 Pas) it is ready for use as a normal bitumen in the asphalt production. The content of rubber granules is 15 to 20 % of the mass of the binder or 1,5 to 2,0 % of the mass of the performed asphalt pavement.

The mixing equipment, used by SRA, is rented from the equipment supplier Phoenix Industries LLC (PI), Arizona. Technical experts from PI have also been responsible for training in production technique and technical support.

Figure 1. *Mixing equipment connected to the asphalt plant*

Figure 2. *Rubber granules, used in SRA´s trials*

Before the road trials of 2007 and 2008, the 'Asphalt Rubber'-binder (AR) as well as the trial mixes, produced with this binder, has been thoroughly evaluated in the laboratory to ensure the right quality of the manufactured products. The rubber content of the rubber/bitumen-mix has been 16-19 %. The target viscosity at 175 º C after a reaction (mature) time of 45-60 min has been 2 to 3 Pas.

On the construction site the AR-viscosity is regularly controlled by a manual field viscometer. If the viscosity is deviating from the target value, the rubber content is adjusted to get the correct viscosity. The AR production temperature has been in the range of 170 to 175 º C.

Samples of the rubber granules and the manufactured AR are taken continuously and analysed at a certified laboratory. The basic bitumen consists of pen grade 70/100. The rubber granules have been purchased mainly from two suppliers, Genan in 2007 and Eximlink in 2008. AR-gradings, supplied by Genan, are shown in Figure 3.

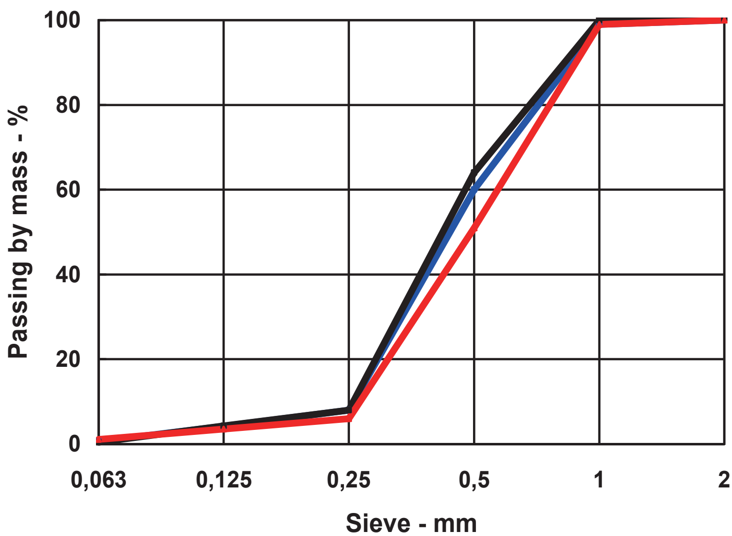

Figure 3. *Three typical gradings of rubber granules from the production in 2007.*

7. Specifications and production of the 'Asphalt Rubber'-mix

The specifications of the 'Asphalt Rubber'-mixes for the road trials are based on the Arizona Department of Transportation "Standard specification for Road and Bridge Construction, 2000". Certain modifications of the specifications for gradings and void contents have been made to fit the Swedish practice for surface course pavements.

During 2007 and 2008 nearly 30 000 ton of 'Asphalt Rubber'-mixes have been produced. The major part of the produced mixes was Stone Mastic Asphalt (SMA) - mixes, designated SMA 11 and 16. However, the filler content and the content of fine aggregate 0/2 mm have been decreased to provide room for the rubber granules (see Figure 4). The target added content of 'Asphalt Rubber' has been in the range of 8,3 to 9,0 % by mass. For the mix design the target void content, provided by Marshall impact compaction at a compaction temperature of about 170 ºC, has been 2,0 to 2,5 %. The temperature at the production of the 'Asphalt Rubber'-mix has been 165 to 170 ºC.

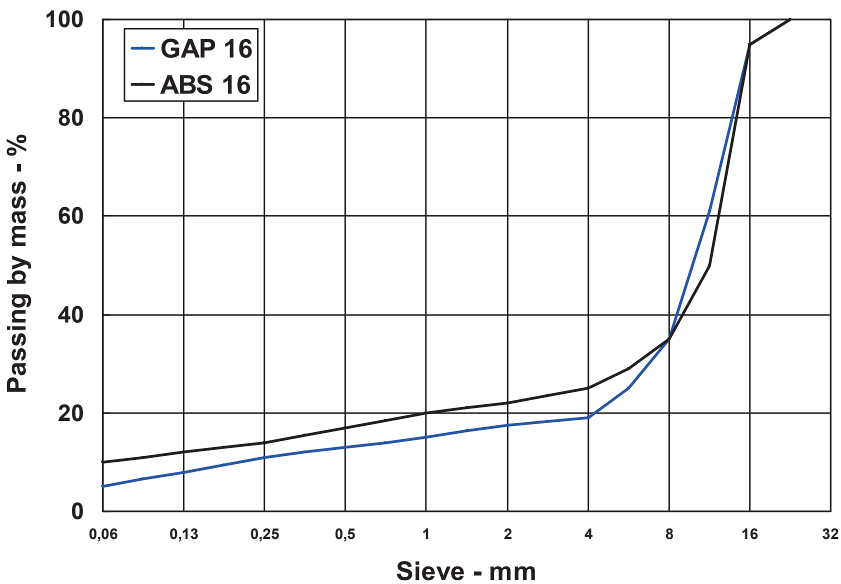

Figure 4. *Typical aggregate gradings for the 'Asphalt Rubber'-mix, GAP 16, and the reference mix, SMA 16*

Two interesting minor trials with an open-graded (void content > 15 %), low noise 'Asphalt Rubber'-pavement, based on the concept used in Arizona, have also been carried out. However, it was observed that this concept needs some adjustment and development, mainly because of the studded tyre use during the winter in Sweden.

8. Evaluation of performance and functional properties of the 'Asphalt Rubber'-mix/ pavements

Asphalt mixes and constructed pavements, involved in the project, are studied by a comprehensive monitoring programme, including field observations and measurements and extensive laboratory studies of primarily the achieved functional properties.

ASU (Arizona State University) in Phoenix and VTI (Swedish Transport and Road Institute) in Linkoping have been engaged in cooperative work on laboratory studies. Also different contractor laboratories have been involved in the factory production control and quality control and evaluation of the constructed 'Asphalt Rubber'-pavements.

ASU has studied two asphalt mixes, laid on road E6 in the vicinity of Malmoe 2007 and consisting of a 'Asphalt Rubber'-mix, GAP 16 (a gap-graded asphalt mix with a nominal upper aggregate size of 16 mm) and a reference asphalt mix, SMA 16 (stone-mastic asphalt mix with a nominal upper aggregate size of 16 mm and a penetration bitumen with grade 70/100). The results of this study are reported separately in these AR2009 conference proceedings.

In this paper some results from the VTI study, based on the test methods used in Sweden are presented. At VTI the main part of the studies has been carried out on cores, extracted from the pavement on the outer ring-road in Malmoe, constructed in 2007. The principal purpose of the 'Asphalt Rubber'-mix, laid on this test section, is to counteract the crack propagation from the underlying courses. The existing pavement, constructed during 2000-2001, is a conventional asphalt pavement of 10 cm thickness (4 cm surface course and 6 cm binder course), laid on a cement bound granular base. Extensive reflection cracks from the underlying cement bound base course appeared shortly after the traffic opening in 2001.

9. Testing conducted by VTI in Sweden

Functional properties such as flexibility (stiffness at different temperatures) resistance to fatigue, stability, resistance to abrasion from studded tyres and durability are essential for the assessment of the quality of an asphalt pavement. At the VTI study most functional properties have been determined by test methods, which are in line with current European test standards. Special test procedures, developed at VTI, have been used for the winter conditioning and crack propagation tests. See Table 1.

Table 1. *Test methods used at the VTI study*

Property	Type of test	Method used by VTI	Corresponding European Standard
Stiffness	Indirect tensile test	FAS Metod 454[a]	EN 12697-26
Resistance to fatigue	Indirect tensile test	VTI own method	EN 12697-24
Stability (Resistant to permanent deformations)	Uniaxial cyclic compression test with confinement (Dynamic creep)	FAS Metod 468[a]	EN 12697-25
Resistance to abrasion from studded tyres	'Prall'-test	FAS Metod 471[a]	EN 12697-16
Water sensitivity	Indirect tensile test	FAS Metod 446[a]	EN 12697-12
Durability – sensitivity to winter conditioning	Freeze-thaw cycles with saltwater; reduction of stiffness modulus	VTI's own method	Stiffness modulus according to EN 12697-26
Crack propagation sensitivity	Wheel-tracking test	VTI's own method	Wheel-tracking machine according to EN 12697-22

[a]Test method prepared by FAS, the former Swedish Asphalt Pavement Association.

9. Results of the VTI study

The results are based on cores, extracted from an 'Asphalt Rubber'-pavement, laid on the outer ring-road E 6 between the Sallerup and Fredriksberg interchanges. Cores with 100 mm and 150 mm diameters, sampled September 2007. have been dispatched to VTI. Some results are presented below. The complete results will be presented in a future VTI-publication.

9.1. Stability according to the dynamic creep test

The results of the stability/dynamic creep test are shown in Figure 5. The mean accumulated strain after 3600 load cycles amounts to 14 500 microstrains and fulfils the most strict requirement (<15000 microstrains) for surface courses according to SRA´s technical specifications, 2008.

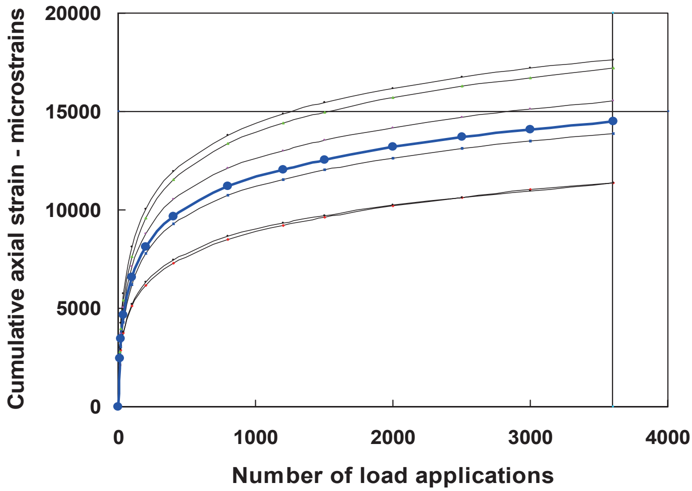

Figure 5. *Accumulated strain as a function of the number of load cycles at 40 ºC (single test specimens: black curves; blue average curve).*

9.2. Resistance to fatigue

The loading of the fatigue test was applied in the same way as the loading of the stiffness test and continued, until failure occurred in the test specimen. The 'Asphalt Rubber'-test specimens sustained about 1 million load applications at 100 microstrains and satisfied the most strict fatigue requirement of SRA´s technical specifications, 2008.

9.3. Crack propagation sensitivity

VTI has developed a special test procedure for determination of the crack propagation sensitivity of the rubber asphalt, based on VTI´s wheel tracking machine (WTT), specified under clause 6.2 'Extra large devices' in EN 12697-22. Test slabs are placed in the wheel-tracking device on a divided soft rubber base in order to generate cracks through the pavement slab. Duplicate strain gauges are mounted on the lower and upper surface of the slab, enabling recording of the crack propagation course from the bottom to the top of the slab. The test was conducted on 3 slabs, prepared using the 'Asphalt Rubber'-mix, and 3 slabs, prepared using the reference asphalt mix, SMA 16 (bitumen 70/100).

Test conditions

Slab size:	50 x 70 x 4 cm (slab mass ≈ 30 kg).
Temperature:	+5 °C.
Wheel load:	Adjusted to the desired strain level
Number of wheel passes:	Adjusted to the crack propagation

The number of wheel passes to initiate cracking at the bottom and at the top of the slab are recorded at a given level of initial strain (= recorded strain after 100 wheel passes). The initial strain reflects the load applied to the test specimen. The difference between the two numbers of wheel passes is a measure of the crack propagation rate of the test specimen. The results for the six test slabs are shown in Figure 6. The rubber asphalt slabs are less sensitive to crack propagation at the same initial strain level than the reference slabs.

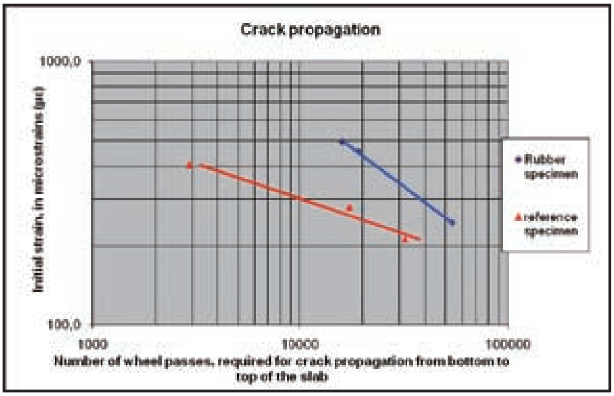

Figure 6. *Effect of initial strain on the required number of wheel passes to cause complete crack propagation of the rubber asphalt and reference test specimens.*

9.4. Sensitivity to winter conditioning

The winter conditioning method has been developed at VTI by Peet Höbeda. The conditioning phases are intended to simulate those stresses that roads are exposed to with alternating temperatures around zero in combination with salting. The effect of the conditioning is assessed by measuring the stiffness modulus. The ratio of the stiffness modulus before and after the conditioning is designated Q-winter and expressed in percent (if Q-winter is 100 %, the winter conditioning has no influence). The advantage of the stiffness modulus test, besides its sensitivity to variations of the properties of the test specimen, is its non-destructive mode, i.e. the same test specimen can be tested before and after conditioning. In this case the winter conditioning did not reduce the stiffness modulus of the 'Asphalt Rubber' -test specimens; the actual Q-winter value was equal to or greater than 100 %.

10. Environmental studies

Comprehensive studies have been carried out to explore the impacts of the 'Asphalt Rubber'-concept on the work environment and the environment of surroundings.

10.1. Work environment

Initially a laboratory study was conducted, where bitumen fumes were generated at 160 0 C and cooled down to a condensate. The condensate was analysed chemically to get a picture of what substances could be found in the different bitumen products. The programme for the field measurements was established based on the results from this study. The measurements on the paving site included the content of polycyclic aromatic hydrocarbons (PAH), released from bitumen and asphalts with rubber additives at six different pavement sites during 2008. Measurements were conducted in the inhalation zone of the paver operator and the screed operators, above the augers of the paver and also on the workers at the production unite for ‘Asphalt Rubber’.

The results of the measurements were as follows.

1. The highest PAH-contents arose directly above the augers. The maximum benso(a)pyrene and naphthalene contents of the inhalated air was far below the limiting values

   - 0,03 $\mu g/m^3$of benso(a)pyrene << threshold limit value = 2 $\mu g/m^3$;

   - 2,4 $\mu g/m^3$of naphthalene << threshold limit value = 50 000 $\mu g/m^3$;

2. More PAH was released, when the mixture was strongly heated, as by using a heater especially in combination with remixing.

3. The PAH-exposure was lower at the plant, where rubber was added, than around the asphalt paver at the site. The wind direction and velocity in conjunction with the temperature of the asphalt mix appeared to be the factors controlling the degree of exposure at the paver.

10.2. Leaching tests

The Swedish Geotechnical Institute (SGI) in Linkoping has investigated the leaching of organic substances and compounds from road normal asphalts, produced with and without additives of rubber (from recycled tyres), granulated to particles < 20 mm.

The leaching contents of most of the analysed substances and compounds were low for both materials. The acute toxic effects of the leachate were also low.

The rubber additive caused somewhat increased leaching of other PAHs and of cresoles. However, the contents and the accumulated amounts of these compounds were generally low. There was no carcinogen PAH above the detection limit in any leachate.

The rubber additive generated leaching of bensotiazol, detected by GCMS-screening. The estimated contribution of semi-quantative and accumulated leached contents of bensotiazol to the nearby surroundings of the road were considerably higher for the asphalt with added rubber. However, the preliminary assessment, based on the US limiting value for a closely related compound in surface and ground water, is that leached bensotiazol should not generate any serious contents in leachates, caused by rain on the monolithic road surface of the material. There was also no relation between leached bensotiazol and acute toxic effects.

10.3. Particle emission caused by abrasion from studded tyres

VTI has studied the emission of inhalable abraded particles, using the road test machine, shown in Figure 7. Two runs were conducted; one run with an 'Asphalt Rubber'-pavement, GAP 16, and the other one with a reference pavement, SMA 16. The particle size distribution of the emitted particles and their chemical and morphological properties were analysed, especially PM10 (= the mass concentration of inhalable particles finer than 10 µm, expressed in $\mu g/m^3$), which is regulated by an EU directive and implemented in Sweden as an environmental quality standard.

Altogether, the results showed that the 'Asphalt Rubber'-pavement contributed to lower emissions of both PM10 and ultra fine particles than the reference pavement.

Figure 7. *VTI road test machine for abrasion and particle studies*

11. Concluding remarks

The overall experiences from the studies so far are positive, and there is no reason to discontinue the project. The results from the comprehensive tests and investigations, covering technical, performance-related and environmental aspects, are in agreement with the results, obtained in US.

The remaining parts of the project include:

- Development of a cost effective management of the equipment for involving rubber in the asphalt production process;

- Reduction of the production temperature, which is advantageous for the working environment as well as the emissions to the surroundings.

- Further development of the concept of ‘Asphalt Rubber’- pavements for “low noise pavements” to improve the resistance to abrasion from studded tyres.

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The Use of Asphalt Rubber in a Motorway Section Pavement Rehabilitation in Portugal

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ABSTRACT: Taking advantages of its several benefits, there has been a growing use of asphalt rubber in the bituminous mixes for the pavement wearing course. Asphalt rubber in the surfacing course, among other advantages, can strongly contribute to cope with the reflective cracking and to mitigate the road noise, in maintenance and rehabilitation works.

An open graded asphalt rubber was used in the rehabilitation works of a highly trafficked motorway section (M5), with over 120 000 vehicles per day, for the surfacing course. The motorway section, built in Portugal in the early 40's, was the first one with portland cement concrete slab pavement, with a 2x2 cross section. In the early 90's, the existing pavement was widened to 2x3 and to 2x4, again with portland cement concrete slab pavement, overlaid in the entire cross section with bituminous mixes. During the last years, extensive reflective cracking was observed on the pavement surface. The paper describes the existing previous pavement distresses, the technical solutions and the results, emphasizing the contribution of asphalt rubber to overcome the reflective cracking and to mitigate the road noise.

KEYWORDS: asphalt rubber, pavement rehabilitation, reflective cracking, noise.

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1. Introduction

The motorway section of M5 in Portugal, between Lisbon and the National Stadium Interchange, with a traffic of over 120 000 vehicles per day, has a length of 8 km and the pavement is composed by concrete slabs covered with bituminous layers. The original pavement was in concrete and built in the decade of 1940, being the first motorway in Portugal, with two carriageways of two lanes each.

In 1988 the Portuguese main motorway concessionary (BRISA) was committed to carry out the pavement widening to three lanes in each carriageway. In early 90's the widening and rehabilitation works took place, with concrete slabs both inside and outside of the existing cross-section and then the entire platform was covered with bituminous layers.

In the last years, extensive longitudinal and transverse cracking, among other localized distresses, were observed on the pavement. BRISA decided to carry out pavement rehabilitation works and in the year 2006 the engineering consultant company COBA was committed to undertake the study of the sub-section between km 5+500 and km 8+000.

The rehabilitation works were carried out during the month of August of 2007, which is the traditional period of holidays with less traffic in the motorway and were restricted to night time and weekends.

One year later, during the month of August of 2008, the rehabilitation works of sub-section between km 2+700 and km 5+500 took place in similar conditions and in both cases an open graded asphalt rubber layer was laid as a finishing course with very promising results. Sub-section between km 0+000 and km 2+700 is scheduled to be rehabilitated in August 2009.

2. Description of the Existing Pavement Condition

The distresses observed on the bituminous surface before rehabilitation were mostly longitudinal and transverse cracks, mainly originated from the underlying concrete slabs joints. A frequent occurring situation was the existence of a longitudinal crack approximately along the middle of the centre lane, with a path apparently associated with the longitudinal joint between the old concrete adjacent underlying slabs. The thickness of the bituminous layers acknowledged from cores extracted from the pavement, varied between a minimum of 9,5 cm to a maximum of 22,5 cm, with frequent values of 14 to 18 cm.

The sub-section from km 5+500 to km 8+000, was the one which presented more defects on the pavement. On the left carriageway, between the underpass located at km 7+850 and about km 7+600, the pavement was comparatively in worst condition, with several types of distresses occurring. Close to the underpass joint there was a substantial settlement on the pavement. Deformations mainly of longitudinal surface depression type, were observed mainly on the right lane, with substantial ruts between km 7+700 and km 7+600. On the acceleration lane, approximately between km 7+450 and km 7+270, deformations and cracking were observed. An illustrating schematic diagram and photo are presented in Figures 1 and 2.

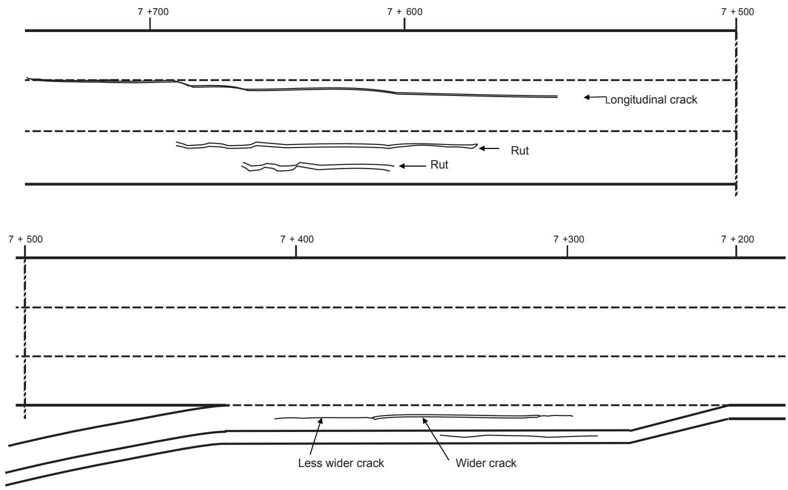

Figure 1. *Schematic diagram with the location and the type of previous existing distresses on the left carriageway pavement*

Figure 2. *Deformations and ruts on the left carriageway right lane pavement, approximately at km 7+600*

Deflection tests were carried out to assess the pavement structural condition, using a Carlbro van-integrated PRI 2100 model Falling Weight Deflectometer. The results obtained normally didn't present values larger than 100 µm, indicating that the pavement was generally in good structural condition. However, in some zones the values were significantly higher. For example, close to the longitudinal crack of the central lane, on the left carriageway, between km 7+700 and km 7+600, deflection values on the plate center (Df1) were about 450 µm (fig 3).

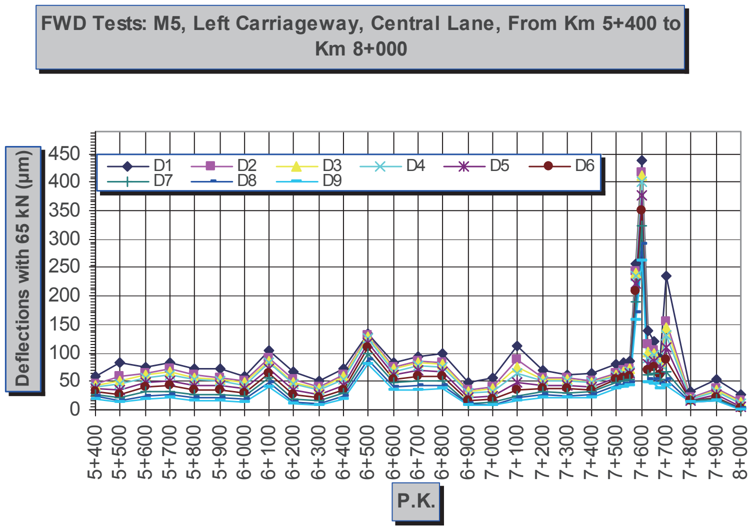

Figure 3. *Deflections with the Falling Weight Deflectometer on the central lane of the left carriageway.*

In order to assess the pavement structure (thickness and layers), crack depth and layers bond, some cores samples have been taken. Figure 4 shows two of those cores samples, and it can be observed the concrete slabs underlying the bituminous layers (the older one of 40's and the latest from the widening of 90's) as well as the crack propagated through the bituminous layers.

Figure 4. *Cores samples extracted from the pavement and the crack propagation through the bituminous layers can be observed*

The sub-section between km 2+700 and km 5+500 and the sub-section between km 0+000 and km 2+700, were comparatively in better condition than the above described. The distresses were again mostly longitudinal and transverse cracks, mainly originated from the underlying concrete slabs joints.

3. Rehabilitation Works

3.1. Rehabilitation Solutions

Several solutions were developed to solve the pavement distresses of the sub-section between km 5+500 to km 8+000, the first of the three sub-sections where BRISA decided to carry out the rehabilitation works. The objective was to eliminate or retard with good efficiency the crack propagation to the bituminous surface and reinforce the structural capacity of the weak zones, which were detected in the deflection tests. In addition, the final surface after rehabilitation, should also meet the BRISA strict specifications for motorways, concerning riding quality, skid resistance and texture.

The longitudinal regularity evaluated by the International Roughness Index (IRI), in 100 m intervals, after rehabilitation, should comply with the following:

- Max. IRI of 1,5 m/km to be achieved in each batch, in more than 50% of measurements of the analyzed batch;

- Max. IRI of 1,8 m/km to be achieved in each batch, in more than 80% of measurements of the analyzed batch;

- Max. IRI of 2,0 m/km to be achieved in each batch, in 100% of measurements of the analyzed batch.

The existing values (year 2006) before rehabilitation works are summarized in Table 1:

Table 1. *Values of I.R.I measured in year 2006*

Eastbound

IRI (m/km)	<3	<2,5	<2	<1,8	<1,5
%	92.6	77.8	22.2	11.1	0.0

Westbound

IRI (m/km)	<3	<2,5	<2	<1,8	<1,5
%	100.0	82.4	58.8	41.2	17.6

The solutions were named as corrective actions and labelled from A1 to A8. A1, A2, A6 and A8 were mainly related to the milling of the existing bituminous surface layers, cracks sealing and the placing of glass grids and bituminous mixes. A3, A4 and A5, were related to the treatment of the damaged slabs, using high initial strength concrete and the filling of the gap between slabs with grout.

After all the corrective actions have finished, an open graded asphalt rubber mix was laid as surface course, with 3 cm of thickness, on the top of a underlying 5 cm asphalt concrete layer.

The IRI final results after the rehabilitation works were finalized, at the beginning of September 2007, are presented in Table 2:

Table 2. *Values of I.R.I (m/km) measured after rehabilitation works*

Eastbound

IRI	**<3**	**<2,5**	**<2**	**<1,8**	**<1,5**
%	100,0	100,0	100,0	91,7	54,2

Westbound

IRI	**<3**	**<2,5**	**<2**	**<1,8**	**<1,5**
%	100,0	100,0	100,0	95,8	83,3

Deflections measurements were carried out in order to assess the effectiveness of the rehabilitations solutions, along parallel alignments in the vicinity of the former cracks.

Deflections measurements before and after rehabilitation works regarding the areas where corrective actions A3 and A5 were carried out, are presented in Tables 3 and 4. The results showed significant differences between the two sets of measurements, with a considerable decrease of the deflections.

Table 3. *Deflections measurements before and after rehabilitation works regarding the area covered by corrective action A3*

P.K.	Before Rehabilitation Deflections in µm		After Rehabilitation – Deflections in µm							
			Alignment 1		Alignment 2		Alignment 3		Alignment 4	
	D1	D9	D1	D9	D1	D9	D1	D9	D1	D9
7+700	387	57	93	28	90	27	122	43	143	46
7+680	271	115	79	31	82	31	156	48	116	38
7+660	492	104	81	29	78	31	107	35	112	37
7+640	400	96	73	26	78	27	105	37	113	38
7+620	379	174	83	32	81	31	117	40	107	39
7+610	457	183	76	28	96	29	119	41	134	44

Table 4. *Deflections measurements before and after rehabilitation works regarding the area covered by corrective action A5*

P.K.	Before Rehabilitation Deflections in µm		After Rehabilitation – Deflections in µm							
			Alignment 1		Alignment 2		Alignment 3		Alignment 4	
	D1	D9	D1	D9	D1	D9	D1	D9	D1	D9
7+760	135	43	61	20	57	19	109	25	80	26
7+750			94	30	97	32	140	44	114	38
7+725			98	31	96	30	102	30	106	31
7+700	236	44	164	48	191	44	129	40	168	48
7+675			150	41	126	43	115	36	115	37
7+650			108	30	115	37	109	33	111	37
7+625	139	47	115	38	105	37	98	32	129	38
7+600	439	262	140	52	123	42	127	47	141	46
7+575	256	157	112	41	115	47	103	36	131	43
7+550			120	41	115	42	103	36	116	38

As mentioned previously, sub-section between km 2+700 and km 5+500 was generally in better condition. In the rehabilitation works of this sub-section, the existing bituminous surface was subjected to a micro grinding, previous to the placing of the asphalt rubber wearing course, in order to eliminate the irregularities.

After all the corrective actions have finished, which included sealing and laying of a glass grid on persisting cracks, an open graded asphalt rubber mix was laid as surface course, with 4 cm of thickness. This thickness was recommended as the minimum above the glass grid by the supplier of this product.

3.2. Asphalt Rubber Mix Layer

The open graded asphalt rubber mix was composed by fine and large aggregates, crumb tire rubber modified bitumen and a commercial filler.

A 35/50 pen grade asphalt cement base was modified adding crumb tire rubber, in a proportion of 21% in weight (over total weight of binder), complying with specifications (%varying between 20 and 23%), with a specified grading and in appropriate blending and reaction conditions.

This modification is made with appropriate equipment, which is installed next to the conventional bituminous plant, with a "by-pass" to the circuit of the original binder, between the bitumen storage tank and the bituminous plant mixer. This equipment heats and mixes the original binder with the rubber, allowing a chemical/physical reaction ("digestion") between these two materials.

The crumb rubber specification for the grading was according to Table 5:

Table 5. *Grading of the crumb rubber*

Grading	
Sieve opening ASTM	**% passed**
2 mm (n.º 10)	100
1,18 mm (n.º 16)	65 – 100
0,6 mm (n.º 30)	20 – 100
0,3 mm (n.º 50)	0 – 45
0,075 mm (n.º 200)	0 – 5

The requirements (ASTM D6114 Standard) for the rubber modified bitumen were according to Table 6:

Table 6. *Requirements for the rubber modified bitumen*

Tests	Requirements
Apparent Viscosity Brookfield at 175ºC, cP (AASHTO TP48)	2000-4000
Cone Penetration at 25º C 100 g, 5s, 1/10 mm (ASTM D5)	20 minimum 75 maximum
Resilience at 25ºC (ASTM D 3407)	15% minimum
Ring and Ball Temperature (ASTM D 36)	54º C minimum

The mix aggregate grading specification was according to the following (Table 7):

Table 7. *Grading of mix aggregate*

Sieve opening ASTM	% passed
12,5 mm (1/2")	100
9,5 mm (3/8")	90 – 100
4,75 mm (nº 4)	35 – 50
2,00 mm (nº10)	6 – 10
0,425 mm (nº40)	3 – 7
75 µm (nº200)	2 – 3

The amount of commercial filler in the bituminous mix is, at least, 2% by weight of the mineral aggregate and should be Portland cement or hydraulic lime appropriately hydrated.

The bituminous mixing formulation is composed by two distinct stages. At the first stage the physical characteristics of the mixture are determined by the Marshall method and two distinct mixtures, at most, are defined, to be tested at the second stage.

The following values were specified for the test results of the bituminous mixture with the Marshall method:

- Number of blows in each specimen edge face 50
- VMA value (percentage of Voids in the Aggregade mix), minimum 25%
- Porosity .... 9 to 15%
- Retained strength minimum 80%
- Percentage of rubber modified binder .... 9,5% to 10,5%
- Cantabro Loss, maximum 15%

At the second stage, an experimental section is carried out whereby, after appropriate conclusions are drawn out, the final mixing composition to be used on job site is defined and approved by the site Engineer.

The job composition used was composed by 49% of basaltic 4/6 fraction, 39,9% of granodioritic 6/10 fraction, 1,8% of hydraulic lime and 9,3% of rubber modified bitumen. The asphalt rubber was produced by the "wet process".

A thermo-adherent polymer modified emulsion tack coat was placed with a 0,35 kg/m2 rate previous to the laying of the asphalt rubber wearing course.

The rubber asphalt layer was placed and finished with wide pavers, working in parallel in order to cover the entire carriageway, avoiding thus longitudinal construction joints.

Figure 5 shows the placing of the rubber asphalt wearing course.

Figure 5. *Placing the rubber asphalt wearing course*

The reduction of tire-road noise was evident, with significant difference in comparison with the remaining sections of traditional concrete asphalt wearing course. The reductions achieved are approximately of 5 dB, according to measurements already carried out in another section of road in Portugal.

4. Conclusions

The use of asphalt rubber is experiencing an interesting growth due to the advantages of its properties regarding resistance to crack propagation and noise reduction, as well as those related to the environmental issues.

Asphalt rubber has been used since 1999 in Portugal and started with the rehabilitation of single carriageway roads. Considering the encouraging experience and the results achieved, the main motorways concessionary BRISA has recently decided to use this technology in the pavement rehabilitation of some sub-sections of two motorways (M5 and M9) in the Lisbon district.

The paper described a pavement rehabilitation case of a highly trafficked motorway section, in which the use of an open asphalt rubber was considered a good option for the wearing course, in combination with other solutions for the underlying layers.

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Crumb Rubber From Scrap Tyres For Use in Asphalt Pavements in the UK

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ABSTRACT: An examination of the potential advantages and disadvantages of scrap tyre crumb rubber in road construction (Crumb Rubber Modified Asphalt, CRMA), in comparison to traditional asphalt. The paper highlights the economic influence of UK government initiatives such as the Landfill Tax, Aggregate Tax and proposed Incineration Tax before considering the financial implications of the performance of CRMA in comparison to traditional asphalt and the potential reduction in pavement thickness and increase in service life. Examining the friction properties and cold weather performance of the pavement as well as environmental and noise pollution issues and the subject of scrap tyre recycling: The paper brings together research, experience and best practise from around the globe to draw conclusions for the suitability of CRMA on the UK road network.

KEYWORDS: Crumb, Rubber, Asphalt, UK, Noise, Environment.

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1. Introduction

In Britain over 100,000 tyres are removed from cars, vans and trucks every day, resulting in an annual disposal requirement for around 440,000 tonnes[1]. Until recently the UK recovered value from around 70% of these with the balance largely being disposed of to landfill, with one cubic metre of landfill space required for every 17 whole tyres[2].

In the UK, in July 1999, this traditional, yet environmentally harmful, means of disposal was addressed by the Landfill Directive. This prohibited the disposal of whole tyres in new landfill sites by July 2003 and shredded tyres by July 2006[3]. With increasing pressure to determine an alternative means of scrap tyre disposal, one potential use crumb rubber modified asphalt (CRMA).

Research to improve and enhance the performance of asphalt through the addition of natural and synthetic rubber has been undertaken worldwide for some considerable time. Historically, the objective has been to improve the asphalt's physical characteristics through improvements in its elasticity. Results of several demonstration projects have also indicated increased fatigue resistance, a reduction in reflection cracking, improved skid resistance and increased durability[4].

Despite apparent advantages, CRMA has received mixed reception across the world. The United States, is currently the largest user of CRMA; by as early as 1991, Arizona had paved in excess of 10% of its highway network with CRMA, whilst Texas had paved nearly 4,000 lane miles and recycled 3.5M tyres[5].

2. Aim.

This paper aims to examine the potential advantages and disadvantages of the use of scrap tyre crumb rubber in UK road construction, concentrating upon its use in the friction or wearing course layer.

3. Background.

A one-inch CRMA surface course recycles the equivalent of 375-2,200 scrap tyres per lane mile[6]. Assuming that all 203,000[7] miles of the UKs roads were just two lanes wide and that just 5% were paved annually with one inch of CRMA this would require 13.8M tyres (138,000 tonnes), almost twice the number of tyres disposed to landfill in 1999[8]. (Calculated conservatively at 600 tyres per lane mile.).

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1 Used Tyre Working Group.
2 Manhole Adjusting Contractors Inc.
3 For existing landfill sites there was potential flexibility for continuing to landfill tyres beyond 2006.
4 Takallou & and Sainton.
5 Manhole Adjusting Contractors Inc.
6 Manhole Adjusting Contractors Inc.
7 Institute of Civil Engineers, Local Transport and Public Realm Survey.
8 6th annual report of the Used Tyre Working Group

There are many factors to consider when specifying the use of rubber in pavements including cost, specifications, type of equipment to be used, expertise of the contractor and subsequent recycling potential. The reported advantages and potential issues associated with CRMA include:

Table 1. *Advantages and potential issues surrounding CRMA.*

Advantages	Potential Issues
• Reduced pavement thickness. • Increased pavement life. • A reduction in reflection cracking. • Reduced traffic noise. • Reduced maintenance costs. • Reduced pollution and increased environmental quality.	• Potentially high initial costs. • Examination of lifecycle economics. • An apparent lack of a specifications. • The requirement scrap tyre uniformity. • Environmental concerns including the ease of recycling CRMA. • The potential requirement for modifications to asphalt plant or equipment.

4. Engineering Analysis

The methods of modifying asphalt, through the use of crumb rubber, are simplified into two processes:

- **The Wet Process.** The Wet Process mixes the crumb rubber with the binder, allowing time for their reaction prior to mixing with the aggregate. This results in a pavement mixture known as Asphalt-Rubber.

- **The Dry Process.** The Dry Process simultaneous mixes the crumb rubber, the binder and the aggregate. The range of materials produced using this method is collectively known as Rubberised Asphalt.

Throughout, the paper will utilise the term crumb rubber modified asphalt (CRMA) to include both Asphalt-Rubber and Rubberised Asphalt. In instances where information is specific, to either Asphalt-Rubber or Rubberised Asphalt, then the material will be identified as appropriate.

The crumb rubber utilised in CRMA is primarily derived from scrap tyres. Once collected from the point of recovery, the tyres, with appropriate processing, will yield 4 to 5 Kg of crumb rubber[9] per tyre.

4.1. Collection

The detailed examination of the comprehensive collection and processing system, required to ensure efficient and economical compliance with the requirement to recover 100% of

9 Emery.

discarded scrap tyres, is beyond the scope of this paper. However, initial examination reveals that a quantum change is necessary to meet the additional demand associated with complete recovery, primarily focussing upon greater recovery capacity and efficiency. The current cost of transporting used tyres to the reprocessor comprises a significant element (as much as half)[10] of the overall tyre recovery charge. There is therefore potential for an increase in illegal disposal as the Landfill Directive takes full effect.

One means of reducing illegal disposal is through increased accountability and the ability to identify and track individual tyres from production to disposal. A proposal which would undoubtedly incur significant administrative costs and resistance from garages, motorists and producers alike. Conversely such a system may lead to a reduction in theft if the police were also able to access information linking, cars, tyres and their owners.

Recycling may also be encouraged through the generation of an economically viable situation; financial incentives offer greater potential for success than a system based upon legislation and enforcement through accountability. Such a system is also more efficient as compliance is increased as a result of the incentive and thus requires only limited policing, rather than enforcement.

4.2 Tyre Shredding & CRMA Production

Shredding is an essential preliminary process, reducing bulk and minimising transportation costs. Tyres, by design, are hardwearing and robust requiring a large amount of energy to create crumb rubber. Cutting and shredding equipment is relatively expensive to maintain due to the abrasiveness of the rubber and the presence of the steel cord and bead wires which blunt the cutting blades. As a result, the main market for shredded tyres is in CRMA, or other construction materials, where the presence of some metal residue is acceptable, albeit large quantities may damage asphalt plant and equipment.[11]

The time required to disperse, blend or react the crumb rubber is dependant upon its physical and chemical properties. Reaction time is also inversely proportional to temperature, doubling with every 10℃ (18oF) decrease in temperature. The susceptibility of the finished product to the physical and chemical composition of the crumb rubber and thus the method of processing is significant. The finer the material the quicker it will react. For example, cryogenic grinding results in a clean flat surface, leading to reduced reaction rates and lower elastic recoveries, in comparison to the ambient grinding processes.

Variations in the rate and duration of the reaction affect the materiel characteristics. Rubberised Asphalt applications have exhibited widely varying performances ranging from acceptable to disastrous, the primary reason for which appears to be poor control in combining the gradations of the aggregate and the crumb rubber. Product and thus performance inconsistency is compounded by a lack of understanding and thus control over the volume changes which occur as the crumb rubber swells during processing and handling.

10 Hallet.
11 Rubber Pavements Association, Crumb Rubber Modifier in Asphalt Pavement.

The Colas test section, using Colsoft material, laid in Surrey, UK, in 1999, had to be re-laid prior to the commencement of the trial. Colas meanwhile, now recommend a maximum transportation time of 30 minutes to prevent aggregate segregation and reaction with the binder. Similarly the Florida Department of Transportation and Public Facilities (DoT) opted not to develop State Specifications for this process.

Quality control is critical to the success of CRMA. Regardless of methodology, processes must be operated and controlled as to afford the contractor, designer and client, confidence in the all stages of the production process. This requires care and attention not only in the supply and production of the crumb rubber but also during its subsequent incorporation with the asphalt.

4.3 Placement of CRMA

4.3.1 Transportation

The transportation of CRMA may be undertaken using any truck typically used for conventional asphalt. A critical factor however, is the adhesion of the material to the truck bed, plant or equipment. In all cases the most effective means of preventing adhesion is through the maintenance of the mix temperature.

The criticality of the compaction temperature increases with the rubber content. Thus it is suggested that there may be a requirement for additional insulation during transportation. Similarly, the traditional image of a fleet of sheeted asphalt trucks, waiting to discharge into the paving machine, is one that is not compatible with CRMA, due to the increased temperature susceptibility. The use of CRMA requires strict coordination, planning and supervision at all stages of production and placement.

4.3.2 Placement

Information regarding the placement of CRMA is contradictory, depending upon the origin of the source and further complicated by the absence of a nationally accepted standard for CRMA, within the UK. The most recent guidance issued in the UK, is the document Civil Engineering Applications of Tyres[12]. This states that Asphalt-Rubber should be heated to between 149-190℃ before compaction and subsequently, laid at a temperature of at least 121℃ using standard paving equipment, with compaction completed as soon as possible. Conversely, in the US, where Asphalt-Rubber has been an accepted pavement material for some time, Arizona (DOT & PF) requires that the temperature of Asphalt-Rubber be at least 135℃ just prior to compaction, and be completed prior to the mix temperature reaching 104℃.

For the Dry mix it is commonly accepted that, during laying, the temperature of the mixture should be at least 121℃ and a finishing roller must continue to compact the mixture until it cools below 60℃ to prevent swelling due to the continued reaction between the

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12 Hylands & Shulman.

bitumen and the rubber. Meanwhile, Road Research Laboratory (RRL) reports[13] state that in order to maintain good workability of the mix and to ensure adequate dispersion of the rubber it is necessary to keep the mixing temperature of the modified materials at approximately 15℃ above those for traditional mixes.

4.3.3 Compaction

Compaction of CRMA requires additional control measures and careful supervision if a quality surface is to be obtained. Due to the "sticky" nature of the modified asphalt, compaction must be undertaken with either vibratory or steel-wheeled rollers. Pneumatic compaction increases rutting, resulting in less effective compaction, as well as increased roller "pick-up". As such, the use of pneumatic rollers should be ignored at all times. The problem of mix "pick-up' maybe reduced through the addition of simple detergent to the roller water spray systems.

The higher temperatures required for the inclusion of crumb rubber means that the rollers are unable to begin rolling immediately behind the spreader, due to the potential for roller "pick-up" and for pushing and shoving in the mat, at higher temperatures. Higher laying temperatures are compounded by the cooling characteristics of the rubberised mat which, in general, cools relatively quickly, to approximately 71-77℃, and then retains heat for longer than traditional mixes[14]. As a result traffic must be kept off the surface for extended periods of time due to the longer cooling time.

Density testing, during rolling, is usually performed using nuclear density meters. However, on site values have been found to be lower than those subsequently obtained from roadway core samples, a factor attributed to the presence of the rubber. Whilst such an anomaly may be resolved through the use of a correction factor, the failure to develop a national standard for the UK has hindered acceptance of this material.

4.4 Reluctance to Accept New ideas

Two separate agencies, or clients, control the road network within the UK. The Highways Authority is responsible for the UK motorway and trunk road network, which comprises approximately 5% of the road network, whilst individual local authorities are responsible for the remaining 95%[15].

The Specification for Highway Works and the Design Manual for Roads and Bridges are the controlling documents for the use of materials in the UK motorway and trunk road network. These documents include specifications for engineering, relating to the construction and subsequent maintenance of roads, including the pavement surface. The use of tyres is not included and as such any scheme utilising CRMA must seek authorisation on a scheme-by-scheme basis with the Highways Agency requiring comprehensive information regarding potential risk and benefits[16].

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13 Szatkowsk.
14 Amirkhanian.
15 ICE, Local Transport and Public Realm Survey.
16 Hylands & Shulman.

Meanwhile local authorities utilise their own specifications, which may permit the use of alternative materials. Contractors and suppliers wishing to utilise alternative materials are again forced to individually tailor their applications for each local authority. Such an arrangement removes any potential savings or benefits of economy of scale and the requirement for scheme-by-scheme approval only serves to deter many contractors from pursuing alternative materials through increased administrative costs and uncertainty.

The UK's performance on tyre disposal compares favourably with that of Europe. However, there is someway to go towards matching the performance of the US. In 1998, the UK recycled just 10.5% of its tyres whilst California recycled 26%, a figure which in 2001 had increased to 42%. It is suggested that the development of standards for the use of tyres in civil engineering applications has contributed to this performance. In addition to regulating the use of tyres, primarily to ensure fitness for purpose, guidance is provided on the testing of physical properties as well as potential pollution aspects.

Thus, disjointed evolution, coupled with an over-riding lack of commitment or direction within the UK has not assisted in the acceptance or development of alternative materials, including CRMA as approved paving materials.

A major barrier to the widespread use of CRMA has been the increase in cost when compared to conventional asphalt; traditionally attributed to the material being in its developmental stage. Early Asphalt-Rubber binder had to be used within a few hours of production, rendering potential users vulnerable to site delays, equipment breakdowns or the vagaries of the weather. Subsequent development of a stable binder removed such financial risk and allowed centralised production and storage. This increased product uniformity and thus consistency of the binder qualities, which in turn aided development of consumer confidence in the quality of the product.

Yet despite these developments in Asphalt-Rubber, the UK still appears to be concentrating its efforts on Rubberised Asphalt. A material which, by the very absence of a reaction during production has a very limited shelf life (less than an hour) between production and placement.

4.5 Performance of CRMA

Historically, CRMA research has aimed to utilise the beneficial properties of rubber to improve the engineering characteristics of the asphalt. It follows that asphalt, with increased elasticity, should show greater resistance to permanent deformation under the traffic loads.

Reflection cracking often occurs in normal asphalt wearing courses laid over existing roads. The onset of reflection cracking can be delayed by laying a greater pavement thickness, effectively strengthening the upper layers. Conversely, increasing the binder content of the material reduces cracking, through increased pavement flexibility. However, this cannot be beyond defined limits for a given geographical location, without the risk of deformation in hot weather.

The increased viscosity of Asphalt-Rubber reduces the tendency to deform at high temperatures, thus allowing the proportion of binder to be increased, to show an advantage in crack prevention. Theoretically, the greater elasticity of Asphalt-Rubber, over traditional asphalt, should increase this advantage further. Whilst Asphalt-Rubber pavements have shown advantages as regards resistance to reflection cracking there is no discernable evidence as to whether this is a result of the increase in binder content (achievable through the presence of the rubber) or the contribution of the rubber to the overall material elasticity.

4.5.1 Deformation and Durability

Whilst several experimental road sections have been constructed in the UK over recent decades, the resultant data was often inconclusive due to the incompleteness or inadequacy of the tests. One such example was the examination of rubberised open-textured bitumen macadam as a non-splash wearing course, conducted on the M40 at High Wycombe in 1967[17]. Expected to improve resistance to compaction, by traffic, of the open-textured surface, its relative performance could not be assessed as no equivalent un-rubberised material was laid. More recently, in 1999, Colas laid a Rubberised Asphalt test section in Surrey, however, there is currently no indication as to the long-term performance of these sections, or the product, Colsoft[18].

The apparent lack of available information regarding CRMA experiments, in the UK, places increased reliance upon experimentation and research conducted elsewhere. These findings, primarily from the US, are applicable to construction in the UK, provided allowances are made for any differences in traffic conditions and climate.

Results of several demonstration projects have indicated increased fatigue resistance, retardation of reflective cracking, improved skid resistance and increased durability[19]. In 1983 the California DoT (CalTrans) compared equal thickness of asphalt-rubber and asphalt in an experiment conducted on Route 395. In 1987, after four years of observations and testing, it was decreed that all future projects involving Asphalt-Rubber would utilise thinner pavement thickness. A two-to-one reduction of traditional asphalt v Asphalt-Rubber is now utilised in most CalTrans projects, with the same reduction strategy being adopted by most local agencies in California.

Later, in 1988, the Florida DoT commenced a comprehensive 10 year-long performance evaluation of various CRMA mixes. Results from this period are shown below in Figure 1 to 6[20]. It can be seen from the cracking and patching data in Figure 2 that all sections with Asphalt-Rubber showed approximately 1 to 6% cracked areas, depending upon the amount of rubber, whist traditional asphalt or Rubberised Asphalt returned 30% cracked areas. Additionally, the cracking is relatively insignificant in sections containing 10 and 15% rubber, therefore suggesting that the optimum rubber content lies within this range.

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17 Thompson & Szatkowsk.
18 Harding.
19 Takallou & Sainton.
20 Choubane, Sholar *et Al*

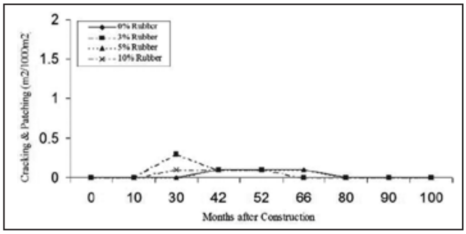

Figure 1. *Cracking and Patching versus Time on SR-120 Demonstration Project.*

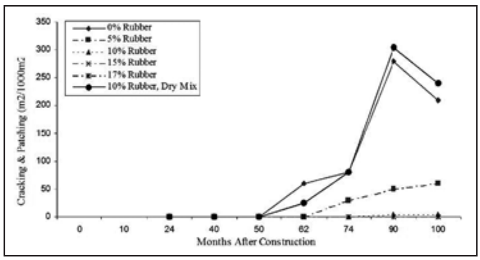

Figure 2. *Cracking and Patching Versus Time on SR-16 Demonstration Project.*

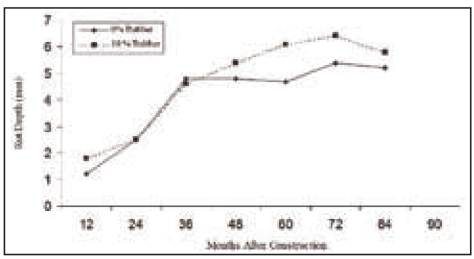

Figure 3. *Rut depth Versus Time on I95-Southbound Demonstration Project.*

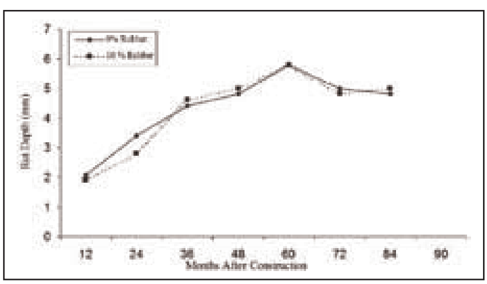

Figure 4. *Rut depth Versus Time on I95-Northbound Demonstration Project.*

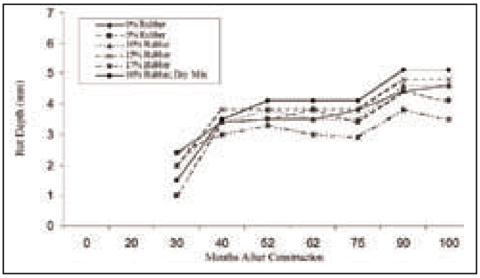

Figure 5. *Rut depth Versus Time on SR-16 Demonstration Project.*

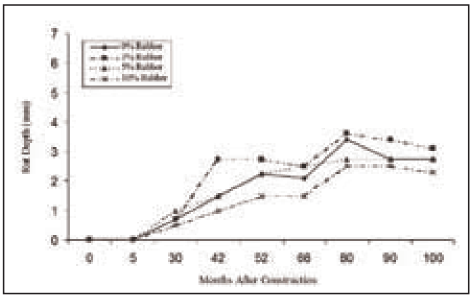

Figure 6. *Rut Depth Versus Time on SR-120 Demonstration Project.*

Whilst crack data, as a performance indicator, shows clear advantages from the use of Asphalt-Rubber, the benefits, when measuring rut depth, are less apparent for low rubber contents. Figures 5 and 6 indicate that the benefits of Asphalt-Rubber are difficult to determine when only small percentages of rubber content are utilised. However, as the rubber content increases an improvement in performance is measurable, as shown at Figure 6, which indicates the lowest rut depth at 17% Asphalt-Rubber whilst Rubberised Asphalt returned the worst results.

In conclusion it is evident from the data collected during the Florida DoT evaluation that all Asphalt-Rubber sections performed significantly better than the traditional and Rubberised Asphalt test sections. It should be noted that research, by Takallou and Sainton, claims Rubberised Asphalt mixes with laboratory fatigue lives ranging from 2 to 7 times longer than conventional mixes, however, these have yet to be confirmed in the field.

As a result of the findings of the 10 year performance evaluation the Florida DoT initiated the implementation of specifications requiring the use of crumb rubber (specified as ground tyre rubber) in all asphalt surface mixes. Similar conclusions were drawn in May 1963, in the UK, when inspection of the A15 at Yaxkley identified a 50% increase in pavement life compared with normal materials. The Road Research Laboratory (RRL)[21] concluded that there was sufficient evidence to indicate that, the resistance to reflection cracking was almost

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entirely due to the properties of the modified asphalt, which almost entirely eliminated cracking for the five years of observations. Despite these findings, after 45 years, CRMA is still not a widely accepted materiel in the UK construction industry.

This early UK report also stated that Asphalt-Rubber appeared to show no advantage on lightly trafficked roads[22]. On cursory examination this is a disappointing comment for advocates of CRMA technology, however not unexpected. CRMA shows potential for improving the performance of road pavements under vehicle loadings through utilisation of the characteristics of the added rubber. It follows that if the stresses induced in the pavement are negligible, then there will be limited advantage to be gained from improving the pavements elasticity through the addition of rubber to the mix. In such scenarios the pavements deterioration will be influenced more by the cumulative action of weathering and ageing, than traffic loading.

4.6 Maintenance, Repair & Utility Work

It has already been identified that CRMA requires stricter control measures than for traditional asphalt. It is therefore proposed that CRMA is less suitable for the small, piecemeal, quantities often associated with the reinstatement of utility works. Thus, when assessing the suitability of CRMA as a pavement material consideration must be given to the potential impact of subsequent utilities work.

The use of dissimilar adjacent materials can lead to accentuated aging and subsequent failure, at the material interface; where a relatively rigid pavement material (traditional asphalt) is laid adjacent to a relatively compressible one (CRMA). CRMA may therefore be more suited to motorways and main truck roads where minimal disturbance by future utility work is anticipated. Conversely, CRMA is not unsuited to locations where frequent utility work will be conducted, however additional consideration should be given to reinstatement.

It should also be noted that in the event of a road traffic incident (RTI) the characteristics of Asphalt-Rubber differ from those of traditional asphalt. The increased viscosity, and associated higher mixing and laying temperatures, increases the pavements resilience, to heat damage in the event of a fire. This benefit is only marginal however as the temperature difference required during production, 15℃, is negligible in comparison to the temperatures experienced during a fire. The use of CRMA also increases the pavements susceptibility to damage from fuel or chemical spillage, often associated with major RTIs. Solvents degrade both the traditional binder and the rubber component of the pavement, increasing the volume of material area requiring replacement in comparison to a traditional pavement.

4.7 Safety

The use of CRMA raises several potential safety issues which must be considered in its evaluation. The potential increase in pavement service life offered by CRMA reduces the requirement for roadworks to repair or refurbish the pavement. It has been shown that the frequency of RTIs is up to 70% higher in roadworks than in normal flowing traffic. Thus, if

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CRMA pavements offer an increased service life, the frequency of roadworks will be reduced, in turn lowering the number of RTIs. The primary concern however is the pavements ability to provide safe driving conditions.

4.7.1 Road User Safety

In general, a dry pavement surface provides good tyre-pavement friction whereas a wet pavement reduces skid resistance. For speeds in excess of 40 mph, pavement macro-texture assists in reducing hydroplaning, by allowing the water escape from the tyre-pavement interface. Macro-texture testing indicates that CRMA pavements have better texture characteristics than conventional mixes, primarily due to their improved waterproofing characteristics[23].

Figure 7. *San Antonio Interstate Highway IH 35 before and after paving with Asphalt Rubber (RPA News, Vol 7, No.4, Spring 2004).*

Macro-texture may be described as a combination of stone porosity and the void ratio of the pavement as a whole. Whilst these characteristics remain unchanged, the rubber component within CRMA coats the stone, effectively waterproofing it, and thus increasing the speed of dispersal of the water from the road. Thus, the use of CRMA generates better overall surface conditions and reduces vehicle-stopping distances by allowing the surface to drain more effectively.

Increased visibility, during adverse weather conditions, will theoretically reduce the frequency of RTIs. However, poor visibility is a key factor in reducing vehicle speed and increasing vehicle separation, as drivers compensate for the road conditions. Thus, a disproportionate increase in visibility, in comparison to stopping characteristics, could potentially be disadvantageous and therefore requires closer examination.

Examination of the accident statistics and climatic data for IH 35, San Antonia, Texas,[24]

23 Kuennen.
24 Rubber Pavements Association News, San Antonia Asphalt Rubber, 2004.

(shown in figure 7), shows that despite an increase of 34.6% in the number of wet days in the year following re-surfacing, major accidents were reduced by over 43% under all conditions and by more than 51% on wet days. Figures 8 and Figure 9, show the skid resistance data obtained as part of the Florida DoT 10 year-long performance evaluation[25] indicates that both Asphalt-Rubber and traditional asphalt pavements provided similar friction performance. Thus, on the basis of this information, there is no advantage or disadvantage from the use of Asphalt-Rubber when considering the long-term skid resistance as a performance indicator.

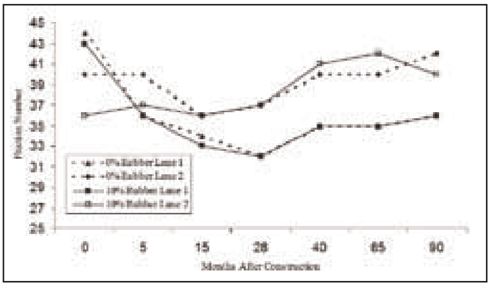

Figure 8. *Friction number versus time on I-95, Northbound, demonstration project.*

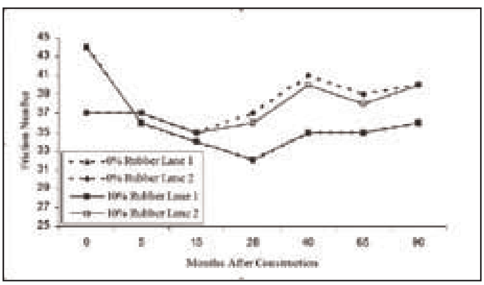

Figure 9. *Friction number versus time on I-95, Southbound, demonstration project.*

4.7.2 Cold Weather Maintenance and Road Safety

Ice reduces friction, preventing vehicle tyres interacting with the road surface. Winter route maintenance traditionally utilises a sand or salt mixture applied prior to and during the formation of snow and ice on the road. The rheological properties of Rubberised Asphalt, which increase its rutting resistance, by allowing temporary deflection under load, also serves to reduce susceptibility to inclement weather. The increased elastic response of the material reportedly causes ice formed on the pavement to break under transient vehicle loadings.

Takallou[26], in detailed laboratory investigations and Canadian field trials considered the effect of temperature upon Rubberised Asphalt, concluding that at –6 ℃ the rubber

25 Choubane, Sholar *et Al*.
26 Takallou,Hicks & Esch.

particles lose their elasticity and act as a weak aggregate. At higher temperatures however, they act more as an elastic aggregate. Therefore, at temperatures required for ice formation, the flexibility afforded by Rubberised Asphalt offers the potential to reduce ice formation, increasing friction and thus potentially reduce the incidence of accidents due to inclement road conditions.

CRMA contains Carbon Black, a component of the scrap tyre, the presence of which results in the pavement retaining its darker appearance for a longer period of time. In addition to generating the image of a freshly laid surface for longer and thus increasing public perception of the performance of the local authority or highway agencies, the darker colour has additional benefits. Both Minnesota and Alaska DOT & PF have recorded additional de-icing benefits[27], attributed to the darker colouring resulting in the absorption of more solar energy than traditional asphalt. This serves to not only delay the onset of ice but also causes the snow to melt faster. This benefit appears to be based upon anecdotal evidence and whilst logical reasoning and the application of basics physics supports such a claim, the author was unable to identify any scientific or numerical quantification of this characteristic. Further research is therefore required to identify the impact of carbon black upon the thermal properties of CRMA and whether this holds any potential road safety benefits.

The use of salts and chemicals during winter route maintenance programme creates an aggressive environment, reducing the service life of road infrastructure such as bridge bearings or other exposed metal elements. Additionally, sand, salt or chemicals will normally only reduce stopping distances for a short period of time as they quickly become dispersed, due to the action of passing vehicles. Frequent reapplication is therefore required, increasing the aggressiveness of the immediate environment and the logistical burden of transporting and distributing the de-icing material.

The continually varying road surface conditions, which result as the material is dispersed and ice reforms, renders driving conditions more hazardous. This is compounded by the findings of the Alaska DOT & PF who installed 12 experimental sections[28] to analyse the performance of Rubberised Asphalt[29]. Conducted under icy conditions with some roadway sand occasionally present, test results indicated an average 25% reduction in stopping distances. However, it should be noted that when tested during the summer, the Rubberised Asphalt pavements, demonstrated lower friction numbers in comparison with the conventional surfaces. CRMA can afford similar, if not improved, friction values when compared to its standard counterparts, as shown earlier in Figure 8 and Figure 9. It is concluded that, had the Rubberised Asphalt, exhibited the same summer friction values as conventional mixtures, then the cold weather results would have been further improved, thereby strengthening the case for the use of Rubberised Asphalt in regions susceptible to snow and ice conditions.

4.7.3 Constructor and Maintainer Safety

Whilst CRMA will not reduce or remove the dangers associated with construction, a reduction in the requirement for road maintenance will reduce the exposure frequency of

27 Takallou, Hicks & Esch.
28 Totalling in excess of 34 lane miles constructed between 1979 and 1987.
29 Takallou, Use Of Rubber Modified Asphalt Pavements In Cold Regions.

construction workers to such potential hazards. It can also be seen that if the use of CRMA allows for a reduction in pavement thickness, there will be a corresponding reduction in construction activity. Thus, CRMA offers the potential to reduce the degree and frequency, of exposure to the hazards of road construction and maintenance.

When considering any material, it is necessary to evaluate the properties and characteristics of the material. This should include the information on the safety on the material and include all aspects from its manufacture to disposal. One area to note is that the presence of crumb rubber, can result in fumes and smoke at typical compaction temperatures. However, the author was unable to find tests or statistics specifically relating to UK health and safety legislation as regards exposure levels for emissions during asphalt production and placement. Internationally, data is also limited in its availability and examination of Emery's [30] findings indicates that air emissions for both conventional and CRMA processes overlap and exhibit wide variability. It is concluded that there is no discernable difference between the emissions, rather that the variance is influenced by characteristics of the operation and maintenance characteristics of each plant and rather than differences in the material.

5. Environmental Analysis

Traditional research has focused on the addition of rubber to improve the engineering characteristics of asphalt pavements but, with increased environmental awareness backed by European and UK directives and legislation, emphasis is now upon scrap tyre disposal. This is recognised by the ICE which publicly[31] supports the need to protect and enhance the environment and to use resources in a way that does not disadvantage future generations; the achievement of these objectives requires solutions which strike an informed balance in terms of cost, benefits, sustainability and acceptability within the broader legislative framework, and involves an evaluation of whole life costs.

The implementation of CRMA as the default material, for highways, offers the potential for environmental savings: Thinner pavements, utilising the CalTrans findings, reduce the materials required by 50%. Conversely the use of crumb rubber does require the rubber to be processed, which has its own energy requirements. However, as a result of the landfill directive the majority of tyres require processing irrespective of the disposal or recycling regime implemented. Thus, a reduced pavement thickness continues to offer significant environmental benefits.

CRMA must be evaluated from an environmental as well as financial and engineering material performance perspective to determine the true magnitude of potential benefits. Due consideration must also be given to the recovery and ease of recycling of the CRMA once its useful life has expired. The possible long term contamination of asphalt through the addition of rubber, without consideration of the environmental impact may only serve to exaggerate the pollution problem.

CRMA, as a means of tyre disposal, must not be considered in isolation and whilst outside

30 Emery.
31 Institute of Civil Engineers, ICE Extra.

the scope of this report, a comparison of environmental and financial factors must be made against other disposal methods.

5.1 Noise Reduction

Classed as environmental pollution, excessive noise in urban or residential areas is an increasing nuisance as traffic volumes and vehicle speeds increase. It is thought that more than half the homes in England and Wales are exposed to noise levels exceeding the World Health Organisations recommended daytime level of 55dB[32].

For automobiles, the vast majority of the noise is generated by the interaction of the tyres and pavement whereas for heavier vehicles the engine and exhaust are the main sources. CRMA obtains its noise-reducing properties from a combination of surface porosity and ductility. As a result, whilst tyre noise is reduced, engine and exhaust noise are not appreciably affected. Therefore, a pavement carrying primarily automobile traffic will exhibit a greater decrease in traffic noise following paving with CRMA, than one which has a high percentage of heavy vehicles.

Research has identified significant noise reduction through the use of open graded pavement design and the inclusion of crumb rubber[33]. In Europe, Asphalt-Rubber surface courses have been used to reduce noise levels and increasing skid resistance[34], with results indicating up to a 10dB reduction, equivalent to a 90% reduction in noise. Whilst the results varied depending upon design criteria, in all instances they returned a noise reduction of no less than 50%[35]. Table 2 details the findings of noise research in Sacramento[36], the results of which have been normalized for speed and traffic volume to isolate the noise-reducing properties of the paving materials:

Table 2. *Rubber Modified and Conventional Asphalt Noise Test Results, Sacramento County Roadways.*

Ser (a)	Location (b)	Pavement type (c)	Time elapsed after paving (d)		Change in noise level (e)
1.	Alta Arden Expressway	CRMA	1	Month	-6 dB
2.			16	Months	-5 dB
3.			6	Years	-5 dB
4.	Antelope Road	CRMA	6	Months	-4 dB
5.			5	Years	-3 dB
6.	Bond Road	Conventional Asphalt	1	Month	-2 dB
7.			4	Years	0 dB

32 Environment Agency, Noise from using tyres.
33 The phenomenon was first identified in Brussels in 1981 in an Asphalt-Rubber called “Drainasphalt”.
34 Manhole Adjusting Contractors Inc.
35 The Rubber Pavements Association, Noise Reduction with Asphalt-Rubber.
36 Sacramento County Public Works Agency.

Immediately after paving, traffic noise decreased along all three roadways, albeit with some subsequent loss in noise reduction properties over time. Compaction of the asphalt overlay reduces the porosity of the road surface, which accounts for some of the pavements noise reduction properties. This compaction is deemed to be complete within one year of paving. After this period, pavements paved with CRMA still exhibited good noise reduction, whereas the noise reduction properties of the conventional asphalt have been lost.

Evaluation of noise reduction properties is complicated by several factors: pavement works often alter roadway geometry, impacting upon both vehicle speed and noise emissions. Traffic densities may change over time as a result of the changes in road geometry and network configuration, as well as being subject to the vagaries of external factors not necessarily factored into consideration. In normalisation of the figures, for comparison purposes, there is potential for figures to be adjusted to suit the desired outcome and as such one must treat noise reduction data with caution, unless the full facts surrounding the collection of data are known.

The use of CRMA can assist in the reduction of traffic noise, with obvious benefits for applications in noise sensitive areas such as near schools, hospitals or residential areas. Mitigating noise emissions can also reduce or even eliminate the requirement for expensive and aesthetically instructive sound barriers.

5.2 A Potential Pollutant?

When assessing the suitability of a material, for inclusion in construction, consideration must be given to its physical and chemical stability during service conditions and other scenarios. Thus when a tyre, in whatever form (i.e. whole or crumbed) is used in a soil environment, its total contents and the leachability of regulated elements, as well as engineering properties must be taken into consideration.

Tyres reused on land or landfilled will leach, and research indicates that this is increased through exposure to ultraviolet light and the effects of acidic rainwater[37]. Westerberg meanwhile concluded that tyre shreds, in unsaturated applications, have a negligible effect on the surrounding surface or ground water quality. However, it should be noted that Westerberg recommended that the free iron cord length be minimised, in order to reduce rust formation and subsequent release. Additionally, that the tyre particles be washed and free from contaminants. These recommendations, whilst predominately identified for the use of scrap tyres as engineering fill, are equally applicable to their application in road construction if potential pollution concerns are to be addressed.

Whilst no supporting documentary evidence was found, the author concludes that the incorporation of rubber particles into asphalt would serve to reduce potential contamination to levels below those indicated by Westerberg in his assessment of tyres in their raw state. The interaction of the rubber with the asphalt binder will lead to increased stability, minimising material or potential pollutant migration and as such the leaching of the component materials of tyres is considered negligible. Further research or dissemination of research data in this area is required to allay environmental fears.

37 Collins.

However, despite this apparent chemical stability, it should be noted that the presence of organic solvents could result in the leaching of large quantities of hydrocarbons from the rubber. Such a scenario could occur in the event of a RTI resulting in a fuel spillage. Again this is based upon findings using tyre shreds, in their raw state. As such it is recommended that additional research is required to examine CRMA's susceptibility to solvents.

5.3 Recycling CRMA

Limited evidence is available regarding the environmental, economic and engineering aspects of recycling CRMA. However, Takallou and Sainton, identify several demonstration projects conducted in Ontario, Canada, which addressed the ability to recycle CRMA. Their paper gives details of emission testing during the recycling process and indicates that the CRMA proved to be recyclable. Similarly, Emery concluded that there was no reliable evidence that asphalt pavements containing rubber could not be recycled to the same degree as conventional pavements.

However, whilst Takallou and Sainton addressed the environmental concerns of the recycling process, no indication was given as to the subsequent performance of the new pavement once trafficked. This latter area is of particular interest and must be studied in detail before specific conclusions may be drawn as to the overall ability to recycle the material. The rubber content will undergo a degree of degradation during the recycling process and studies are therefore required to determine the number of re-use cycles to which CRMA may be exposed before a deterioration in pavement performance is identified. Thereafter, consideration must be given to the subsequent disposal of CRMA once its useful life has expired.

6. Economic Analysis

There are several areas for consideration when assessing the short and long term economic viability of CRMA, including:

- Equipment. Specialist grinders or shredders required to produce crumb rubber, as well as any potential effect upon traditional plant and equipment utilised.

- Serviceability and performance. Serviceability and performance must be balanced against the whole life cost, including maintenance regimes and final materiel disposal costs.

- Political. Any political involvement will significantly affect success, this may include incentives to encourage reuse or recycling or legislation regarding the disposal of scrap tyres.

- Material costs. In the UK, material costs, traditionally simple to calculate, have become increasingly complex with the impact of the Aggregate Tax, Landfill Tax and even the proposed Incineration Tax, as well as fluctuations in the price of crude oil must also be evaluated. Additional factors include the pavements salvage value, the quantity of recycled material incorporated and the ease of recycling CRMA itself.

This paper will now briefly examine some of the key financial issues, which will ultimately determine the potential success, or financial viability of the use of CRMA, both as a disposal medium and future construction material.

6.1 Governments Financial Influence for Change

Howlett[38] identifies several tools available to government to bring about behavioural change in an attempt to better manage waste. Financial instruments are commonly used to penalise poor behaviour and reward desirable behaviour, amongst the more commonly implemented ones are resource and disposal levies as well as disposal bans[39]. The UK government has already gone someway towards driving behavioural change, through the use of the Aggregate Tax, the Landfill Tax and the Landfill Directive.

The UK Landfill Directive came into force in July 1999, prohibiting the land filling of whole tyres[40] in new landfill sites by July 2003 and shredded tyres by July 2006. For existing landfill sites there was potential flexibility for continuing to landfill tyres beyond 2006. An earlier example of government legislative support was the Intermodal Surface Transportation Efficiency Act of 1991, passed by the US Federal Government[41]. Section 1038 of the Act prescribed that, starting in 1994, 5% of the total tonnage of federally funded roads were to be CRMA[42], with the required percentage doubling every year until reaching a maximum of 20% in 1997.

However, this pertinent example of government intervention cannot be raised without noting that, despite making the roads quieter, tests in the early 1990s found that the roads were less durable than expected and as such the Federal Government did not pursue the policy. However, prior to abandoning the Act, considerable investment and technological development was made in the field of CRMA. As a result, the states of California, Arizona and Florida now utilise a large proportion of their scrap tyres in asphalt roads and the pavements have been found to be more durable than their traditional counter-parts, as well as being quieter[43].

Waste disposal levies, or landfill taxes, often seen as a financial tool, by which the government can address environmental matters, present conflicting issues:

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38 Howlett, Landfill Levy - Tax or Tactic?

39 A resource levy is a levy placed on raw materials to internalise the full costs associated with the extraction of non-renewable resources; provides an incentive to minimise waste in production, increase\material reuse or seek alternative materials.

40 For the purposes of the directive, the definition of tyres is taken to mean car and commercial vehicle tyres, which form the greatest element of total used tyre arisings. The Landfill Directive includes all tyres save bicycle and tyres above 1.4 metres outside diameter, thus motorcycle and other (small plant and equipment) tyres are also included.

41 House of Commons, Select Committee on Environment, Transport and Regional Affairs, Minutes of Evidence, 12 December 2000.

42 A mix was characterised as being CRMA if it contained at least 1% of crumb rubber by weight.

43 House of Commons, Select Committee on Environment, Transport and Regional Affairs, Minutes of Evidence, 12 December 2000.

- Assessing the environmental cost of waste is difficult.

- The free market does not appear to adequately charge waste generators for the full environmental impact of waste.

- The collection of a levy at the point of disposal significantly reduces the administrative burden. However, whilst relatively easy to collect at the point of disposal it may serve as a deterrent to legitimate disposal. Subsequent levy increases, to assist with the cost of removing illegally dumped waste, would only serve to exacerbate the situation.

- A disposal tax, at the point of purchase, encourages consideration of alternative products during the procurement process, although it increases the administrative burden. An administrative and financial link must be established between the producer and disposer, in order to ensure the correct distribution of monies. In comparison to the "collection at point of disposal" system there is an increased requirement for administrative and financial auditing due to the nature of the system and associated transfer of responsibility/monies attributable to the waste.

6.1.1 Aggregate Tax

On the 1st April 2002, the British Government implemented the Aggregate Tax, applicable to almost all stone leaving the quarry. In 2002 the headline rate was £1.60 per tonne (£2/T in 2009), the actual increase to the customer is far greater. Quarry by-products, and less popular sizes, such as scalpings, fine sand and dust, are sold cheaply and it is not economically viable to apply the full rate of tax to these products, as they often compete with exempt materials. Therefore, the more popular sizes have to bear a higher proportion of the tax. Thus, in 2002, the aggregate tax resulted in a mainstream aggregate price increase of between £2.00 & £2.75 per tonne[44].

The impact of the aggregate tax is reflected in the price of both traditional asphalt and CRMA. As component materials costs rises, the economic benefits afforded by a CRMA thickness reduction strategy become increasingly significant.

6.1.2 Landfill Tax and the Landfill Tax Credit Scheme

The Landfill Tax is often described as the UK's first environmental tax. Since its introduction, in 1996, the rate has steadily risen. In 2008, the tax was £32 per tonne for active wastes and £2.50 per tonne for inactive waste. There is clear indication that the Landfill Tax "escalator" is set to continue at £8 per tonne per year albeit the rate for inactive waste will remain extant.

The Landfill Tax Credit Scheme provides a convoluted and, it may be argued, ineffective attempt at funding sustainable waste management. In March 2001, The Environment, Transport and Regional Affairs Committee (ETRAC) in its report, Delivering Sustainable Waste Management recommended that rather than attempt reform of the existing system

44 Durward.

that it be replaced in its entirety. The implementation of a new system consisting of a fund, taking a percentage of the revenue from the Landfill Tax (and the proposed Incineration Tax) would allow monies to be directed towards minimising, waste generation and increasing the re-use and recycling of waste. The provision of such financial incentives for recycling and reuse of materials, including scrap tyres in CRMA, would greatly enhance their financial competitiveness in comparison with traditional materials.

6.1.3 Incineration Tax

Recent discussion[45] on the incineration of tyres has focussed upon their high calorific value, prompting debate as to whether tyre incineration is a means of waste disposal or merely an alternative fuel source.

Under the Koyoto agreement, the UK government has a commitment to reduce emissions of the greenhouse gas, carbon dioxide, by 25.6% of 1990 levels by 2010. The UTWG[46] identified that this could be achieved through the increased use of carbon neutral fuels such as tyres. In addition to burning whole tyres, a practice currently employed in areas such cement kilns, recyclers have identified a means to reform and refine tyres and oil, from scrap cars, into an oil which may then be burnt in existing power stations. However, the UK already burns too much old oil and fails to comply with the EU's Waste Oil Directive of 1975[47]. As such, increasing the number of tyres burnt for fuel would only compound this non-compliance.

6.2 Production and Placement Costs

6.2.1 Tyre Collection and Processing

UK companies operating in this sector effectively fall into two main groups. A handful of companies, operating on a national basis, handle over half of those tyres currently recovered. Tyre arisings are strategically brought together at a regional level, sorted as necessary and subsequently distributed through reprocessors. Providing a management service between tyre disposer and reprocessor these companies offer the standard of service required to facilitate production of a quality sensitive material such as CRMA.

The second group is more fragmented, operating at a local level. The task of economically pulling together widely dispersed, isolated, low level arisings is a significant one especially if these arisings are to be matched to a particular recovery facility.

The cost of collection alone, for a truck tyre, has been estimated as being £3.25 each. For reuse, rather than recycling, there are also additional costs associated with inspections, documentation and retreading etc. In July 2003, the cost associated with the disposal and recovery of used tyres was in the region of £100 - £140 per tonne of tyres[48]. This covers the retailer's costs, including the provision of storage, transport costs in getting used tyres to disposal and recovery facilities and the gate fee raised by such facilities for accepting used

45 NCE, A burning Issue.
46 UTWG 6 report
47 M Glaskin, Crunch Time for the Scrap Industry.
48 UTWG 6 report

tyres. Gate fees charged by disposal and recovery facilities vary considerably by process and location, and may be subject to market variations. Because of this fluidity, the paper does not seek to give an indicative spread of costs here. However, as a rule of thumb, landfill costs, for disposal or engineering, are significantly cheaper than the other options available.

6.2.2 Plant & Equipment Costs

From an equipment perspective it can be determined that there is generally no significant environmental or production modification involved and the method of crumb rubber addition will not add to the costs. Opinion on the impact upon plant production is varied but it is generally accepted that CRMA production would result in some loss. This is attributable to increased operating temperatures required for CRMA production and the requirement to prevent contamination of subsequent material. The temperature sensitivity of CRMA during compaction will also impact upon production, and thus cost, as the rate of production must be better tailored to the actual laying rate.

6.2.3 Life Cycle Cost Analysis

To evaluate alternative pavement materials, it is necessary to consider the total cost over its service life. The most effective method for measuring the cost-effectiveness of alternative designs is Life Cycle Cost Analysis[49] (LCCA), as shown schematically at Figure 10. The primary purpose of LCCA is to quantify the long-term economic implications of initial pavement decisions.

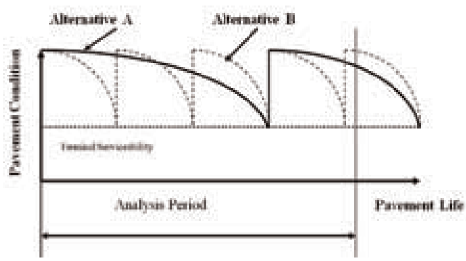

Figure 10. *The Principle of Life Cycle Cost Analysis for multiple pavements.*

A LCCA should include, where possible, all costs including initial construction, rehabilitation, and maintenance costs, incurred directly by the agency responsible for the pavement throughout the life of the pavement. Meanwhile consideration must also be given to the indirect costs user including travel time delay costs, vehicle operating costs, accident costs, as well as an assessment of discomfort costs both for road users and those living or working nearby.

Development of a comprehensive LCCA should include consideration of the following factors:

49 Emery

- Pavement recycling. Consideration must be given to the costs and practicality of pavement recycling and its salvage value. It has been identified that further research is required into the number of times that a CRMA pavement may be recycled whilst retaining the beneficial properties within the rubber component.

- Winter route maintenance. Any potential benefits in winter route maintenance relating to the thermal properties of CRMA or the reduction in ice formation due to pavement movement under transient vehicle loads should be examined. Secondary effects, resulting from a reduction in de-icing chemicals, may also include prolonging the service life of elements such as bridge bearings and road furniture.

- Road user costs. Road user costs are related to the efficiency of the road network. This must be examined at a variety of levels, from the pavements effect upon vehicle efficiency through to the impact of roadworks for pavement repair or refurbishment:

- An increase in service life will reduce the frequency of roadworks and thus reduce disturbance to vehicular traffic and impact upon the economy.

- Travel time costs are closely related to vehicle speed. Jong-Suk Jung[50] shows that prolonged CRMA life results in a uniform vehicle speed throughout the pavements life whilst the pavement speed for a traditional asphalt pavement slowly declines as pavement roughness increases. Overall Jong-Suk Jung identifies that CRMA pavements show substantial costs savings over a period of 25 years. Whilst there was no discernable difference in costs in the first 5 to 10 years, thereafter the higher maintenance costs for the conventional pavement began to make significant impact upon the overall analysis.

6.3. Noise Reduction

American research indicates that the potential cost savings from the use of open graded design incorporating rubberised asphalt, in the field of noise reduction at least, are in excess of 10 times the cost of constructing sound reduction structures[51]. Noise wall cost estimates, from Plastral, for February 2004 indicate that, depending upon material employed, the average cost of noise wall construction is as follows:

Assume 2.5m high noise wall, located on both sides of the carriageway.

Average noise wall cost per m^2	€200/m^2.
Average noise wall costs	€1000 /Lm of carriageway
Noise Wall Construction	£666 / Lm or £1,072,896 / mile.

It is readily apparent that significant cost savings are available through the utilisation of CRMA's noise reduction properties. It would be idealistic to believe that CRMA could prevent the construction of a noise wall structure altogether, especially when considering the

50 Jong-Suk Jung, Kaloush & Way, Life Cycle Cost Analysis: Conventional versus Asphalt-Rubber Pavements.
51 Rubber Pavements Association – Noise Reduction with Asphalt Rubber, 1999.

prolonged analysis period utilised. More realistically it may be assumed that the use of CRMA will reduce the existing noise levels. However, as traffic volumes increase, there will still be a requirement for such noise abatement measures. Utilising the figures above for noise wall construction costs, the potential economic benefits of delaying noise wall construction are shown graphically in Figure 11: assuming an interest rate of 4%, a delay of just 3 years in noise wall construction will result in present day savings of £33K. Higher interest rates or a greater postponement in construction serves to further increase the available savings.

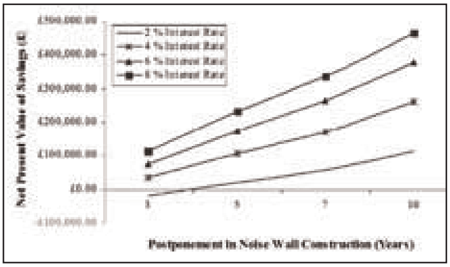

Figure11. *Cost savings through postponement of noise wall construction per £1M.*

Thus, irrespective of its other properties, considerable financial savings are available from the use of CRMA in circumstances that would otherwise require noise wall construction. Thus CRMA should be considered in sensitive areas such as outside schools, hospitals etc where the aesthetic or practical considerations of noise abatement structures renders them unsuitable.

6.4 Tyre Subsidies

The additional costs associated with the incorporation of scrap tyres into asphalt production may be simplified as follows:

Weight of passenger car tyre	= 9 Kg
Disposal, recovery & processing	= £ 100 / T
Number of passenger car tyres / T	= 111 Tyres/T
Additional cost per tyre	= Approx £1 /tyre

The author has been unable to establish exact figures for the cost of CRMA in the UK, however Colas [52] indicate that the cost of a CRMA mix is typically 25% more than that of conventional asphalt. Thus:

Assume cost of traditional asphalt	= £ 40/T
Cost escalation for utilisation of CRMA	= 25%
Cost of CRMA	=£ 50/T

52 Harding.

For the purposes of this paper it is assumed that CRMA utilises 3.6 scrap tyres per Tonne. Thus the £ 10/T cost escalation in the price of CRMA above traditional asphalt equates to £2.77 per tyre of which £1 is attributable to collection and processing.

If such a cost escalation were to be offset through the use of a disposal levy then it may be assumed that the administration of such a charge would itself incur additional costs, associated with the collection and distribution of such monies. For the purpose of this analysis it is assumed that this would equate to an increase of the equivalent price of CRMA from £2.77 to £3.00 per tyre.

The typical purchase price for an average car tyre (excluding high performance tyres etc) is £40.00. It is proposed that a disposal charge of £ 3.00 (7.5%), against such a purchase price, is not unreasonable for a product which is, for many, a relatively infrequent purchase. Such a levy, applied at the point of purchase off-sets the additional costs associated with the collection and processing of scrap tyres and encourages the use of retreaded tyres. Whilst the figure of £3.00 has been calculated in the context of CRMA, the monies raised, if such a scheme were to be introduced, would not be exclusive to the subsidisation of CRMA but utilised to subsidise any disposal route.

6.5 Economic Comparison

In the simplified assessment of costs given above there has been no consideration of the potential benefits from utilising CRMA. These benefits have significant impact upon any LCCA. In essence however there are three possible options:

6.5.1 Ignore the potential benefits of CRMA. In this scenario the CRMA is being utilised primarily as a scrap tyre disposal medium. It has been shown earlier that only a relatively small disposal levy(£3) is required per tyre in order to off-set the additional costs of CRMA in comparison with traditional asphalt. Such a levy negates any additional cost, assuming the subsequent performance of the two materials is unaltered.

6.5.2 Implement the CalTrans reduction strategy. The CalTrans reduction strategy allows a reduction in pavement thickness whilst utilising the same pavement service life as traditional asphalt. Whilst the unit cost (per CuM) of conventional asphalt is less than that for CRMA, the total cost is more due to the difference in the pavement thickness. Thus, in line with the Caltrans reduction principle, even in circumstances where CRMA is twice as expensive as conventional asphalt the construction costs would be the same. Whilst CRMA offers reduced cracking and increased durability.

6.5.3 Utilise the same pavement thickness as traditional asphalt. The CalTrans strategy utilises a reduction strategy to reduce the pavement thickness whilst returning a similar life expectancy as for a traditional pavement. It may be assumed that rather than utilise the beneficial properties of CRMA to reduce the pavement thickness, application of CRMA to the same depth as traditional asphalt would serve to prolong the pavements service life and reduce maintenance costs during this period.

7. Conclusions and Recommendations

The existing UK scrap tyre collection and re-processing framework requires consolidation and improvement to effectively capture 100% of the UK's tyres. Such a target will only be achieved through an economically viable commercial framework utilising financial incentives. The proposed solution is a disposal levy of approximately £3.00 per tyre. The alternative is to generate a demand for scrap tyres, through their inclusion in an alternative material such as CRMA. Without one or both of these it is suggested that there will be an increase in illegal disposal.

With strict quality control measures, CRMA offers significant improvements over traditional asphalt. However, there is a clear performance between Asphalt-Rubber and Rubberised Asphalt. The performance of Rubberised Asphalt has been found to extremely variable, it was this that led the various State transport departments within the US to concentrate their research and development upon Asphalt-Rubber.

Asphalt-Rubber returns similar friction values to traditional asphalt whilst showing an increased service life and a reduced maintenance requirement for the same pavement thickness asphalt. An alternative to increasing the pavements service life is to reduce the pavement thickness. The optimum rubber content was found to be in the region of 15-17% when utilising cracking data or rutting depth as a measure of performance in comparison to traditional asphalt.

The flexural properties of Asphalt-Rubber have been shown to increase safety through a reduction ice formation. Further research is required to support the, as yet anecdotal, claims regarding the influence of Carbon Black. Meanwhile Asphalt-Rubber pavements have been shown to lower accident frequency by reducing the water present upon the road surface. However the author would suggest that as a result of the higher vehicle speeds in the UK compared to the US where the information was obtained there is a requirement to ascertain the relationship between improved visibility, vehicle speed and the friction characteristics of CRMA.

Serious consideration must also be given to the environmental impact of CRMA. Research is required to support the author's assumptions that inclusion of crumb rubber within asphalt will serve to improve the material's stability. Additionally, detailed research is required to ascertain the degradation of the rubber in CRMA during the pavements service life and subsequent recycling process. In the event that degradation is identified, a means of extracting or recycling the rubber components must be identified to avoid the long-term contamination of the UK road network. As a scrap tyre disposal medium, CRMA must be compared to the spectrum of disposal methods as well as traditional asphalt. When examined against traditional asphalt, the increase in engineering performance offsets the increased unit cost of the material.

The utilisation of CRMA to reduce current noise levels and effectively delay noise wall construction offers the potential for significant financial savings. CRMA should also be considered over traditional asphalt for use in sensitive locations such as near hospitals, schools and residential areas.

The quality of Britain's roads is in decline, demand on the road network continues to rise and the issue of maintaining Britain's roads becomes increasingly important. In 2000, the Government published its 10 year transport plan containing commitments to halt the deterioration of local roads by 2004 and eliminate the maintenance backlog by 2010. In reality, every year since its inception in 1996, the ICE Annual Survey has shown an increase in the road maintenance backlog.

The use of CRMA offers the opportunity to increase pavement service life and reduce pavement maintenance costs: providing a disposal route for scrap tyres whilst potentially assisting to improve the current state of the nations road network. As early as May 1963, inspection of the A15, Yaxkley, UK, identified a 50% increase in pavement life compared with normal materials. The Road Research Laboratory (RRL)[53] concluded that there was sufficient evidence to indicate that, the resistance to reflection cracking was almost entirely due to the properties of the modified asphalt, which almost entirely eliminated cracking for the five years of observations. Despite these findings, after 45 years, CRMA is still not a widely accepted materiel in the UK construction industry.

It is recommended that support is required at a national level to develop guidelines, direction and technical support for the use of CRMA and other alternative materials within the construction industry. As such this requirement reiterate the findings of The Environment, Transport and Regional Affairs Committee in its 2001 report "Delivering Sustainable Waste Management" which stated that cited those in authority are:

"*guilty of thinking without imagination and planning without ambition, of finding problems instead of solutions and aiming for short-term goals without a vision of the system of resource use and waste management which we should be striving for*".

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Chapter 5

Function, Environemental and other Aspects

Noise Characteristics and Field Performance of Five Different Wearing Courses in Arizona

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ABSTRACT: In 1999, the Arizona Department of Transportation placed five different asphalt concrete pavement wearing courses as test sections on Interstate I-10, a highly trafficked Arizona highway. The wearing courses consisted of: Asphalt Rubber Open Graded Friction Course (AR-ACFC), Standard Open Graded Friction Course (ACFC), Polymer Modified Open Graded Friction Course (P-ACFC), Permeable European Mixture (PEM), and Stone Matrix Asphalt (SMA). A continuous review of these sections over the years has shown that the pavement sections experienced different amount of cracking and wear after eight years of service. Most importantly, AR-ACFC pavement sections experienced the least cracking and wear after eight years of service, while the other test sections showed considerable cracking and wear. On-Board Sound Intensity (OBSI) noise measurements were conducted in 2002 and again in 2008 using sound Intensity equipment. The results indicated that the least noise was observed for the AR-ACFC mixtures. In addition, 30 core samples from these test sections were obtained to evaluate their dampening (impedance) properties in the laboratory using an Ultrasonic Pulse Velocity (UPV) testing method. The results of these noise measurements and laboratory testing are discussed along with the degree of surface deterioration of each pavement test section in the field.

KEYWORDS: Asphalt rubber Open Graded Friction Course (AR-ACFC), Wearing Course, Ultrasonic Pulse Velocity Test, Ultrasonic Pulse Time, Effective Flow Resistivity, Air Voids, Tire / Pavement Noise, Pavement Surface Deterioration.

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1. Introduction

Globally, road traffic noise has become a major problem in several metropolitan areas. Automotive technology advancement has decreased the average noise per automobile by about six decibels (dB) in the last 25 years. One way to reduce highway noise affecting the adjacent residential areas is by constructing sound barriers (walls) along the city freeways. Typical noise barrier walls cost around 1.25 million US Dollars per kilometer (Gibbs *et al*, 2005; Road Noise, 2004). The public has brought up so much attention to the noise problem that transportation agencies, academic institutions and private entities are looking for innovative ways to address the problem. In Europe and the United States, several successful highway projects have been reported with the use of open graded or porous friction course pavements. The success has been mainly contributed to the high air voids content of these asphalt mixtures (Lerch *et al*, 2003). However, problems like clogging of pavement air voids were reported to possibly degrade the dampening acoustic effects of these porous pavements (Losa *et al*, 2003).

In Arizona, the Asphalt Rubber Asphalt Concrete Friction Course (AR-ACFC or sometimes referred to as ARFC for brevity) mixtures have become the number one public demand item. Residents have been asking their transportation officials to place asphalt rubber (AR) overlays on highways nearby their neighborhoods because of the great noise reduction they provide. In fact, legislation has been passed to overlay 185 kilometers of the freeways and highways in the Phoenix metropolitan area with AR mixtures because of the significant noise reduction produced by these mixes (MAG, 2005). In addition, research and field experience with these mixes have shown that AR mixes have great resistance to rutting, fatigue and thermal cracking (Way, 2003).

In this direction, Arizona Department of Transportation (ADOT) placed five asphalt concrete pavement wearing courses as test sections on a highly trafficked Arizona highway in 1999. A continuous review of these sections over the years has shown that the ARFC pavement sections experienced the least cracking and wear after eight years of service, while the other test sections showed considerable cracking and wear.

2. Objectives

The objective of this study was to evaluate the tire / pavement noise characteristics of five different pavement wearing courses placed as test sections in the State of Arizona, United States, by means of field noise measurements. In addition to the field noise measurements, 30 core samples from these test sections were obtained to evaluate their dampening (impedance) properties using an Ultrasonic Pulse Velocity (UPV) testing method in the laboratory.

3. Wearing Course Experiment

The Arizona Department of Transportation (ADOT) conducted a preventative maintenance pavement preservation experiment on Interstate – 10 (I-10) in Arizona during the year 1999 (Scofield, 2000). As part of this experiment, 32 test sections, with replicate cells, were constructed constituting five asphalt concrete pavement wearing courses types. The location of these test sections on the I-10 were randomly sequenced as shown in Figure 1. Figure

2 presents a layout of the I-10 test sections (starting at section number 99-0 and ending at section number 99-31). The sections are marked East bound from Milepost 186.2 to 195.3. The Annual Daily Traffic (ADT) is about 60,000 with 25% trucks. The total Equivalent Single Axle Load (ESALs) is about 26 Million. Table 1 shows the present pavement condition for each mile post (Zareh, 2008). The table includes recently measured pavement ride quality, friction numbers, rutting and percent cracking at each Mile Post. ADOT utilizes a MU meter or Side Force testing device to measure pavement surface friction. Measurements are reported as a skid number, that is, the measured value of friction times 100. In Arizona, the intervention level for friction reported for interstate, primary and secondary roadways is 34 (Mu Meter).

The five different pavement types under the study were as follows.

- Permeable European Mixture (PEM) as named per the project specifications
- Stone Matrix Asphalt (SMA)
- Asphalt Rubber Open Graded Friction Course (AR-ACFC)
- Polymer Modified Open Graded Friction Course (P-ACFC)
- ADOT's Standard Open Graded Friction Course (ACFC)

Table 2 provides the aggregate size gradation, asphalt cement content and type of Performance Grade used for the types of wearing courses used in the study. The highest asphalt cement content used was for the AR-ACFC mixture (~9.2%). The type of polymer used in the P-ACFC mix was either Styrene-Butadiene (SB) or Styrene-Butadiene-Styrene (SBS). The thickness of the wearing courses were 19 mm (¾") except for the Permeable European Mix whose surface thickness was 32 mm (1 ¼"). Each wearing course was supported by three structural layers beneath it. The pavement layers were designed for a 12 to 15 year service life.

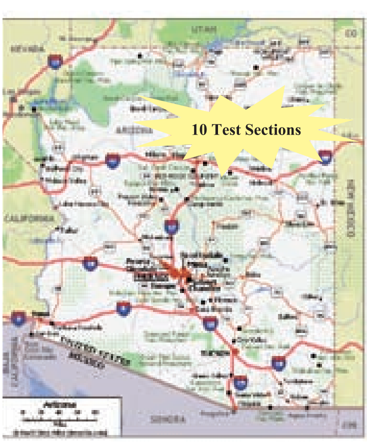

Figure 1. *Location of I-10 Test Sections, Arizona, United States*

Figure 2. *Schematic of the I-10 Test Sections, Arizona, United States*

Table 1. *Present Pavement Condition of I-10 Wearing Course Experiment Test Sections by Mile Post (Zareh, 2008)*

East Bound Mile Post No.	Mile Section	January 2008 Ride, IRI (m/km)	June 2007 Friction (Number)	January 2008 Rutting (mm)	March 2007 Cracking (%)
MP 186	186-187	1.09	50	3.8	3
MP 187	187-188	1.15	47	2.3	3
MP 188	188-189	1.95	51	5.6	6
MP 189	189-190	1.28	49	3.8	7
MP 190	190-191	0.96	51	3.3	2
MP 191	191-192	1.47	51	4.6	5
MP 192	192-193	1.39	54	4.3	5
MP 193	193-194	0.99	49	4.1	5
MP 194	194-195	1.01	46	5.6	4

Table 2. *Binder and Mixture Properties of I-10 Wearing Course Experiment Test Sections*

Size	AR-ACFC	ACFC	P-ACFC	PEM	SMA
¾	100	100	100	100	100
½	100	100	100	80-90	100
3/8	100	100	100	35-60	70-90
No. 4	30-45	35-55	35-55	10-25	30-50
No. 8	4-8	9-14	9-14	5-10	20-30
No. 200	0-2.5	0-2.5	0-2.5	0-2.5	8-13
Asphalt Content (%)	9.2	6.0	6.0	6.0	6.5
Performance Grade Binder	PG 76-22+	PG 64-16	PG 76-22+	PG 76-22+	PG 76-22+

4. Tire / Pavement Field Noise Measurements

In December of 2007, ADOT in conjunction with the Rubber Pavement Association (RPA) and Arizona State University (ASU) performed spot check highway noise measurements on the 32 test sections (Carlson et al, 2007; Biligiri, 2008). This 2007 noise measurements were not performed using On-Board Sound Intensity (OBSI) technique. Rather, a hand held noise meter was attached to the running board of a van, in such a way that the noise meter was in close proximity to the tire / pavement interface. Freeway speeds are typically above the cross-over speeds for all vehicle types. Above the cross-over speeds, tire pavement noise is the dominant traffic noise source, being much higher than the aerodynamic and the engine sources (Biligiri, 2008). This procedure was used to measure implicitly the tire pavement interaction noise. This was a similar technique that was used in Arizona in the early 1990's to measure the tire / pavement noise. The sound meter was calibrated to measure the sound intensity in the range of 80 to 130 decibels (dB), which is an appropriate range of noise measurements in the field. A computer was connected to the sound meter to store the data. Stored data was transferred to a PC via an RS-232 interface and analyzed using the system software. The total time taken for one full run on the 32 test sections depended on the speed of the test vehicle. Four runs were performed at three different speeds, namely, 100, 120 and 135 km/h (60, 72 and 75 mph) and the readings were recorded simultaneously.

4.1. Data Analysis

The sound levels were analyzed separately for each test section corresponding to each range of Mileposts (for a mix type). A typical plot of noise measurement at 100 km/h (60 mph) is shown in Figure 3. As seen from the figure, the least noise levels were observed for AR-ACFC test sections. This was true for all speeds, and it was observed that the tire / pavement noise levels were higher at higher test speeds.

For each mix type, the noise levels measured on each test section were grouped and averaged. Figure 4 shows the average values of tire / pavement noise levels for each mix type at the 100 km/h (60 mph) test speed. The least noise level was for the AR-ACFC mix and the highest noise was on the P-ACFC pavement type, with a difference of around 2.5 dB. In general, the noise level of each test section appeared to be related to the degree of surface deterioration. The AR-ACFC experienced the least cracking and wear after eight years of service with the other test sections showing considerable cracking and wear as illustrated in Figure 5.

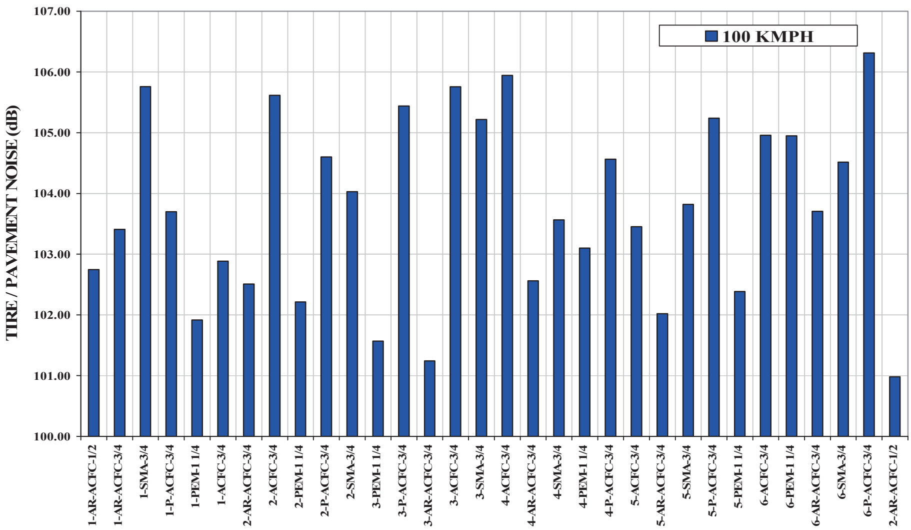

Figure 3. *Tire Pavement Noise Levels (dB) for All the Test Sections at 100 km/h (60 mph)*

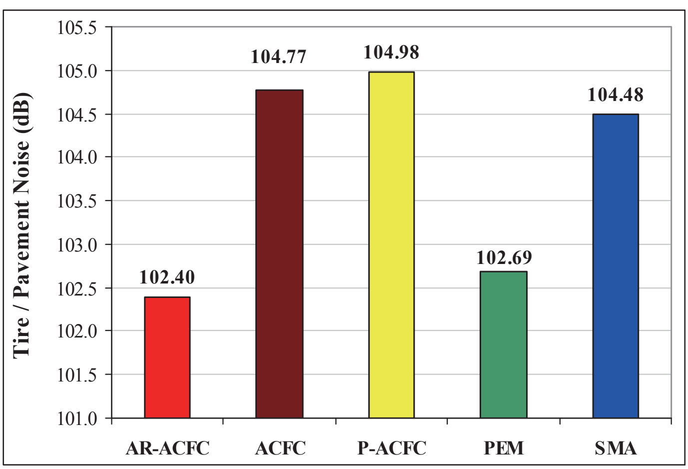

Figure 4. *Average Tire / Pavement Noise Levels for Each Mix Type at Test Speed of 100 km/h (60 mph)*

Figure 5. *Illustration of 2007 Pavement Surface Deterioration after Eight Years of Service*

4.2. OBSI Noise Measurements Comparison: 2002 and 2008

OBSI noise measurements were taken during the fall of 2002 by ADOT as part of the Arizona's Quiet Pavement Program. In addition, Dynatest Inc. measured noise levels on the I-10 sections using an OBSI technique during March 2008 at 100 km/h (60 mph). This independent set of Dynatest measurements were done as part of a larger California – Arizona highway noise study (Scofield, 2008; Kohler et al, 2007; CALTRANS, 2006; Scofield and Donovan, 2003). Figure 6 shows a comparison of noise readings between the years: fall 2002 and March 2008. In addition, the previously spot check measurements taken in December 2007 are shown on the same Figure 6 for comparative purposes. As can be observed, the data of the average tire / pavement noise values are very similar in trend for the 2007 and 2008 measurements. The least noise observed for all years and measurements is for AR-ACFC mixtures. This difference agrees with visual distress observations made in the fall of 2007. Several sections that exhibited higher noise have greater amount of raveling and cracking.

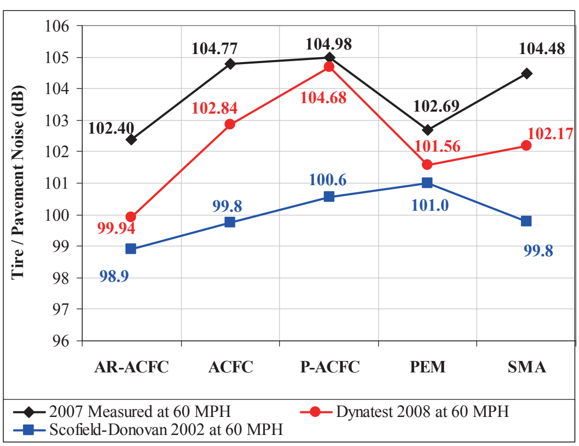

Figure 6. *Comparison of Average Tire / Pavement Noise (dB) for Arizona I-10 Test Sections, Years of 2002, 2007 and 2008*

5. Laboratory Evaluation of I-10 Field Cores – Dampening Properties

In addition to the field noise measurements, 30 core samples from I-10 test sections were obtained to evaluate their dampening (impedance) properties using an Ultrasonic Pulse Velocity (UPV) testing method in the laboratory. This section documents the UPV test methodology, experimental test results conducted on the field cores as well as developed predictive model to validate laboratory test results.

5.2. Non-Destructive Ultrasonic Pulse Velocity (UPV) Test

Non destructive evaluation techniques have been widely used in industry to measure the elastic or shear modulus of composite materials (Pellinen, 2001). The nondestructive UPV technique is based on the measurement of wave velocities through material as described in ASTM C597-02: Standard Test Method for Measuring Pulse Velocity through Concrete (ASTM E494-05; ASTM C597-02). The pulse velocity, v is related to the density and elastic properties of a solid by (Krautkämer and Krautkämer, 1990):

$$v = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}} \qquad [1]$$

where:
- v = velocity (in/s or m/s),
- ρ = density of material (lb/ft^3 or kg/m^3)
- E = modulus of elasticity (psi or kPa)
- μ= Poisson's ratio

In the laboratory, the pulse velocity is calculated as a ratio of the measured path length to the measured pulse time. That is,

$$v = \frac{L}{T} \qquad [2]$$

where:
- L = Distance between centers of transducer faces (in or m)
- T = Ultrasonic Pulse Time for transit (micro-sec or sec)

The Elastic Modulus is calculated as follows:

$$E = K\rho v^2 \qquad [3]$$

Where:
- E = Modulus of Elasticity (psi or kPa),
- K = a constant dependent on the Poisson's ratio and confinement,
- ρ = density of material (lb/ft^3 or kg/m^3)
- v = velocity (in/s or m/s)

5.3. Ultrasonic Pulse Time Transit Model

The ultrasonic pulse time transit model developed in this study was based on mathematical calculations programmed in an Excel™ spreadsheet (Biligiri, 2008). The input parameters included pavement volumetric properties such as asphalt cement, air voids; aggregate gradation (and type), and any admixtures such as crumb rubber inclusions. Portland cement concrete mixtures can be also analyzed in this spreadsheet by manipulation of input properties.

Mixture volumetrics such as Volume of air voids in the total mix, Va (%), Volume of rubber, Vr (%), Effective Volume of asphalt or bitumen, Vb (%) and Volume of aggregate solids, Vs (%) are calculated using input parameters. The surface areas of mix components are based of surface factors for each sieve obtained from NCAT Handbook (Roberts et al, 1996). The calculated mixture volumetrics as well as surface area of mix components are used to calculate Ultrasonic Pulse Time (UPT) of total mix for each pavement material components as follows:

$$\text{Total UPT (micro-sec or sec)} = T_i = t_{air} + t_{rubber} + t_{bitumen} + t_{aggregate} \quad [4]$$

The UPT addition of various pavement materials components is justified based on the four types of mixture properties present in the pavement material. The theory is based on the basic assumption that the mixture is a unit volume and the wave traverses through each component. The difficulty of measuring time through each component can be easily manipulated if one adds transit times of each pavement material component.

For a known pavement material thickness, L, Ultrasonic Pulse Velocity (UPV) can be calculated as follows.

$$\text{UPV (m/sec or in/sec)} = v = \frac{L}{UPT} \quad [5]$$

With the calculated value of the density from mixture volumetrics, one can calculate the value of Impedance as well as Effective Flow Resistivity, EFR in cgs Rayls as detailed next. The value of EFR can be directly input in the Federal Highway Administration Traffic Noise Model (FHWA TNM).

Ultrasonic pulse velocities for standard materials have been reported in the literature (Turner and Pretlove, 1991; Howe, 1998; Cha and Cho, 2007; Demirboga et al, 2004; Aracne-Ruddle et al, 1999; Mochinaga et al, 2006). For example, the ultrasonic velocity in rubber is about 2,115 m/s and for bitumen, it is about 1,800 m/s. As an initial calculation, the total transit time of two pavement materials: a conventional dense graded asphalt mixture and an asphalt rubber mixture that is known to dampen noise were analyzed using the model. The total transit time was calculated as a summation of transit times of different asphalt mixture's components. It was observed that the total transit time for the asphalt rubber friction course mixture was 1.35 times greater than the conventional mixture. This represents the dampening characteristic of the asphalt rubber mixture over the dense graded asphalt mixture. With the available transit velocity, one can calculate the impedance of the material, which is given by

$$Z = \rho * v \qquad [6]$$

Where:
Z = Impedance, N-sec/m^3,
ρ = density of the material (kg/m^3),
v= ultrasonic wave velocity through the material, m/sec

As explained previously, higher the transit time, lower the velocity and vice versa. FHWA TNM used by the United States to model traffic noise uses another form of sound absorption, the Effective Flow Resistivity, EFR which is basically the impedance of the material; also defined as the property of the material offering resistance to acoustic wave through an open pore (Menge et al, 1998; Sandberg and Ejsmont, 2002; Rochat et al, 2007). The TNM Technical Manual provides values of EFR in cgs Rayls for various ground types. Mathematically, EFR, σ is directly proportional to Impedance, Z. Both are expressed in the same units, N-sec/m (MKS units) or cgs Rayls (cgs units). It is noteworthy that

$$\text{EFR in cgs Rayls} = 0.1 * \text{EFR in N-sec/m}^3 \qquad [7]$$

Typical values of EFR for different pavement materials have been measured and reported by Rochat et al, which is shown in Table 3 (Rochat and Hastings, 2008).

Table 3. *Typical Effective Flow Resistivity Values for Different Types of Pavements (Rochat and Hastings, 2008)*

Pavement Type	EFR [cgs Rayls]
Portland Cement Concrete (PCC)	20,000
Old Dense Graded Asphalt Concrete (DGAC)	14,500
New Bonded Wearing Course (BWC), 30 mm	12,300
Asphalt Rubber Friction Course (ARFC or RAC-O)	6,000 - 6,100
Open Graded Asphalt Concrete (OGAC)	2,000 - 4,200

5.4. Experimental Program and Test Results

Cores were sampled along the right travel lane from five different pavement types on the I-10 test sections. Samples were cored with a diameter of 4 inches (100 mm) and to a depth of anywhere between 7.5 and 10.5 inches (~187-262 mm) that also included an inlay of dense graded asphalt concrete. Cores were brought to ASU laboratories and surface of each of the cores were sawed to obtain regular five different wearing course pavement materials. Thus, thickness of five different course materials varied anywhere between 0.5 and 1.5 inches (~12.5-37.5 mm) depending on the pavement type.

Air voids of samples were estimated using a CoreLok device. After the air voids of samples were determined, acoustic dampening properties were determined using ultrasonic pulse velocity test as described in the previous sections.

Ultrasonic pulse velocity test methodology was conducted using 20 kHz transducers on 30 field cores. Honey was used as couplant between sample and transducer interface. UPT was noted for the samples under investigation. The measured length of the sample was used to calculate UPV and hence, Impedance and EFR for all pavement material samples. Table 4 summarizes the ultrasonic pulse velocity test results for the 30 field cores samples.

5.5. Model Prediction versus Laboratory Measurements

Both model predictions and actual test results on I-10 field cores were compared and the results are shown in Figure 7. As observed in the figure, there is a good correlation between the model predictions and laboratory measured Ultrasonic Pulse Time. It is noted that the model under-predicts high UPT values; one reason for the under-prediction is attributed to the input parameters used in the model, such as the use of literature reported ultrasonic wave velocity for rubber particles and aggregates. Nevertheless, there is a rational correlation between both the predicted and measured values.

5.6. EFR Sensitivity to Air Voids and Asphalt Content – I-10 Field Cores

The developed model was also used to understand the impact of material's volumetric change on EFR. An example showing changes in mixtures' air voids is shown in Figure 8. With an increase in air voids, there is a decrease in EFR which means that there is a positive effect of air voids increase on noise dampening. This is rational and agrees with experts' opinion on the role of the air voids in the mix's noise characteristics.

Figure 9 shows a change in EFR for the different mixtures with respect to asphalt content. The figure shows both predicted and measured EFR values for all the mixtures. It can be seen that the predicted EFR values are higher for mixtures with lower asphalt content while the least EFR was observed for the mixture with highest asphalt content (~9.2%). Note that the lowest calculated EFR values were for asphalt rubber mixtures that also possessed highest air voids levels.

Table 4. *Ultrasonic Pulse Velocity Test Results for I-10 Field Cores*

Mixture Type	Average Air Voids (%)	Height (mm)	Measured UPT (sec)	Calculated Ultrasonic Pulse Velocity (m/sec)	Density (kg/m^3)	Calculated Z, Impedance ($N\text{-}sec/m^3$ or MKS Rayls)	Calculated Z, Impedance (cgs Rayls)	Average Measured Z, Impedance (cgs Rayls)
PEM	17.14	26.4	8.20E-06	3,165	2.0785	64,460	6,446	6,738
		33.1	1.02E-05	3,191	2.0924	65,437	6,544	
		33.2	1.00E-05	3,267	2.0556	65,812	6,581	
		31.1	8.60E-06	3,563	2.1445	74,878	7,488	
		22.1	7.30E-06	2,982	2.0386	59,578	5,958	
		28.8	7.60E-06	3,728	2.0287	74,111	7,411	
P-ACFC	20.96	37.5	1.17E-05	3,154	2.0712	64,015	6,401	5,891
		34.7	1.05E-05	3,250	2.0789	66,224	6,622	
		17.4	6.50E-06	2,628	1.9631	50,551	5,055	
		28.7	9.47E-06	2,984	1.9803	57,914	5,791	
		20.8	6.40E-06	3,194	1.9951	62,444	6,244	
		28.4	1.03E-05	2,716	1.9666	52,336	5,234	
AR-ACFC	17.17	13.5	5.70E-06	2,326	2.0959	47,783	4,778	5,754
		19.2	6.80E-06	2,774	2.0784	56,491	5,649	
		17.2	5.90E-06	2,863	2.1076	59,129	5,913	
		24.7	8.60E-06	2,824	2.0846	57,700	5,770	
		20.4	7.30E-06	2,752	2.1422	57,775	5,778	
		18.3	5.60E-06	3,214	2.1064	66,352	6,635	
ACFC	13.80	19.2	6.00E-06	3,145	2.2768	70,172	7,017	7,339
		23.5	6.00E-06	3,855	2.2100	83,492	8,349	
		23.9	5.60E-06	4,209	2.3652	97,557	9,756	
		25.8	7.00E-06	3,624	2.2971	81,587	8,159	
		22.7	8.17E-06	2,739	2.0293	54,474	5,447	
		20.0	7.10E-06	2,773	1.9529	53,077	5,308	
SMA	9.65	26.6	6.60E-06	3,974	2.0796	80,996	8,100	7,452
		23.6	6.70E-06	3,472	2.1581	73,424	7,342	
		24.6	6.07E-06	3,991	2.1905	85,667	8,567	
		18.3	6.97E-06	2,581	2.1738	54,980	5,498	
		31.1	8.10E-06	3,784	2.0997	77,863	7,786	
		31.0	8.50E-06	3,593	2.1063	74,163	7,416	

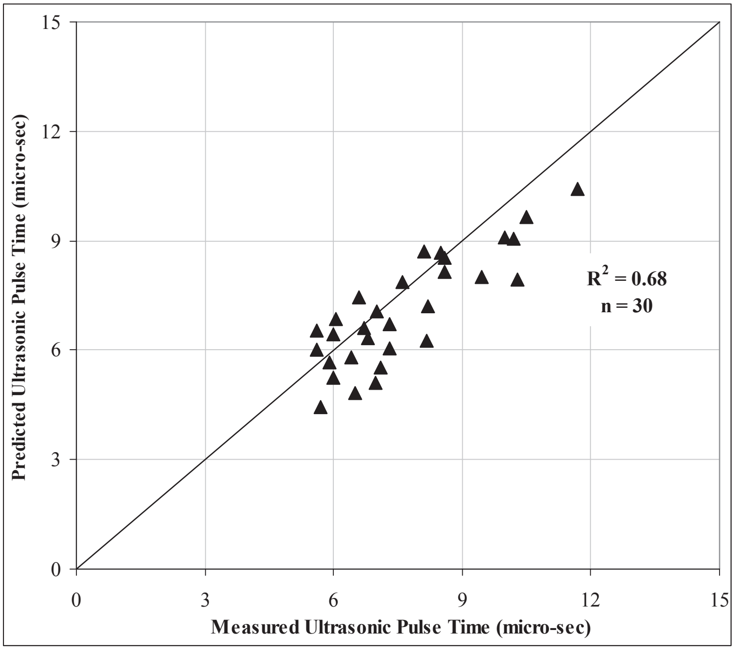

Figure 7. *Comparisons between Predicted and Measured Ultrasonic Pulse Time (micro-sec) for I-10 Field Cores*

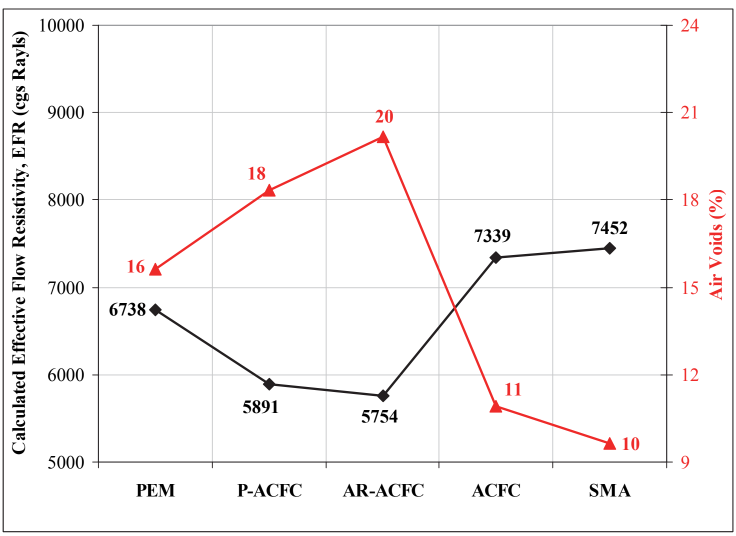

Figure 8. *Sensitivity of EFR (cgs Rayls) to Varying Approximate Air Voids Levels – I-10 test sections*

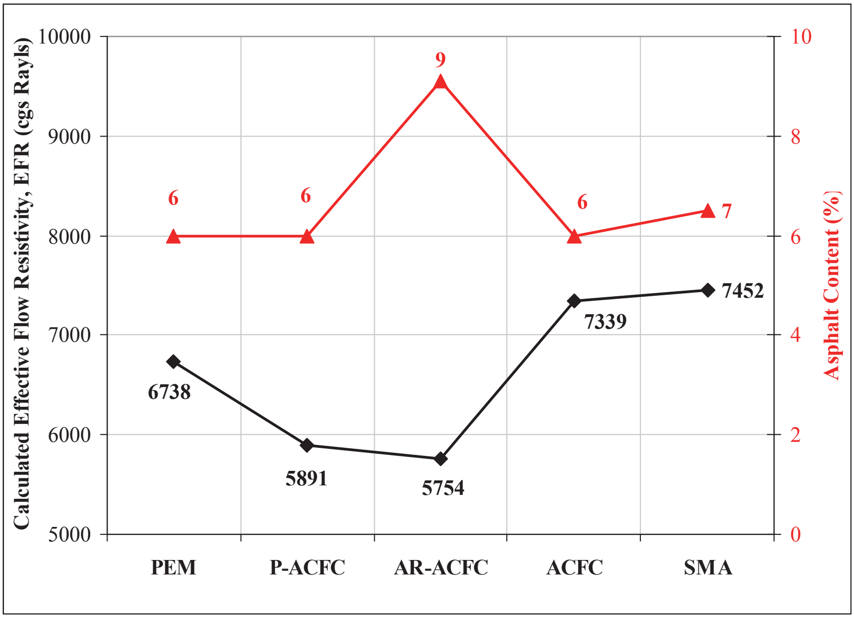

Figure 9. *Sensitivity of EFR (cgs Rayls) to Varying Asphalt Content – I-10 test sections*

6. Field Measured Tire / Pavement Noise versus Laboratory Calculated EFR

Figure 10 shows a relationship between field measured tire / pavement noise and calculated EFR (estimated from laboratory UPT measurements) for I-10 test sections. Field noise measurements shown in the figure are as measured by Dynatest Inc. on the I-10 sections using an OBSI technique during March 2008 at 100 km/h (60 mph). As mentioned previously, this independent set of Dynatest measurements were done as part of a larger California – Arizona highway noise study.

As observed, field noise was lower for mixtures with lower EFR except for P-ACFC mixture. It must be noted that the field cores were randomly sampled from just five sections out of the total 32 sections. Again, the sampling might not have represented exactly the five different pavement types. Additionally, P-ACFC field cores mixtures had higher air voids compared to the other materials and hence, calculated EFR were low.

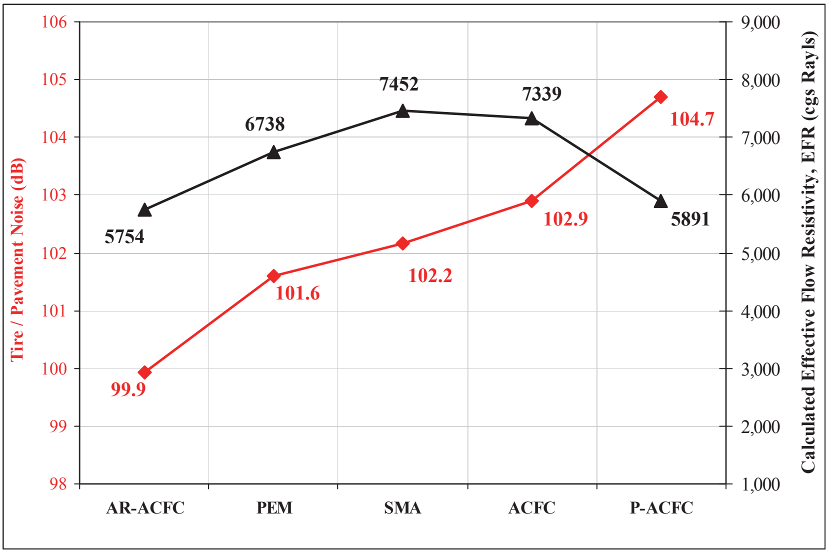

Figure 10. *Relationship between Tire / Pavement Noise and Calculated Effective Flow Resistivity for All Mixtures – I10 Test Sections*

This apart, as mentioned previously, several sections that exhibited higher noise had greater amount of raveling and cracking. This was particularly observed for P-ACFC mixtures. Hence, the relationship between field measured noise and estimated EFR were not direct. Nevertheless, four other sections showed excellent correlation between field noise and estimated EFR. Overall, AR-ACFC mixes had the lowest recorded field noise in addition to the lowest laboratory calculated EFR. This evidently proves that AR-ACFC mixtures had the highest potential for noise dampening; also, the surface deterioration on these sections was the least amongst all the wearing courses.

7. Summary and Conclusions

The purpose of this study was to conduct field noise measurements on five different pavement wearing courses that were placed as test sections on a highly trafficked Interstate in Arizona, United States. The pavement sections were placed in the year 1999 and included: Permeable European Mixture (PEM), Stone Matrix Asphalt (SMA), Asphalt Rubber Open Graded Friction Course (AR-ACFC), Polymer Modified Open Graded Friction Course (P-ACFC), ADOT's Standard Open Graded Friction Course (ACFC). Noise measurements were performed using a vehicle similar to On-Board Sound Intensity (OBSI) technique.

Noise data were collected in 2002, 2007 and 2008. The least noise observed for all years and measurements was for the AR-ACFC mixtures. In general, the noise level of each test section appeared related to the degree of surface deterioration. The AR-ACFC experienced the least cracking and wear after eight years of service with the other test sections showing considerable cracking and wear.

In addition, 30 core samples from these test sections were obtained to evaluate their dampening properties using an Ultrasonic Pulse Velocity (UPV) testing method in the laboratory. The results of these noise measurements and laboratory testing were discussed along with the degree of surface deterioration of each pavement test section in the field. Correlations between predicted and laboratory measured UPT for all the pavement materials under investigation were performed. The correlation between the model prediction and laboratory measured UPT was good. The developed model was also used to understand changes in materials volumetric on UPT and EFR, such as changes in air voids and asphalt content. Relationships between field measured noise and calculated EFR (estimated from laboratory measured UPT) for each mixture type were established. Overall, AR-ACFC mixes had the lowest recorded field noise in addition to the lowest laboratory calculated EFR.

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Asphalt-Rubber Open Graded Friction Course Reduces Noise-The Quiet Pavement Program

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ABSTRACT: The Arizona Department of Transportation recognized a need to provide the citizens of Maricopa County living next to its freeways with a less noisy quieter freeway. The Maricopa County freeway system is referred to as the MAG freeway system and it is composed of over 1200 lane miles (2000 lane kilometers) of multi-lane concrete pavement. Noise is defined as a loud sound of any sort that is disagreeable or unwanted and the people living next to the MAG freeway highways complained that the freeways were too noisy. Throughout the ages of civilized history, noise has been an annoying irritant to mankind. In recent years the noise level of freeways in urban and suburban areas in the United States and the MAG freeway system in particular has increased perceptibly as freeway traffic has increased and more people are living closer to freeways. To reduce such a noisy irritant generated by traffic on concrete or asphalt pavements Arizona has employed a 25 mm open graded friction course asphalt-rubber hot mix surface course. This paper reviews the experience in Arizona and research in California as well, with using asphalt-rubber open graded friction course pavements to reduce the noise by 3-12 decibels. These findings have led to Arizona developing a large scale program of covering over 1200 lane miles (2000 lane kilometers) of relatively new and new concrete with a 25 mm (1 in.) asphalt-rubber open graded friction hot mix surface course to substantially reduce noise. This program is referred to in Arizona as the Quiet Pavements Program and has been very successful.

KEYWORDS: asphalt rubber, open graded friction course, noise, measurement, performance
Asphalt Rubber 2009

1. Foreword

This paper is a continuation of a paper presented at the AR2006 Conference (Ali, 2006). Since 2006 the Arizona Department of Transportation (ADOT) in cooperation with the Maricopa Association of Governments (MAG) has continued to build and expand the MAG freeway system which is composed of a concrete pavement covered with a 25 mm (1 in.) asphalt-rubber (AR) open graded friction course (ARFC) to reduce noise. Typically the placement of ARFC reduces the noise by 4 dBA. This study reports on the ten year Quiet Pavement Program Pilot Research Study being conducted in cooperation with Federal Highway Administration (FHWA) for purposes of determining how long the reduction of noise associated with the ARFC placement will continue.

2. Background on Noise and Noise Measurement

Noise is defined as a loud sound of any sort that is disagreeable or unwanted. Through the ages of civilized history noise has been an annoying irritant to mankind. The Roman Emperor Caesar decreed in 20 BC that carts could only move through the city of Rome during the night, since their noisy din during the day interfered with the daily business of Rome. Carts moving through Rome at night were acceptable since Caesar and his court lived in the mountains far from the noise of the carts in the city. As the centuries passed by mankind has tried to live in a peaceful world. Horse drawn wagons and coaches with wooden wheels and iron rims generated plenty of noise in the 1800’s. In the late 1800’s in England road builders used wood blocks and even rubber blocks to deaden the wagon wheel noise. In the 1900’s with the advent of automobiles and rubber pneumatic tires it appeared the tire/pavement noise had finally ended for all time. However, as more automobiles and trucks took to the highways, freeway noise gradually crept back into the city.

To measure the tire pavement noise several measurement instruments were developed. The noise measurement instrument, which is most commonly used, is an electro-acoustical device with a microphone that converts sound pressure (a scale value) into an electronic or voltage signal, or vice versa. The instrument has a variety of names such as noise meter, sound meter, or sound level meter. When placed near a sound source, it will display or readout a single number of the corresponding decibel level in dB or dBA. The dB level refers to the sound or noise level in general, whereas the dBA value is a sound or noise measurement adjusted to be level to the sensitivity of the human level of hearing. The value of sound or noise is a function of frequency typically it is obtained by averaging the sound pressure over a pre-set frequency band, typically, from a frequency of about 100 Hz to as high as 20,000 Hz.

There are several noise measurement methods for measuring the traffic related level of noise. The most common method is the wayside method which generally is conducted by measuring the noise with a typical noise meter placed approximately 15.2 m (50 feet) from the center of the roadway to be tested and a typical height of 1.5 m (5 feet), Figure 1.

Figure 1. Wayside method of measuring noise

Recently another mobile trailer form of measuring tire/pavement interface noise has been developed. This method is called the Close Proximity method or CPX method, (a.k.a. "The trailer method"). Figure 2 presents a picture of the trailer. The trailer method (CPX) was used to accomplish the much of the work described in this report. The test tire is mounted in a trailer, which is towed by a towing vehicle. Close to the test tire, generally within 0.1-0.5 m (4 to 18 in), one or more micro- phones are located. The noise level is measured as an average over a certain time interval, usually 4-60 seconds. Most trailers have an enclosure around the microphone and test tire in order to provide screening from wind and traffic noise. Such enclosures are lined on the inside with sound absorbing material. Some trailers may utilize more than one test tire. The method may also utilize especially designed self-powered vehicles that are not of the trailer type. This method is less sensitive to noise generated by other traffic. This equipment is essentially designed for comparing road surfaces.

Figure 2. *CPX trailer method of measuring noise*

The newest form of measuring the tire pavement interface noise is the noise intensity method, now referred to as the On Board Noise Intensity (OBSI) method of tire pavement noise measurement. The American Association of State Highway and Transportation Officials (AASHTO) and the American Society of Testing Materials (ASTM) are both developing standard OBSI noise measurement methods. The sound intensity measuring hardware consists of a probe (microphone pair) held next to the tire/pavement contact patch by a fixture attached to the wheel studs of the test tire/wheel. The microphone is cabled to the interior of the vehicle where the signals are simultaneously captured on a recorder and processed by a real time-analyzer. The specially tuned microphone only picks up the noise of the tire pavement interface and no other noises from wind or other vehicles or any other sound. Figure 3 shows a typical installation.

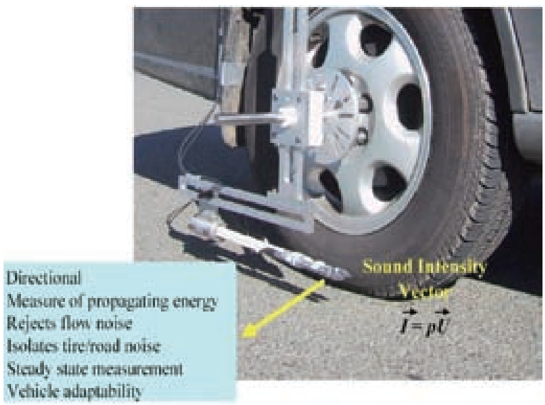

Figure 3. *Noise intensity method of measuring noise*

3. Asphalt-rubber Open Graded Surface Course

To reduce such a noisy irritant generated by traffic on concrete or asphalt pavements both California and Arizona have employed a 25 mm (1 in.) open graded AR hot mix surface course. Asphalt-rubber is a mixture of 80 percent hot paving grade asphalt with 20 percent ground tire rubber produced from waste tires Way (2001). The resultant asphalt-rubber binder mixture is added hot to a hot open graded mineral aggregate to produce an asphalt rubber friction course as the final wearing course of the pavement structure. This paper reviews the experience in both California and Arizona with using asphalt-rubber hot mix pavements to reduce the noise by 3-12 decibels. Research has been conducted in both states to determine the nature of the noise and how best to measure it and to compare the results of such tests to various pavement surfaces to reduce the noise to an acceptable level. Findings from this research include the following observations, namely that roadside measurements have shown that open-graded asphalt rubber friction courses (ARFC) can achieve 3 to 5 dB noise level reduction when compared to traditional asphalt dense graded surfaces and 6 to 12 dB noise level reduction when compared to concrete surfaces. In addition sound intensity measurements (measurements taken close to the tire/pavement interface) have shown that open-graded asphalt rubber friction courses (ARFC) are effective in reducing noise by 4 to 6 dB compared with traditional dense graded asphalt concrete and by 6 to 12 dB when compared to concrete surfaces. These findings have led to Arizona developing a large scale program of covering over 1500 lane kilometers of relatively new concrete with ARFC to substantially reduce noise. This program is referred to as the Quiet Pavements program and has been very successful.

4. Arizona Quiet Pavements Program

Phoenix, Arizona and its surrounding suburban sister cities have experienced a tremendous growth in population in the last 50 years. Phoenix is one of fastest growing cities in all of the country and now is the fifth largest city in population in the United States. With growth in population has come the growth in automobile traffic and the need for more freeways in the Phoenix Metropolitan area. In 1985 the citizens of Maricopa County, which includes Phoenix and numerous sister cities, voted in favor of a 0.5 cents sales tax to fund the design and construction of over 115 miles of new freeways. Since the funding only addressed design and construction of the freeways and not maintenance or rehabilitation it was decided that the freeways would be built with concrete pavements.

Starting in 1986 construction of the freeways began and with time more miles were completed and more people bought homes built near the freeway, and they began to notice the annoying noise of the freeways. Even though sound walls were constructed to mitigate the noise, complaints about noise continued. In the year 2000 the Arizona Department of Transportation (ADOT) began construction of the widening of 17 km (10 miles) of Superstition Freeway which is in the Phoenix Metropolitan area. The freeway construction included widening the concrete pavement from three lanes to six lanes in each direction to accommodate the over 150,000 vehicles a day that use the freeway, Figure 4. As part of this major construction ADOT decided to overlay all the lanes full width with an asphalt rubber open graded friction course (ARFC). The ARFC surface was selected to provide a new surface with a smooth ride, good skid resistance, a new surface that could be plainly re-striped and to reduce the tire/pavement noise.

Figure 4. *Superstition Freeway in 2003 with ARFC surface (Courtesy of Arizona DOT)*

As construction drew to a close and the placement of the ARFC progressed to its ultimate completion drivers, passengers and people living next to the freeway began to notice the new ARFC riding surface was very quiet. Unexpectedly, people began to write the local newspapers and call local radio talk shows in praise of the new quiet riding surface. Although the quiet riding surface was an ARFC it soon began to be called by the local media and people in general simply rubberized asphalt. From this grassroots reaction to peace in the valley, that is the Phoenix Metropolitan area is called the Valley of the Sun, action groups sprang up to petition government to cover all the freeway miles with rubberized asphalt. The local governments as well as state government heard the voice of the people and developed a Quiet Pavements Program in December 2002, (Carlson, 2003) and (Scofield, 2003). Then Governor Jane D. Hull and ADOT Director Victor Mendez informed the public that over the next three to four years ADOT in cooperation the Maricopa Association of Governments (MAG) would overlay the concrete freeway system with rubberized asphalt. The cost of this Program was estimated to be 34 million dollars ($3.50/Sq. meter / $2.90 Sq. yd.) and it was considered a Quality of Life issue for the people of the Phoenix Metropolitan area.

In addition ADOT would work with the Federal Highway Administration (FHWA) to collect before and after the ARFC overlay noise measurements. This research effort would be used to determine whether the pavement surface noise reducing properties of ARFC would remain at a beneficial level over time. If results of the study are satisfactory, they could be mathematically modeled and become part of the FHWA National Noise Model. Presently, the FHWA noise model does not allow the surface noise characteristics to be used as an input into the model. Practically speaking, the national noise model only allows walls or berms to be used as a means of reducing noise. However, asphalt-rubber open graded mixes have been shown to reduce noise to a noticeable degree, Figure 5.

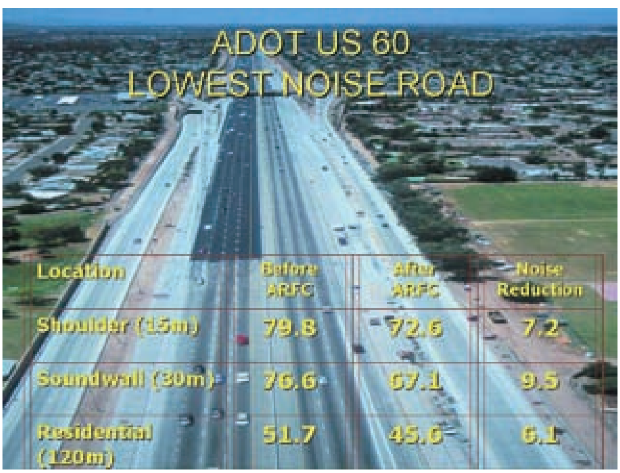

Figure 5. *Superstition Freeway 2002 noise reduction due to asphalt-rubber open graded mix, courtesy Rubber Pavements Association*

5. Noise Synthesis Study

The significant reduction of the tire/pavement noise due to the use of an ARFC and the positive way in which people responded to this triggered the Rubber Pavements Association (RPA) to sponsor a Noise Synthesis Study (Sousa et al., 2004). The purpose of the study is to acquaint people with what tire/pavement noise is and how an ARFC reduces the noise and to what degree it reduces the noise. Tire/pavement noise has been known about for many years. This report contains a summary of 47 studies based on research and data from the US, France, Portugal, Spain, Sweden, Australia, New Zealand and Japan.

Early studies of noise focused mostly on concrete pavements and some asphalt pavements were used as the basis for developing the FHWA noise policy in 1976. Although much research has continued into tire/pavement research since 1976 the FHWA Policy has remained virtually the same over all these years. Starting in the 1990's both California and Arizona began to investigate tire/pavement noise and to begin documenting their findings from actual field measurements. These early projects clearly showed that an ARFC surface measurably reduces noise. What these analytical studies did not show is to what degree people would positively react to reducing freeway tire/pavement noise and how passionate they are about this being a quality of life issue.

Noise is measured in decibels and the larger the decibel values the more irritating the noise. Decibels are commonly abbreviated as dB or dBA. The dB abbreviation refers the level of measured noise, whereas the dBA abbreviation refers to not only the measured value but how people respond to the noise. Normal conversation in an office setting is typically about 40-50 dBA. The noise of a lawnmower is generally about 70-100 dBA depending upon how close you are to it and of a diesel truck can be as much as 90 or more dBA. With regard to highways the FHWA National Noise Policy sets an acceptable level of noise at 67 dBA at a distance of 50 feet away from the centerline of the highway. By contrast the Arizona DOT sets its acceptable level of highway noise at 64 dBA also 50 feet from the centerline of the highway. Figure 6 shows common source of noise and the associated noise level.

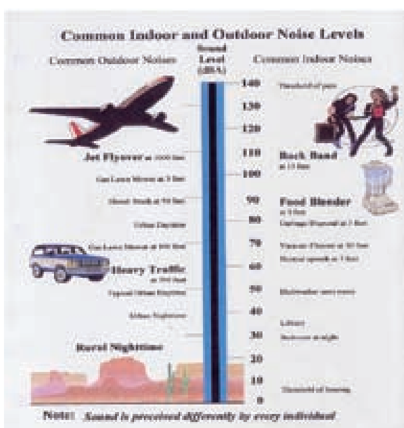

Figure 6. *Noise levels from quiet to very loud (Courtesy of Arizona DOT)*

Highway noise is primarily generated from three point sources, the exhaust or tailpipe noise, the engine noise and the tire/pavement noise. In addition highway noise is also a function of traffic, more cars and trucks, more noise. Automotive engineers have very successfully addressed both tailpipe noise and engine noise by muffling or dampening at the point source. The tire/pavement noise has remained as a somewhat uncontrolled noise source, primarily because the greatest emphasis has been placed on tire/pavement wet weather friction, rather than noise.

Each of the three sources of noise contributes to the total noise. Typically when cars and trucks reach a speed greater than about 60 km/hour (35 miles/hour) the tire/pavement noise begins to become the dominant source of noise. At normal freeway speeds of 90 km/hour (55 miles/hour) or greater the tire/pavement interface noise can represent at least 70 percent of the total noise as shown in Figure 7. Significantly reducing the tire/pavement point source of noise obviously can reduce the overall noise significantly as well.

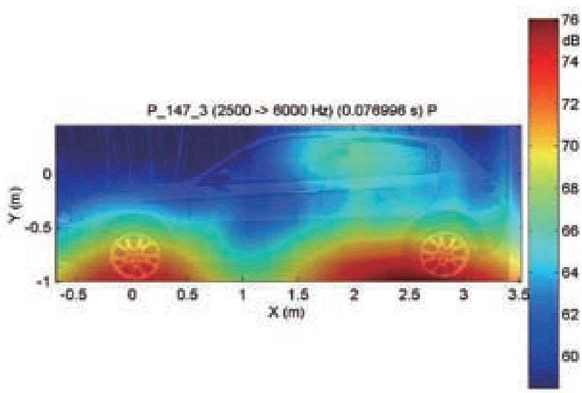

Figure 7. *Noise The source of most highway noise is generated at the tire/pavement interface (Courtesy of ACB Engineering)*

What creates the tire/pavement noise is a subject of much conjecture and theory. The most commonly held theory is that as the tire passes over the pavement a change in pressure occurs at the tire/pavement interface and this pressure change generates the noise. Another theory holds that the friction or rubbing of the tire against the pavement creates a noise from the rubbing action. Also it has been observed that rolling a wheel across a concrete surface creates more noise than rolling the same wheel across a carpet or rubber mat and thus the softness or stiffness of the pavement itself may amplify or muffle the noise.

Reviewing the three theories about what creates the tire/pavement noise it is possible to postulate why an ARFC would provide a very quiet ride. Empirical measurements demonstrate that as the concrete grooves or air pockets are cut or positioned differently in relation to the tire more or less noise develops. Likewise an asphalt open grade friction course surface has a tremendous amount of air pockets which can dampen the pressure change gradient, thus reducing the noise. An ARFC also has a huge amount of air pockets or air voids, which can contribute to less of a pressure change. The ARFC surface is a much smoother riding surface then the concrete. The ride is smoother because the ARFC mix is placed in a continuous manner with minimal joints and the aggregate top size is 9.5 mm (3/8 in.). Such a smooth riding surface and small top size aggregate could contribute to less tire deformation with travel and less squeezing of air between the tire and pavement (less pressure change) and thus less noise.

In addition the ARFC is a rubber like soft surface much like a carpet or rubber mat and this too could reduce noise. The softness of the ARFC comes from a much higher asphalt binder content and crumb rubber content (20 percent of the binder). Typical open graded friction course mixes have about six percent asphalt, whereas an ARFC has 9 to 10 percent asphalt rubber binder. Rubber is commonly used to reduce noise, thus there may be reason to believe that rubber particles in the ARFC may very well contribute to less noise. All of these material related aspects of an ARFC surface would tend to reduce noise and most likely collectively do contribute to less noise. Thus there is good reason to believe that the empirical evidence of less noise with an ARFC surface is related to a yet not fully understood scientific mechanism.

6. Tire/Pavement Interface Noise Compared to Wayside Measurements

Although the scientific mechanism of tire/pavement noise phenomenon may not be altogether clear, clearly some surfaces are less noisy than others. Concrete freeway surfaces can be textured or ground to give different levels of noise. All the studies appear to agree that transverse tined concrete is the noisiest surface. Longitudinally textured concrete is somewhat less noisy and a very specially diamond ground surface referred to by the industry as whisper concrete, is the least noisy of any of the concrete surface textures. An ARFC surface is even quieter that any of the concrete textures as seen in the Table 1. In this Table developed by the Arizona DOT on concrete surfaces and ARFC pavements tested on the Phoenix Metropolitan Freeway system it is clear that the ARFC tire/pavement noise point source level is quietest as shown in the CPX column. The CPX column show actual close proximity noise measurements taken very close to the tire/pavement interface. The 15 m (50 feet) from centerline noise values are estimated from research conducted by the California Department of Transportation that relates a tire/pavement point source measurement to the standard 15 m (50 feet) from pavement centerline measurement which is used in the FHWA National Noise Policy. The ARFC is the least noisy surface or quietest pavement surface in this study. For each dBA reduction in noise, whether at the tire or farther away a noise wall height can be reduced approximately 0.60 m (2 feet). Clearly as shown in Table 1 the ARFC is less noisy in absolute terms but beyond this, there is the second part of the story. When an ARFC is placed over a concrete freeway section the noise level reduction to the human ear seems even more dramatic. From noise testing done by the Arizona DOT and California DOT it was possible to record the noise spectrum over a wide range of frequencies.

Table 1. *CPX noise measurements at the tire/pavement interface and estimated noise 15 m (50 feet) away from tire*

Surface Type	CPX Noise	Noise 15 m
Random Trans	104.9	80.9
Transverse	102.5	78.5
Longitudinal	99.1	75.1
Whisper Grind	95.5	71.5
ARFC	91.8	67.8

The human ear can hear sounds from about 500 Hz to about 20,000 Hz. Sounds in the 1,000 to 2,000 Hz range tend to be of an annoying type to the human ear. By recording the frequency spectrum of the concrete pavements it was observed that transverse textured concrete has a tonal spike in the 1,000 to 2,000 Hz range (Scofield, 2003) that is particularly annoying to human hearing as shown in Figure 8. The longitudinal and whisper texture concrete has less of a spike, however the ARFC actually has a dip in the 1,000 to 2,000 Hz range which means to human hearing there is very much less annoying noise being heard. In effect the ARFC is both reducing the overall noise over all frequencies and the irritating tonal spike noise in the 1,000 to 2,000 Hz range.

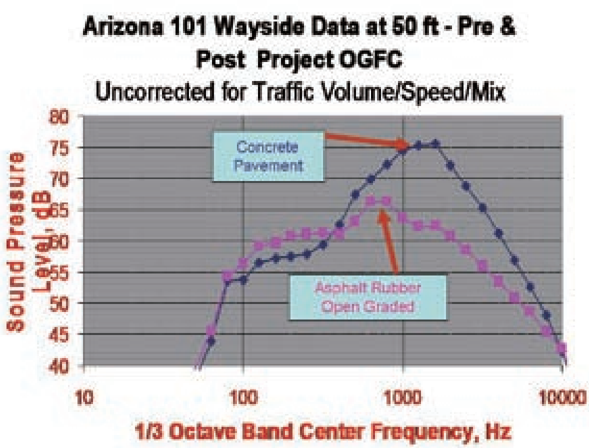

Figure 8. *Noise reduction in the 1,000 to 2,000 Hz range due to an ARFC surface, Scofield (2003)*

As further documentation of an Open Graded Asphalt Rubber Friction Course noise tests from many different surfaces were compared and combined into a single illustration (Donovan, 2003), shown in Figure 9. As can be seen the least noisy surface measured to date is an Open

Graded Asphalt Rubber Friction Course (ARFC). The Open Graded Asphalt Rubber Friction Course (ARFC) produces less noise as documented in numerous reports and by literally millions of people driving over the freeway pavement surfaces in the Phoenix Metropolitan area or living next to a freeway overlaid with an ARFC. The ARFC is less noisy because it contains air pockets, air voids that reduce the pressure change. Also, because of its very smooth riding due to the small aggregate size and because it is a layer of low modulus elastic soft material due to the high percentage of asphalt binder and crumb rubber. The Arizona DOT expects at least a 4-decibel reduction with the use of an ARFC and this reduction has led to a significant improvement in the quality of life of home owners living near the freeway and people driving on the freeway.

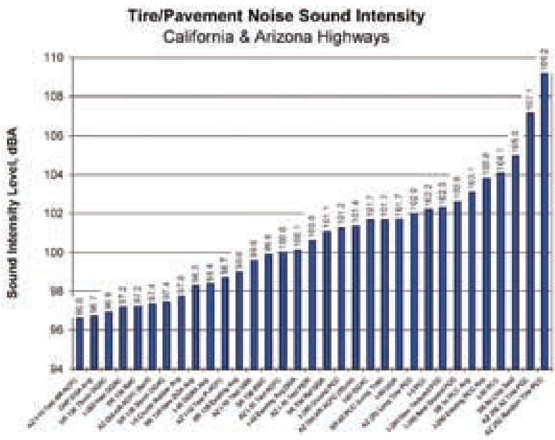

Figure 9. *Comparison of the CPX (near the tire) tire/pavement noise for different surfaces, (Donovan, 2003)*

7. Quiet Pavement Program Pilot Research Study

The Quiet Pavement Program Pilot (QPPP) research study is a ten year study being conducted in cooperation with Federal Highway Administration (FHWA) for the purpose of determining how long the reduction of noise associated with the ARFC placement will continue. The study as of 2010 is six years old. The study involves taking three types of noise measurements. A set of OBSI measurements are being taken at selected sites within the MAG freeway system. Wayside measurements are also being taken at selected at grade sites, and noise measurements are also being taken within selected neighborhoods. Although the study is not complete Table 2 shows the average level of reductions from measurements taken at the various sites before the ARFC was placed and then after six years of service. As can be seen the ARFC is still providing the minimum 4 dBA of noise reduction.

Table2. *ADOT MAG Freeway QPPP noise measurements reduction after six years of service*

Site 1- OBSI Noise Measurments	
Range of Noise Reductions	-4.1 dBA to -13.2 dBA
Average Noise Reduction	8.3 dBA
Site 2- Neighborhood Wayside Noise Measurements	
Range of Noise Reductions	+1.3 dBA to –12.3 dBA
Average Noise Reduction	5.3 dBA
Site 3- Roadway Wayside Noise Measurements	
Range of Noise Reductions	-4.4 dBA to -12.4 dBA
Average Noise Reduction	8.3 dBA

8. Conclusions

From the compilation of the data and analysis of the data obtained in the study the following conclusions can be derived Sousa (Sousa, 2005):

- Roadside measurements have shown that asphalt rubber friction courses can achieve 3 to 5 dB noise level reduction when compared to traditional asphalt dense graded surfaces and 6 to 12 dB noise level reduction when compared to concrete surfaces.

- Sound intensity measurements (CPX) have shown that asphalt rubber surfaces are effective in reducing noise by 4 to 6 dB compared with traditional dense graded asphalt concrete and by 6 to 12 dB when compared to concrete surfaces.

Undoubtedly more research will continue to find even better and quieter pavements, but for now an ARFC surface has set the standard for quiet pavements. Figure 10 is a general overview of the successful ADOT MAG Freeway that continues to reduce freeway noise to this date.

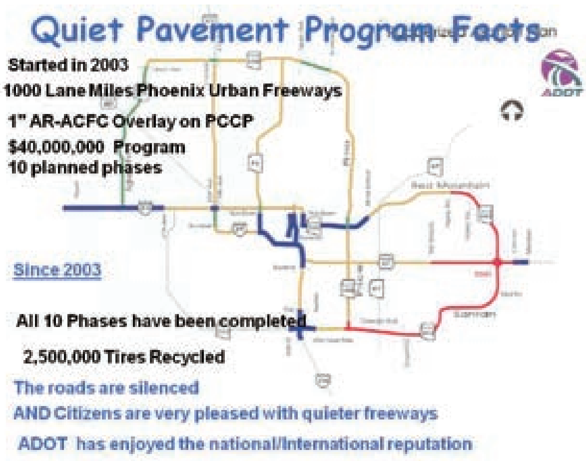

Figure 9. *ADOT MAG Freeway Quiet Pavement Program*

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Development of Innovative Pavement Types to Reduce Traffic Noise

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ABSTRACT. In recent years different innovative noise reducing road surfaces have been developed that either use asphalt or concrete, e.g. single or double layer porous asphalt, noise-reducing stone mastic asphalt, noise-reducing thin layers, surface treated concrete or exposed aggregate cement concrete. The noise reduction potential of such pavements could vary between 1 and 6dB(A) in comparison with conventional asphalt concrete surfaces depending on the road surface type and on its condition. In the context of a cooperation of BMVIT (Federal Ministry of Transport, Innovation and Technology of Austria), ASFINAG (Austrian Motorway Company), ISTU (Institute for Road Construction and Road Maintenance of the Vienna University of Technology) together with Heller Ingenieurgesellschaft mbH (Vienna) and representatives of the road construction industry, different innovative surface layer types have been examined for the first time in Austria under the same ambient conditions regarding to their noise characteristics, their durability and their influence on road safety compared with conventionally used surface layer types (stone mastic asphalt and asphalt concrete). The test section consists of altogether 8 different test fields with the surface layer types noise-reducing stone mastic asphalt, single and double layer porous asphalt and two reference surfaces. Analysing the first noise measurement results the clearly noise related advantage of porous asphalt surface layers both related to the overall traffic (SPB- index) and the individual vehicle categories could be shown. In comparison with the examined asphalt concrete surface layer they perform about 6-8dB(A) lower noise levels. The examined low noise stone mastic asphalt layers exhibit about 2–4dB(A) higher passing-by noise levels than the porous surface layer types.

KEYWORDS: traffic noise, noise reducing pavements, porous asphalt, noise measurements.

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1. Introduction

Due to rising traffic volume it is more and more recommended to reduce traffic noise directly at the source. The noise of a vehicle passing-by results on the one hand on the tyre rolling on the road surface, and on the other hand on sound emitted by the vehicle drive train and exhaust system. A substantial contribution to the reduction of traffic noise could be reached by minimising the tyre/road noise which depends mainly on the driven vehicle speed, on the type of tyre, on the pavement type and on the surface condition of the road.

2. Road traffic noise

2.1 Noise emission sources

The noise emitted by a passing vehicle on the road is a very complex situation. In general, one can divide the different factors influencing the overall noise into the main parts

- tyre/road noise,
- power unit noise and
- wind turbulence noise.

Tyre/road noise is related to the rolling of the tyres on the road surface which depends mainly on the driven speed, on the vehicle and tyre type and on the kind of the road surface and their characteristics. The influence of the tyre/road noise on the overall noise emission of vehicles passing by increases with an increased driving speed. In urban regions with speed limits around 30–50km/h the power unit of a vehicle is an important noise factor, whereas at highway speeds it is mostly negligible. Regarding driving speeds exceeding 120km/h wind turbulence noise plays an increasingly dominating role on the noise emissions.

2.2 Noise-reducing pavements

In recent years, noise-reducing road surfaces have been introduced that can lead to substantial reductions in traffic noise. However, the overall benefits of these noise-reducing surfaces are also influenced by other factors including the tyres of the vehicles, special vehicle parameters or the use of passive noise reduction measures such as barriers and traffic control measures for noise reduction. In this paper Austrian experiences concerning the influence of innovative noise-reducing pavement types are explained.

On the Austrian road network noise-reducing pavements are used for many years. An estimation of the standard pavement types on the Austrian high level road system is shown in Figure 1.

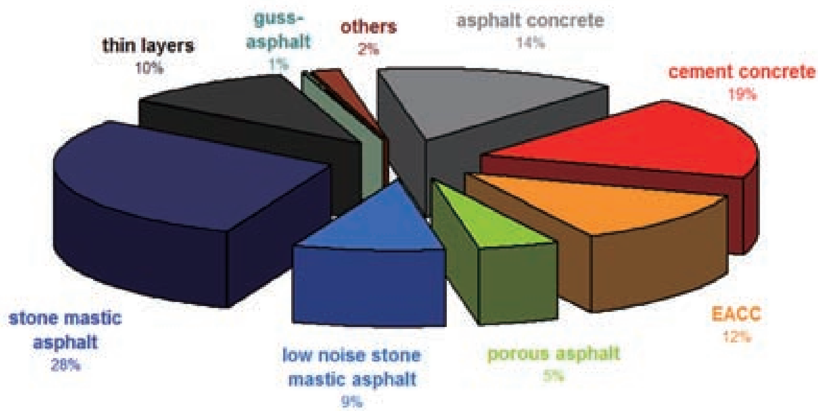

Figure 1. *Estimated road statistics on the Austrian high level road system (Haberl, 2008)*

3. Measurement methods

3.1. General statements

At present several measurement methods are used to assess traffic noise emissions, on the one hand for the classification of road surfaces and vehicles and on the other hand for the collection of the entire noise situation at close range of traffic routes. These methods differ basically in the measuring technique, especially in the different positions of the used microphones. Two principles are usually used, either rolling noise measurements with microphones near the vehicle tyres (near- field) or pass-by measurements with microphones on the side of the road (SILVIA, 2004).

3.1. Near-field measurements

Noise emissions in the near-field of the noise source can be measured by so called rolling noise measurements. The measurement itself in general takes place with a trailer furnished with a testing tyre within a sound-absorbing shroud. Because of the shroud construction of the trailer the used microphones are to a large extent shielded in relation to the background noise. The near-field noise emission is measured with microphones placed in direct proximity of the testing tyre regarding a constant vehicle speed over the whole measuring distance.

3.1.1. Austrian rolling noise measurement method

In Austria a procedure for measuring rolling noise emissions is regulated in the “guidelines and regulations for road construction” – RVS 11.06.64 (RVS, 1997) - and used either for acceptance testing of new pavements concerning the rolling noise or for a technical evaluation of different road surface layers concerning their noise behaviour. The tyre used is the PIARC-tyre with flow longitudinal grooves commonly used for skid-resistance measurements.

The results of rolling noise measurements are analysed by calculating the energy equivalent noise level over a defined road length, which is 500m according to the Austrian standard, expressed in the value LMA. LMA is calculated as the mean value regarding the measured frequency spectrum of two microphones over the measured road segment.

$$LMA_e = 10 \cdot \log\left(\sum_i 10^{\frac{LMA_{e,t,i}}{10}}\right) \qquad [1]$$

with:

LMAe	....	energy equivalent noise level of one test run
$LMA_{e,t,i}$	....	energy equivalent noise level of one microphone within the test run for each frequency spectrum
e	....	number of test run
t	....	number of microphone
i	....	frequency spectrum

Figure 2 shows an example of the rolling noise measurement device used in Austria.

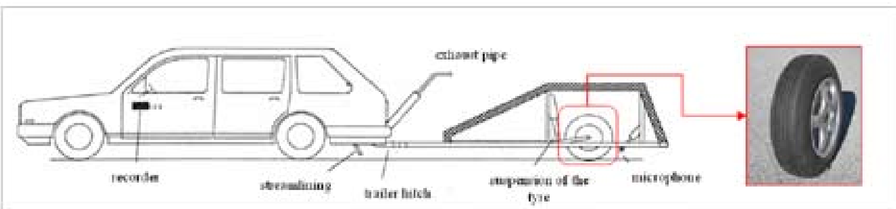

Figure 2. *Schematic of the rolling noise measurement device used in Austria with detail of the Austrian measurement tyre, photo by IFS Ziviltechniker Ges.m.b.H. (Haberl and Litzka, 2008)*

3.1.2 International rolling noise measurement method

An international measuring and analysis procedure for near-field noise measurements, the so called CPX- method regulated in the draft ISO/CD 11819-2 (ISO, 2000), is also based on the collection of the sound pressure level near the tyre/road contact area and exhibits thus many parallels to the Austrian method. The main differences will be found in the construction of the measuring trailer, whereby after ISO/CD 11819-2 more liberties are given in this field, in the use of up to 4 different test tyres and in the method of the evaluation of the measurement results. The 4 test tyres should represent the different tyres available on the market (2 types of summer tyres, 1 winter tyre and 1 heavy vehicle tyre).

The result of a CPX- measurement is expressed as an index value, CPXI – Close Proximity Index, and calculated as a weighted summary of the energy equivalent noise levels of the used test tyres (2 tyres for the survey purpose and 4 tyres for the investigatory purpose).

survey purpose:

$$CPXI = 0,2 \cdot L_A + 0,2 \cdot L_B + 0,2 \cdot L_C + 0,4 \cdot L_D \qquad [2]$$

investigatory purpose:

$$CPXI = 0,5 \cdot L_A + 0,5 \cdot L_D + 0,5 \qquad [3]$$

with:

CPXI	....	Close Proximity Index
LA	....	energy equivalent noise level of tyre A (representative for a summer tyre)
LB	....	energy equivalent noise level of tyre B (representative for a summer tyre)
LC	....	energy equivalent noise level of tyre C (representative for a winter tyre)
LD	....	energy equivalent noise level of tyre D (representative for a heavy vehicle tyre)

An example of a rolling noise measurement trailer and the four measurement tyres typically used in Austria is shown in Figure 3.

Figure 3. *Example of a rolling noise measurement trailer used in Austria and a detail of the 4 different measurement tyres A, D, B and C used in Austria according to ISO/CD 11819 - 2 (photos by IFS Ziviltechniker Ges.m.b.H.)*

3.1. Pass-by measurements

Pass-by measurements are regulated in the international standard ISO 11819 – 1, SPB, statistical pass-by (ISO, 1997). The measurement principle is very easy; the maximum A-weighted sound pressure level of a statistically significant number of individual vehicles passing by is measured at a specified road-side location together with the vehicle speed. An important fact for the comparison of measurement results is a standardised microphone position. Therefore the horizontal distance from the microphone position to the centre of the lane where the vehicles are passing by shall be in the range of 7,5 m ± 0,1 m, the associated microphone height above the plane of the road surface shall be 1,2 m ± 0,1 m (see Figure 4).

The result of a SPB- measurement is expressed as an index value, SPBI – Statistical Pass-By Index, and calculated as a weighted summary of the maximum sound pressure levels of three different vehicle categories, cars (category 1), dual-axle heavy vehicles (category 2a) and multi-axle heavy vehicles (category 2b), at a certain reference speed. Category 2a and category 2b can be summarised as category 2 – heavy vehicles. To ensure that random errors do not become unacceptable large, the number of measured vehicles is minimised for each vehicle category (cat.1 – min. 100veh., cat. 2a – min. 30veh., cat. 2b – min. 30veh. and additionally cat. 2 – min. 80veh.)

$$SPBI = 10 \cdot \log\left[W_1 \cdot 10^{L_1/10} + W_{2a} \cdot \left(\frac{v_1}{v_{2a}}\right) 10^{L_{2a}/10} + W_{2b} \cdot \left(\frac{v_1}{v_{2b}}\right) \cdot 10^{L_{2b}/10}\right] \qquad [4]$$

with:

SPBI .... Statistical Pass-By Index
L_1, L_{2a} and L_{2b} .... sound levels for vehicle categories 1, 2a and 2b at the reference speeds v1, v_{2a} and v_{2b}
W_1, W_{2a} and W_{2b} .... weighting factors which are equivalent to the assumed proportions of vehicle categories in the traffic
v_1, v_{2a} and v_{2b} .... reference speeds of the individual vehicle categories 1, 2a and 2b

Within pass-by measurements the situation under real traffic (different vehicles, different driving speeds, …) can be analysed. Otherwise pass-by measurements are taking place on a certain point only, while rolling noise measurements are recording the situation over a certain length.

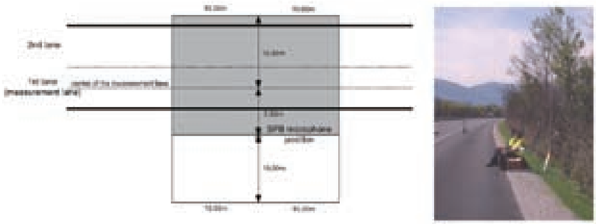

Figure 4. *Schematic and reality for the test set-up for pass-by measurements according to ISO 11819 – 1 (adopted from (Haberl and Litzka, 2008))*

4. Evaluation of the noise reducing potential of different pavement types

4.1 Test section with different noise reducing asphalt pavements

4.1.1 General statements

In the context of a cooperation of the Austrian Federal Ministry of Transport, Innovation and Technology (BMVIT), the Austrian Motorway Company (ASFINAG), the Austrian Alpine

Motorway Company (ASG), the Institute for Road Construction and Road Maintenance of the Vienna University of Technology (ISTU) together with Heller Ingenieurgesellschaft mbH (Vienna) and representatives of the road construction industry, a concept of a test section "Noise-reducing road surfaces" was developed with the aim to investigate the noise-reducing effect of three innovative surface layer types, single and double layer porous asphalt and low noise stone mastic asphalt.

4.1.2 Location and investigated pavement types

The test section is situated in Tyrol at the motorway A12 – Inntal Autobahn – on the carriageway direction Innsbruck (km5,80 – km10,40, see Figure 5).

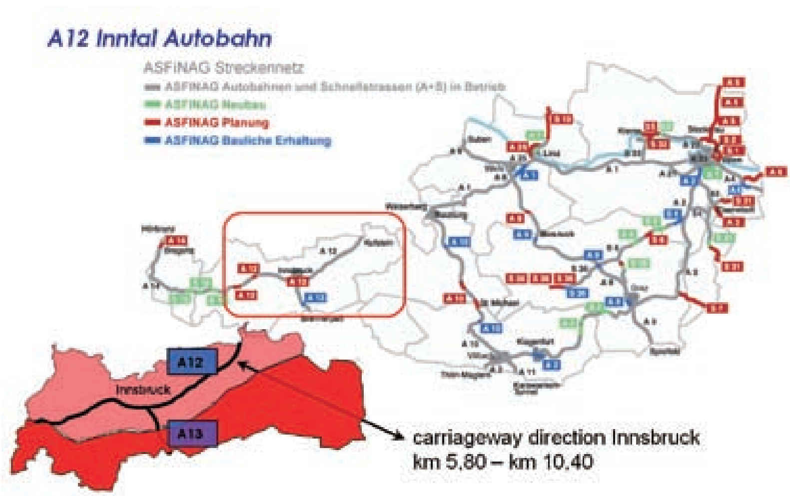

Figure 5. *Test section situated in Tyrol, Austria (adopted from (ASFINAG, 2009))*

It consists of altogether 8 different test fields with the following surface layer types:

- LSMA 8, rubber modified bitumen, noise-reducing stone mastic asphalt
- LSMA 11, rubber modified bitumen, noise-reducing stone mastic asphalt
- LSMA 11, polymer modified bitumen, noise-reducing stone mastic asphalt
- ZDA, polymer modified bitumen, double layer porous asphalt
- ZDA, rubber modified bitumen (CTS bitumen[1]), double layer porous asphalt
- DA 8, rubber modified bitumen (CTS bitumen), single layer porous asphalt

Additionally 2 reference surface layers, asphalt concrete (pmAB11) and stone mastic asphalt (SMA11), were laid on the test section.

1 CTS bitumen is a special product of "Cts Bitumen GmbH" which contains a certain percentage of rubber ingredients

Detailed information like information about the used binder, binder content, binder additives, air void content and layer thickness of the investigated surface layer types is given in Figure 6.

surface course	binder	binder content [M-%]	binder additive	air void content [V-%]	thickness of layer [cm]	length of section [km]
LSMA 8 rubber modified	B 50/70	6,20	1,0% ASAplex M100	10,10	3,00	1,00
LSMA 11 rubber modified	B 50/70	6,00	1,0% ASAplex M100	10,60	3,50	0,50
SMA 11 polymer modified	PmB 50-90S Starfalt OMV	6,30	cellulose fibres	3,00	3,50	0,50
LSMA polymer modified	PmB 50-90S Starfalt OMV	5,70	cellulose fibres	10,60	3,50	0,50
DLPA polymer modified					2,50 + 4,50	0,50
PA 11/16	PmB 50-90S Starfalt OMV	4,90	cellulose fibres	22,90	4,5	
PA 4/8	PmB 50-90S Starfalt OMV	6	cellulose fibres	23,10	2,5	
DLPA rubber modified					2,50 + 4,50	0,40
PA 8/16	B 70/100	4,50	CTS Bitumen	27,30	4,5	
PA 6/8	B 70/100	6,50	CTS Bitumen	25,10	2,5	
PA 8 rubber modified	B 70/100	6,50	CTS Bitumen	25,10	4,50	0,60
AC 11 polymer modified	PmB 50-90S Starfalt OMV	5,40	/	3,40	3,50	0,60

Figure 6. *Information about investigated surface courses*

4.1.3 Pass-by measurement results

Measurements conducted after a laying period of the surface layers of about 2 years show a clearly noise-related advantage of the three porous asphalt surface layers (ZDA with polymer modified bitumen, ZDA with rubber modified bitumen, CTS and DA8 with rubber modified bitumen, CTS) both related to the overall traffic (SPB-Index) and the individual vehicle categories. Analyses of the SPB-Index are shown in Figure 7.

Figure 8 shows the noise reduction potential of the investigated road surface courses compared to stone mastic asphalt (SMA11). The difference in the SPB-Index between the reference surface types SMA11 and pmAB11, which behave as the loudest, and the two double layer porous asphalt layer types, which show the quietest noise performance, is about 8dB(A). A noise reduction in this range is really perceivable and audible for human beings. If one refers to mathematical calculation, a reduction of 6dB(A) may be reached by a reduction of the vehicle fleet to a quarter (Hoffmann et al., 2003).

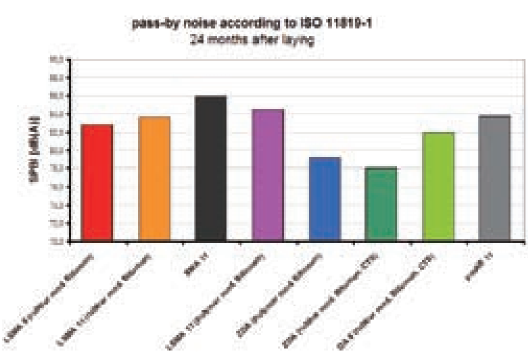

Figure 7. *SPB-Index of the investigated road surface courses according to the international standard ISO 11819-1 (adopted from (Haberl and Litzka, 2008))*

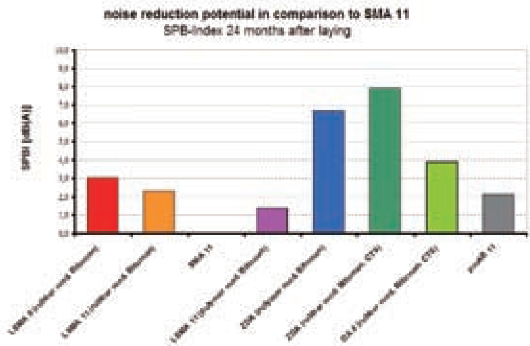

Figure 8. *Noise reduction potential of the investigated road surface courses compared to stone mastic asphalt SMA 11 (adopted from (Haberl and Litzka, 2008))*

4.1.4 Rolling noise measurement results

Rolling noise measurements were accomplished with the two different driving speeds v=70km/h and v=100km/h 2 years after the laying of the investigated road surface types. The results for the 8 examined surface layer types are presented in Figure 9.

In general the results of rolling noise measurements show a similar tendency like those of the pass-by measurements. The examined double layer porous asphalt surface layer types show the lowest noise volumes; however the LSMA8 with rubber modified bitumen lies more or less on the same level. This circumstance can be explained with the fact that by the proximity of the used microphones to the examined tyre and the examined surface layer the absorption effect of the porous asphalt road surfaces may be underestimated. Within pass-by measurements the absorption capacity of porous asphalt layers is much more regarded.

The analyses of the influence of different vehicle speeds on the rolling noise emissions show the same range in the LMA-values regarding the different road surface types.

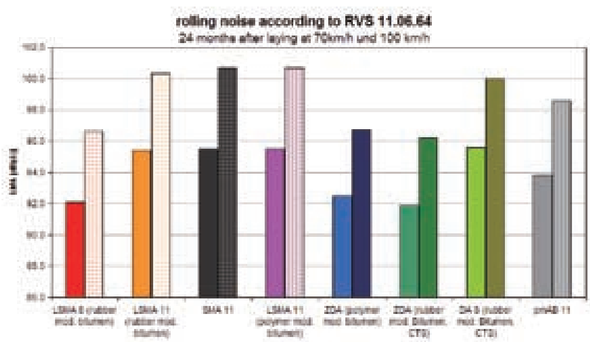

Figure 9. *Results of rolling noise measurements according to the Austrian standard RVS 11.06.64, LMA-value – the first bar refers to 70km/h and the second to 100km/h (adopted from (Haberl and Litzka, 2008))*

Rolling noise measurements were also conducted according to the international standard ISO/CD 11819-2. The analyses of the CPX-Indexes show the same tendencies than demonstrated before regarding the Austrian standard. The results of the CPX- measurements with driving speeds v=70km/h and v=100km/h are shown in Figure 10.

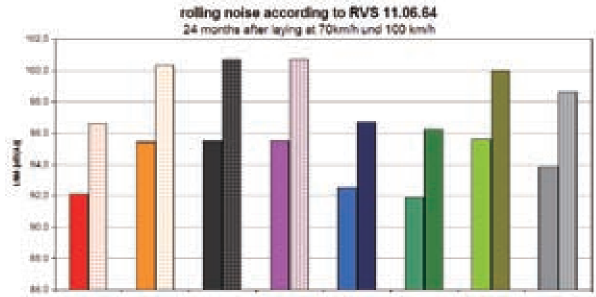

Figure 10. Results of rolling noise measurements according to the international standard ISO/CD 11819-2, CPX- Index – the first bar refers to 70km/h and the second to 100km/h (adopted from (Haberl and Litzka, 2008))

4.2 Long term noise behaviour of road surface courses

An analysis of the noise reduction potential of different road surface courses was performed by the institute of road construction and maintenance of the Vienna University of Technology. Within this research project the standard road surface types, eg. porous asphalt and exposed aggregate cement concrete, situated on the high level road system were examined (see Figure 11).

Looking on the very first years of the lifetime of the investigated porous asphalt layers (DA 11) their big noise reduction potential is demonstrated. But after a period of around 5 years their noise emission performance is about the same level as the other investigated surface courses. This could be an effect of the ongoing clogging processes or of occurring aggregate losses, especially after hard winter times. In contrast to this the two noise-reducing surface courses LDDH8 (noise-reducing thin layers) and LSMA8 (noise-reducing stone mastic asphalt) could be found in the quietest range in all age classes. The remarkable thing is that the examined exposed aggregate cement concrete layers (WB GK8, blue rectangles) are situated in the loudest range at the beginning of their lifetime. But regarding the lifetime of the road surface courses they are advancing to have the best (lowest) noise performance of all examined road surfaces.

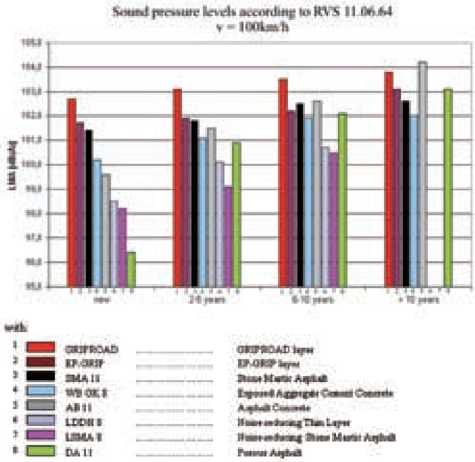

Figure 11. *Long term noise behaviour of different road surface courses (Haberl and Litzka, 2005)*

5. Summary and Conclusions

In the present paper a test section of different innovative noise reducing asphalt pavements was explained in detail. The test section is situated in the western part of Austria, in Tyrol, on the motorway A12 (Inntal Autobahn) and consists of 8 different surface layer types (3 types of low noise stone mastic asphalt, one single layer porous asphalt, 2 types of double layer porous asphalt and 2 reference surfaces - asphalt concrete and stone mastic asphalt).

Conducted noise measurements show the clearly noise related advantage of the three porous asphalt surface layers in comparison with the other investigated road surface courses. The range between the reference surface types SMA11 and pmAB11, which behave as the loudest, and the two double layer porous asphalt layer types, which show the quietest noise performance, is about 8dB(A).

However, a detailed statement both about the noise reducing behaviour and the lifetime of the examined surface layer types will be possible after investigations during a minimum investigation period of about 3-5 years only.

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Field Investigation of Tire/Pavement Noise and Durability for Asphalt Pavements with and without Asphalt Rubber

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ABSTRACT. This paper presents the results of a three-year field study that compares the performance trends of various asphalt surface mixes, with and without asphalt rubber. The performances being investigated include tire/pavement noise, durability, friction, and permeability. About 80 field sections on California highway system were investigated, which covered a variety of asphalt surface mixes: rubberized open-graded mixes (RAC-O), rubberized gap-graded asphalt concrete (RAC-G), open-graded mixes with conventional binders (OGAC), and dense-graded asphalt concrete (DGAC). Roughness, noise, and surface distress data were collected on each section for three consecutive years, and permeability and skid resistance (friction) were collected under traffic closures for the first two years. Cores were also taken in the first two years from each section to determine air-void contents and aggregate gradations in the laboratory. The tire/pavement noise was measured by the On-Board Sound Intensity (OBSI) method. Analysis of the data indicates that RAC-G mixes provide some noise benefit compared to DGAC. However, the noise benefits are not as great *as those from open-graded mixes. The noise levels from RAC-G mixes appear to approach those of DGAC after a few years in service. The data also indicate that RAC-O mixes appear to provide longer noise reduction than OGAC mixes, while both mixes provide noise and permeability benefits.*

KEYWORDS: tire/pavement noise, asphalt pavement, rubber, durability

1. Introduction

The smoothness and quietness of pavements have received increasing attention from both the public and transportation agencies as issues of quality of life for highway users and neighboring residents. The concept of using quieter pavements to reduce noise is being evaluated in California and nationwide over the past several years.

In the last few decades, open graded asphalt concrete (OGAC) surface mixes have been placed in California and other states to reduce the dangers of hydroplaning and poor visibility caused by splash and spray during wet weather (Caltrans, 2006). Due to its high air-void contents and use as a sacrificial wearing course, OGAC mix can also reduce tire/pavement noise and improve road smoothness. The durability of the mixes, particularly raveling distress, and the long-term effectiveness of noise reduction, however, have been a concern and results of different reports vary.

In the last two decades, hundreds of rubberized asphalt concrete (RAC) projects have been constructed throughout California, the majority of which used asphalt rubber mixes in overlays for maintenance and/or rehabilitation of existing asphalt concrete and Portland cement concrete pavements. In California, asphalt rubber is specified to include 18 to 22 percent crumb rubber modifier (CRM) by total mass of the asphalt rubber blend (Caltrans, 2003). The most commonly used asphalt rubber product in California is gap-graded asphalt rubber mixes (called rubberized asphalt concrete, gap-graded, RAC-G), which can provide better resistance to reflective cracking and fatigue cracking than standard dense-graded asphalt concrete (DGAC). The structural and reflection crack retardation equivalencies for RAC-G allows its substitution for DGAC at about one-half the thickness (Caltrans, 2003). There was some evidence from field investigation that the tire/pavement noise tends to be lower on RAC-G pavements than that on the conventional DGAC pavements (Ongel et al. 2008; Lu et al. 2009).

Another use of asphalt rubber product in California is asphalt rubber open graded mixes (RAC-O). RAC-O mixes are primarily used as maintenance blankets, overlays for rehabilitation (including restoration of surface friction), or surface courses for new construction. They are not considered to be structural elements and are often placed about 24 to 30 mm thick (Caltrans, 2003). Compared to conventional OGAC mixes, RAC-O mixes contain higher binder content, and can provide better resistance to pavement distresses, such as reflective cracking, in addition to its noise-reducing property.

Although a variety of mixes are available for pavement maintenance and rehabilitation, their noise reducing properties and the longevity of these properties compared to other mix types are not well quantified. There is a need to identify the life of current strategies, as well as new materials and designs, capable of maintaining smoothness and quietness for the longest time.

The purpose of this study is to determine the noise levels, friction, smoothness, and performance trends of different asphalt surface mixes in the field, which would allow for identification of more durable, smoother, and quieter pavement types.

2. Methodology

2.1. Site Selection

This study presents the analysis of data collected over 3 year from 72 field pavement sections (each with a length of 150 m) in California. The selection of the pavement sections is a full factorial experimental design including four different asphalt pavement surface types, three different age categories, two traffic types, and two rainfall regions. The four mix types include open graded asphalt concrete with conventional or polymer-modified binders (OGAC), open graded asphalt concrete with rubberized binder (RAC-O), rubberized gap graded asphalt concrete (RAC-G), and dense graded asphalt concrete with conventional or polymer-modified binders (DGAC). Age categories include less than 1 year old, 1–4 years old, and 4–8 years old. Traffic type is categorized as "high" if the 2004 annual average daily traffic (AADT) data is greater than 32,000 vehicles/day and is categorized as "low" otherwise. Rainfall is categorized as "high" if average annual rainfall is greater than 620 mm and is categorized as "low" otherwise. Such an experimental design provides a balanced selection of pavement types under various traffic and climate conditions across the state.

2.2. Data Collection

Those selected pavement sections have been tested for three years. The first two years of data included coring, condition survey, permeability, and friction tests within traffic closures, profile and tire/pavement noise measuring within normal traffic stream, and mix property testing on cores in the laboratory. Data collection was continued in the third year, but on a smaller scale, in which coring, permeability, and friction tests were not conducted.

The tire/pavement noise was measured by the onboard sound intensity (OBSI) method. In this method two locations of the sound intensity probe are used: one is at the leading edge and the other at the trailing edge of the tire/pavement contact patch. OBSI measurements are taken at 97 km/h. An alternative speed of 58 km/h is used if the higher speed cannot be reached due to constraints of highway geometry or traffic conditions. Three replicate measurements are collected in three consecutive passes on the sections. Air and pavement temperatures are also recorded during OBSI measurements. Measurements were conducted using a Goodyear Aquatread III tire and Dodge Stratus car in the first two years and using a standard reference test tire (SRTT) and Dodge Stratus car on some sections in the second year and on all sections in the third year. The first two years OBSI results measured from the Aquatread III tire were converted into the equivalent measurements from the SRTT using a series of conversion functions developed in this study, and are all expressed in terms of A-weighted sound intensity levels, dB(A).

Microtexture was measured using the British pendulum tester according to ASTM E303. Microtexture measurements were conducted at 25 m intervals and the results were expressed in terms of British pendulum numbers (BPN). Permeability was measured using a falling-head permeameter, a device developed at the National Center for Asphalt Technology (NCAT) (Cooley et al. 2001). Macrotexture was measured according to ASTM E1845, using a high sampling frequency laser profilometer on the instrumented vehicle used for the sound intensity measurements. Macrotexture results are reported in terms of mean profile depth

(MPD). Roughness was measured with the inertial laser profiler and reported as international roughness index (IRI). Pavement condition surveys were conducted following the Caltrans *Condition Survey Manual* (version year 2000). A total of 12 cores were also collected, six in the wheelpath and six between the wheelpath, at 25 m intervals from the selected pavement sections to determine the air void content and aggregate gradation. Air-void contents were calculated using the bulk specific gravity value obtained from CoreLokTM measurements and the theoretical maximum specific gravity value obtained according to ASTM D2041. After the asphalt from the core samples was burned off in an ignition oven, the aggregate gradation was obtained by sieve analysis according to ASTM C136 and ASTM C117. Thicknesses of the cores were also measured and recorded in the laboratory (Ongel et al. 2008).

3. Results

3.1. Roughness

The IRI measurements were collected every meter in both the left and right wheel paths. The average of the two wheel path measurements along the whole length of each pavement section was used in the analysis.

Figure 1 shows the boxplots of IRIs in three survey years for different mix types for three age categories. As the figure shows, IRI generally increases with time. This is expected because pavement conditions deteriorate with age due to traffic and environmental effect. However, there are some sections, particularly those OGAC sections, showed little change in IRI in the three-year survey period. For new mixes (Age Category "<1 year"), IRI was slightly reduced for OGAC and RAC-O mixes in the second years. RAC-G, on the other hand, showed significant increase in IRI in the first three years after construction. This is due to an outlier in the data set.

In general, all pavement sections showed acceptable IRI values based on FHWA criteria (Ongel et al. 2008), except one old DGAC pavement that has an age of 14 years at the beginning of the survey.

Multiple regression analysis on all the mixes showed that rubberized open-graded mixes have lower initial IRI values than nonrubberized open-graded mixes; while rubberized gap-graded mixes have lower initial IRI values than non-rubberized dense-graded mixes. Among the three pavement types, OGAC, RAC-G, and RAC-O, they all have lower initial IRI than DGAC, but only RAC-O is statistically significantly different from DGAC.

Multiple regression analysis on each individual mix showed that open-graded pavements (both OGAC and RAC-O) are smoother in high temperature regions that in low temperature regions. For both OGAC and RAC-G pavements, IRI is affected by MPD. IRI increases with MPD on OGAC pavements, but decreases with MPD on RAC-G pavements. Traffic volume significantly affects IRI only on RAC-G pavements. Higher traffic volume leads to higher IRI values.

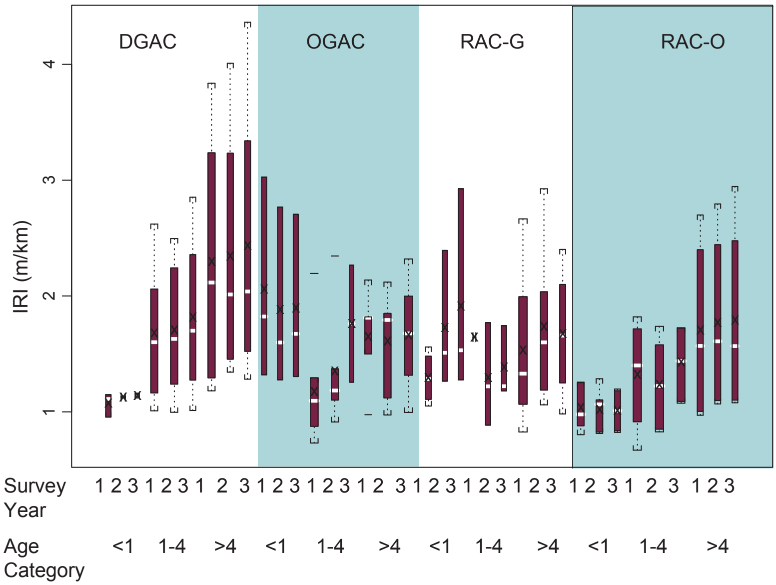

Figure 1. *Comparison of IRI values for different mix types at different ages for first, second, and third survey years*

3.2. Macrotexture

In this study, macrotexture is characterized by mean profile depth (MPD). Figure 2 shows the boxplots of MPDs in three survey years ("Phase" as shown in the figure) for different mix types for three age categories. As the figure shows, MPD generally increases with pavement age for the same pavement section. Except for a few outliers, this increase trend is also obvious among different pavement sections of the same mix type. Without the outlier in RAC-G mixes, RAC-G mixes have higher MPD values than the dense-graded mixes, while the open-graded mixes have higher MPD values than the RAC-G mixes. Among the two open-graded mixes, OGAC mixes have higher MPD values than RAC-G mixes.

Multiple regression analysis on all the mixes showed that at the 95 percent confidence level, age, mix type, and number of high-temperature days significantly affect macrotexture. MPD increases with age, but decreases with the number of high-temperature days. P-values for the interaction terms between Age and Mix Type showed that the growth rate (with age) of MPD of OGAC pavements is significantly higher than that of DGAC pavements. The growth rates of MPD of RAC-G and RAC-O pavements are not statistically different from that of DGAC pavements.

Multiple regression analysis on each individual mix showed that within each mix type, air-void content has no significant effect on the value of MPD. Fineness modulus is significant in affecting the macrotexture of open-graded pavements, including both OGAC and RAC-O, marginally significant in affecting the macrotexture of DGAC pavements, and insignificant for RAC-G pavements. Generally macrotexture increases with fineness modulus. The effect of pavement age on macrotexture is much more prominent (in terms of both statistical significance and practical significance) on non-rubberized pavements (DGAC and OGAC) than on rubberized pavements (RAC-G, and RAC-O).

3.3. Tire/Pavement Noise

The on-board sound intensity (OBSI) averaged from three consecutive passes is used in the analysis. The OBSI results are given in terms of spectral content in one-third octave bands. Summation of the one-third octave band noise levels gives the overall A-weighted sound intensity levels. Because sound intensity measurements are highly affected by test car speed, air density, and type of test tire, the original OBSI results were all converted to equivalent values at the same test conditions before analysis.

Figure 3 shows the box plots of overall OBSI in three years for different mix types for three age categories. As the figure shows, the overall tire/pavement noise generally increases with pavement age. For newly paved pavements, the overall sound intensities measured on OGAC, RAC-G, and RAC-O pavements are lower than the values measured on the DGAC pavements. After the pavements were exposed to traffic, the overall sound intensity measured on RAC-G pavements quickly approached the representative value measured on DGAC pavements. The overall sound intensity measured on the OGAC pavements kept stable for about five years and then increased quickly with pavement age. With a few exceptions, the overall sound intensity measured on the RAC-O pavements kept stable for about seven years and then increased quickly with pavement age. Based on these observations, the rank of the four mix types in terms of noise reduction is RAC-O, OGAC, RAC-G, and DGAC (from best to worst).

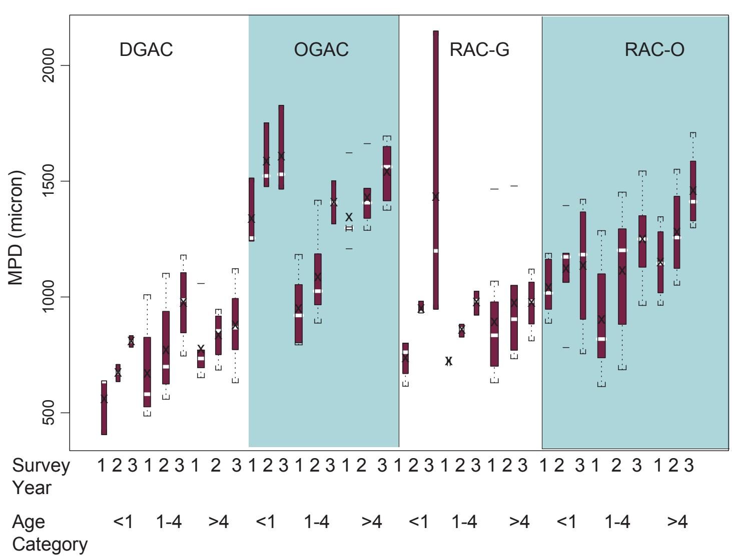

Figure 2. *Comparison of MPD values for different mix types at different ages for first, second, and third survey years*

Regression analysis was conducted to determine the effects of mix properties, distresses, traffic, and weather conditions on the sound intensity levels. A single variable regression analysis was first conducted to check the correlation between the dependent variable and each independent variable, and then a multiple regression model was estimated to consider the effects of various variables simultaneously.

In the third year survey, air-void content, permeability, and friction (BPN) were not measured in the field. To use these variables in the regression analysis, the third-year data were extrapolated from the first two-year data. Boxplots of these three variables are shown in Figure 4 through Figure 6. It can be observed from the boxplots that the air-void content generally reduces with time for all mixes, and the in-situ permeability reduces with time for OGAC, RAC-G, and RAC-O mixes. For RAC-G pavements, the in-situ permeability is as high as at a comparable level to that of open-graded pavements in the first three years after construction, but rapidly reduces to a near-zero level after 4 or 5 years old. Surface friction (BPN) tends to increase slightly with pavement age. Based on these observations, linear extrapolation was applied to estimate the third-year values of air-void content, permeability, and BPN from the first two years' data.

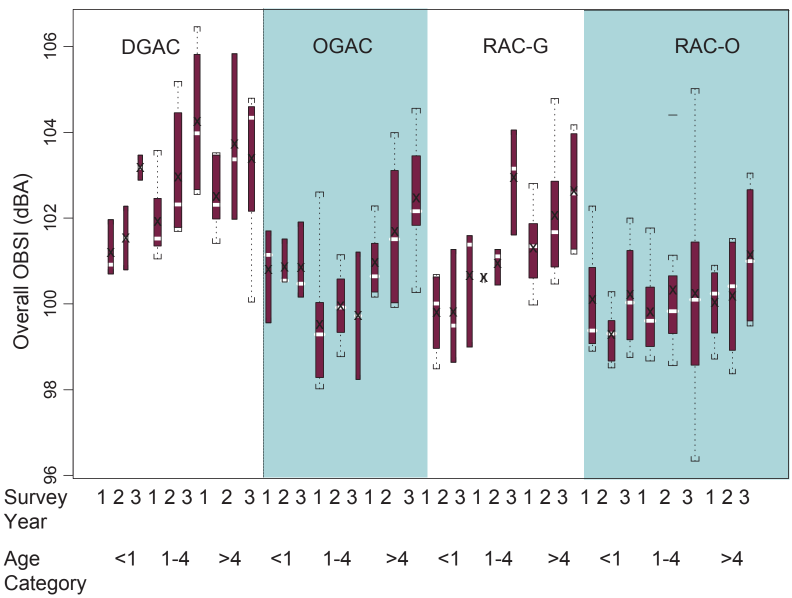

Figure 3. *Comparison of overall OBSI values for different mix types at different ages for first, second, and third survey years*

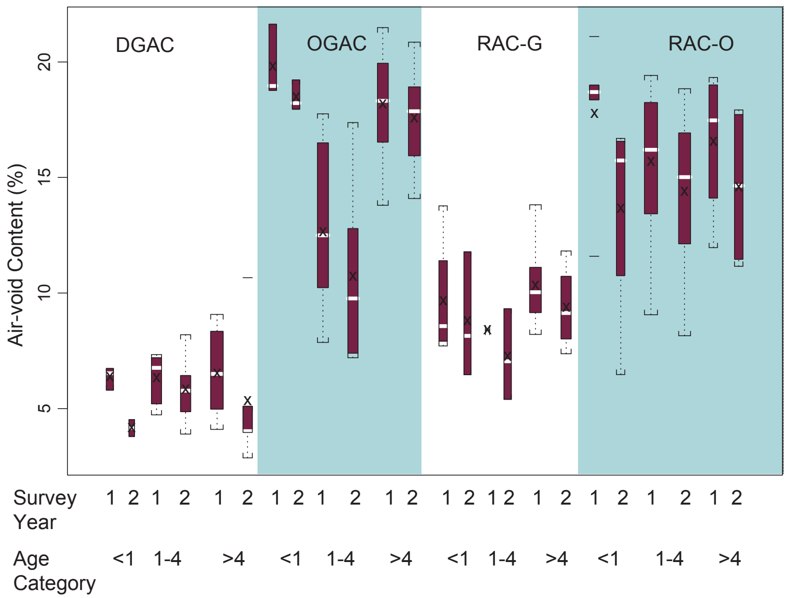

Figure 4. *Boxplots of air-void content for different mix types at different ages for first and second survey years*

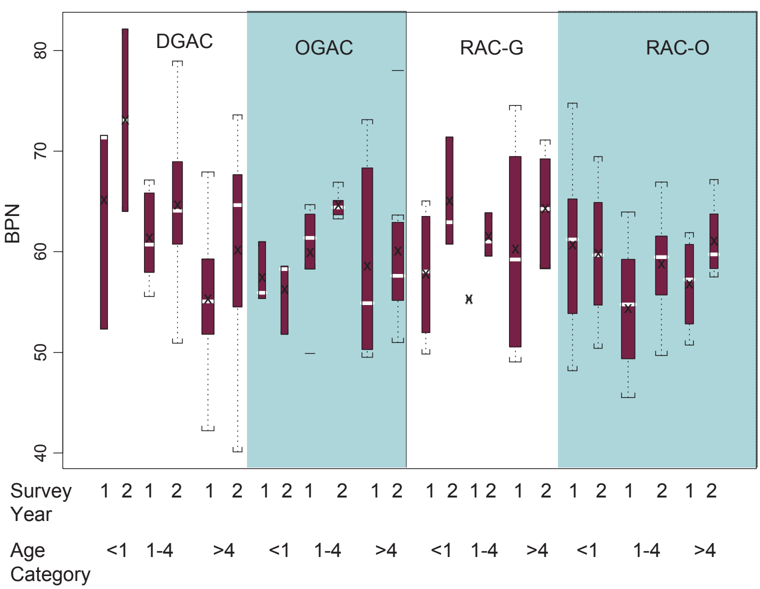

Figure 5. *Boxplots of BPN for different mix types at different ages for first and second survey years*

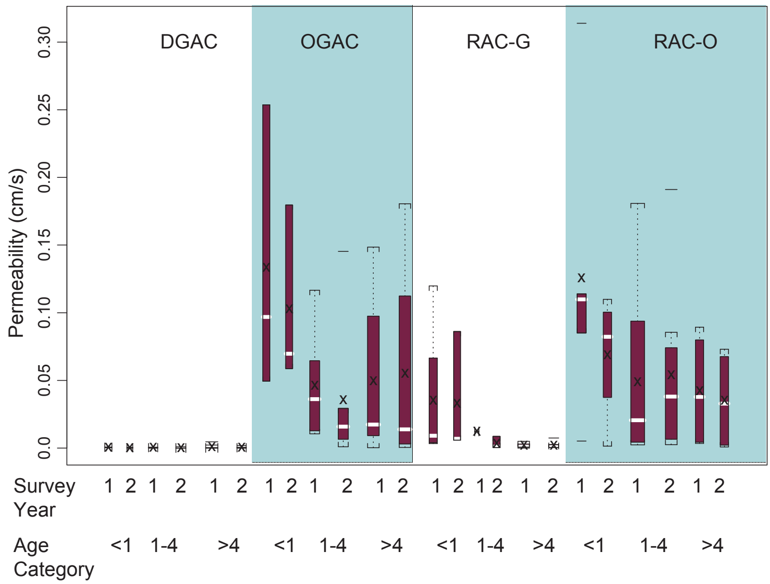

Figure 6. *Boxplots of permeability for different mix types at different ages for first and second survey years*

After excluding highly correlated independent variables, the following multiple regression model was estimated:

$$
\begin{aligned}
&\mathit{Overall\ Sound\ Intensity} = 103.4440 + 0.1488 \times \mathit{Age} - 2.2821 \times \mathit{ind}(\mathit{MixTypeOGAC}) - 1.6271 \times \mathit{ind}(\mathit{MixTypeRAC} - G) \\
&-2.8749 \times \mathit{ind}(\mathit{MixTypeRAC} - O) - 0.0233 \times \mathit{NMAS} - 0.0252 \times \mathit{Thickness} - 0.0018 \times \mathit{NumberOfDays} > 30C \\
&-1.0635 \times 10^{-6} \times \mathit{Age} \times \mathit{AADTinCoringLane} + 0.9139 \times \mathit{Raveling} + 0.5677 \times \mathit{Rutting} + 0.0348 \times \mathit{Age} \times \mathit{ind}(\mathit{MixTypeOGAC}) \\
&+0.0849 \times \mathit{Age} \times \mathit{ind}(\mathit{MixTypeRAC} - G) - 0.0384 \times \mathit{Age} \times \mathit{ind}(\mathit{MixTypeRAC} - O)
\end{aligned}
$$

(1)

where $ind(\cdot)$ is an indicator function, 1 if the variable in the parentheses is true and 0 if false. The estimated values and P-values of the parameters are shown below:

Table 1. *Multiple regression analysis results for overall OBSI*

	Value	Std. Error	t value	P-value
(Intercept)	103.4440	0.9063	114.1421	0.0000
Age	0.1488	0.0758	1.9621	0.0500
PvmntTypeOGAC	-2.2821	0.5856	-3.8968	0.0001
PvmntTypeRAC-G	-1.6271	0.5268	-3.0886	0.0024
PvmntTypeRAC-O	-2.8749	0.5251	-5.4745	0.0000
NMAS	-0.0233	0.0480	-0.4849	0.6284
Thickness	-0.0252	0.0068	-3.6850	0.0003
NoDaysTempGT30	-0.0018	0.0022	-0.8497	0.3968
AgeAADTperLane	-1.0635	0.0000	-0.3473	0.7289
Raveling	0.9139	0.2594	3.5229	0.0006
Rutting	0.5677	0.3921	1.4477	0.1497
AgePvmntTypeOGAC	0.0348	0.0988	0.3518	0.7255
AgePvmntTypeRAC-G	0.0849	0.1041	0.8155	0.4160
AgePvmntTypeRAC-O	-0.0384	0.0936	-0.4100	0.6823

Residual standard error: 1.286 on 154 degrees of freedom; Multiple R-Squared: 0.5518

It can be seen that at the 95 percent confidence level, age, mix type, surface layer thickness, existence of raveling significantly affect the overall sound intensity. The overall sound intensity increases with pavement age and the existence of raveling distress, but decreases with the surface layer thickness. Among the three pavement types, OGAC, RAC-G, and RAC-O, they all have lower initial overall sound intensity than DGAC. The average noise reductions (compared to DGAC pavements) for newly placed OGAC, RAC-G, and RAC-O pavements are about 2.3, 1.6, and 2.9 dB(A), respectively.

The interaction terms between age and mix type are not statistically significant, which indicates that the growth rate of overall sound intensity is not statistically different among the four pavement types. This conclusion is different from the direct observations from Figure 3. This is mostly due to the constraints applied by the multiple regression analysis. The regression analysis assumes a linear increase of noise with age for all mixes, but Figure 3 indicates that the noise development on open-graded mixes is more like piecewise linear. Use of different growth function forms for different mixes in the same regression model significantly increases the complexity of parameter estimation and result interpretation, which is not attempted in this study. Considering the total noise increase during the pavement life covered by the data set in this study (about 10 years), the estimated parameters of the interaction terms indicate that the noise increase is higher on OGAC and RAC-G pavements than on DGAC pavements, and the lowest on RAC-O pavements.

3.4. Pavement Distresses

In the third-year survey, pavement conditions were evaluated in a way different from the way used in the previous two years. In the first two years' survey, the truck lane was temporarily closed and pavement conditions were measured on site during the traffic closure. During the third-year survey, however, lanes were not closed. Instead, high-resolution digital photos were taken from the shoulder along the whole length of each section, and pavement conditions were assessed afterwards in the office, based on the pavement surface images. It has to be noted that some distresses, such as rutting, could not be evaluated accurately solely based on surface images. Because of the different ways of distress assessment in the first two years and the third year, some distresses were recorded as less severe in the third year than the previous years. A basic assumption was made in post-processing the distress data, that is, the third-year distress is no less than the second year.

In this study, four major distress types, including bleeding, transverse/reflective cracking, raveling, and wheel path cracking, were analyzed.

Figure 7 shows the percentage of bleeding area measured in three consecutive years for individual pavement sections of four mix types. In this figure, bleeding includes all three severity levels (low, medium, and high). The figure shows that bleeding may appear two to four years after construction on all pavement types, and it tends to appear earlier on rubberized pavements than on nonrubberized pavements. Among the four mix types, RAC-G pavements seem to be most susceptible to bleeding distress in terms of both the time of occurrence and the extent of distress. Multiple regression analysis reveals that bleeding increases with age, number of wet days, number of high-temperature days, and cumulative truck traffic, but decreases with the number of freeze-thaw cycles. Among the four pavement types, OGAC and RAC-O pavements have no significant difference from DGAC pavement, but RAC-G pavement is significantly (statistically) more prone to bleeding.

Because all the sections investigated in this study are overlays of AC or PCC and it is difficult to distinguish the thermal and reflective cracking mechanisms based only on surface condition observations, the analysis in this study combines the thermal cracking and reflective cracking as one distress type. Figure 8 shows the average length of transverse/reflective cracking (at all severity levels) per unit length of pavement observed on each pavement section in three years.

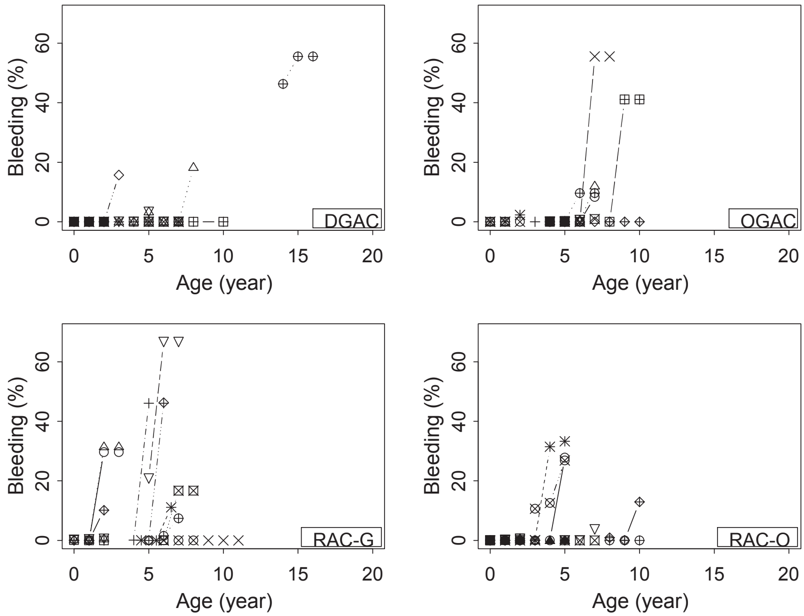

Figure 7. *Bleeding development trend in three years for various pavements*

It can be seen that transverse/reflective cracking generally propagates with pavement age. The transverse/reflective cracks seem to initiate earlier and propagates faster on the rubberized asphalt pavements (RAC-G and RAC-O) than on the nonrubberized pavements (DGAC and OGAC). This observation seems to suggest that use of rubber does not help prevent cracking. However, evidence from another study has indicated that the increased cracking in the rubber mixes may be biased by the condition of the underlying pavements, because RAC-G and RAC-O mixes tend to be placed more on pavements with greater extent of cracking (Lee et al. 2007).

Figure 9 shows the percentage of area with raveling (including all severity levels) in the three survey years. It can be seen that raveling may occur on all types of pavements, and in general, raveling starts earlier on DGAC and RAC-G pavements than on open-graded pavements. Pavements overlaid with DGAC mixes seem to experience more raveling than pavements overlaid with other mixes (OGAC, RAC-G, and RAC-O). Raveling in the two open-graded mixes tend to initiate and develop quickly after five years in service.

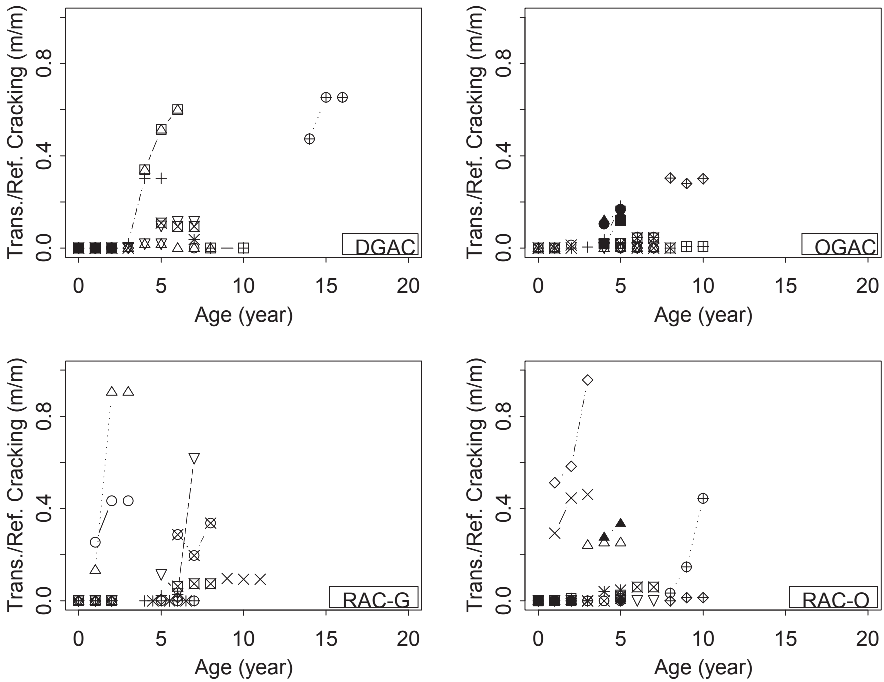

Figure 8. *Transverse/reflective cracking development trend in three years for various pavements*

In the condition survey, all the cracks in the wheelpath were recorded as fatigue cracks, because no data is available to determine whether they were caused by reflection. Fatigue cracking was evaluated as the areas of cracking at three severity levels (low, moderate and high).

Figure 10 shows the percentage of area with fatigue cracking (including all severity levels) in the three survey years for four pavement types. It can be seen from the plots that fatigue cracking may occur on all types of pavements, and in general, it increases with pavement age. Limited data indicate that fatigue cracking seems to initiate earlier on DGAC and RAC-G pavements than on open-graded pavements.

Multiple regression analysis shows that at the 95 percent confidence level, pavement age, existence of underlying PCC slabs and cumulative truck traffic are significant in affecting fatigue cracking. The estimated parameters indicate that fatigue cracking increases with pavement age and cumulative truck traffic. The existence of underlying PCC slabs increases the potential of fatigue cracking in the surface layer. Pavement type is an insignificant factor, indicating there is no significant difference in the fatigue performance of the four mix types.

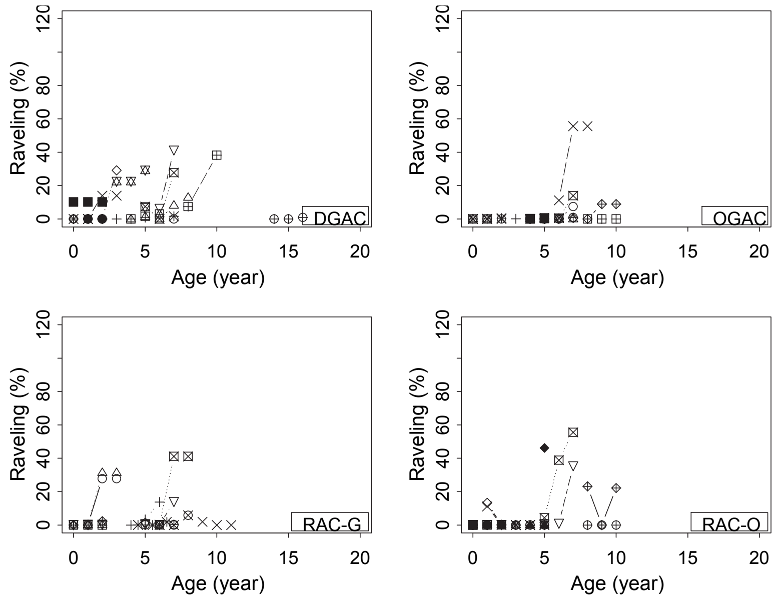

Figure 9. Raveling development trend in three years for various pavements

4. Conclusions

In this study, field data regarding tire/pavement noise, surface condition, ride quality, and macrotexture have been collected for three consecutive years from asphalt pavements placed with rubberized and nonrubberized mixes used in California. Analysis of the three-year data revealed the following findings:

- All four mixes investigated provide acceptable smoothness for riders. However, there is some evidence showing that inclusion of rubber in the mix can provide a pavement with smoother surface.

- The macrotexture of rubberized asphalt pavements is generally smaller, and less affected by age, than that of nonrubberized asphalt pavements.

- Compared to the average noise level of a DGAC mix, the newly placed asphalt mixes reduce the noise by about 2.3 dB(A) for OGAC, by about 2.9 dB(A) for RAC-O, and by about 1.6 dB(A) for RAC-G. After the pavements are exposed to traffic, this noise reduction benefit generally changes slightly for about five to seven years and then begins to lose quickly with pavement age. Inclusion of rubber in the open graded mixes tends to extend the noise-reducing property for two years longer. In other words, inclusion of asphalt rubber in the open-graded mixes can extend the noise-reducing duration.

- RAC-G mixes are initially quieter than DGAC mixes due to its higher initial permeability. After being opened to traffic, the permeability of RAC-G mixes reduce significantly to near zero values, and the noise level increases quickly to near the values measured on DGAC mixes.

- Bleeding seems to be a major distress type on asphalt rubber mixes, particularly on RAC-G mixes.

- Inclusion of rubber does not seem to change the ravelling potential of open-graded mixes.

- If all other conditions are the same, the study suggests that RAC-O mixes can provide longer noise reduction benefit than conventional OGAC mixes. However, the binder content of RAC-O needs to be selected carefully to prevent excess bleeding distress.

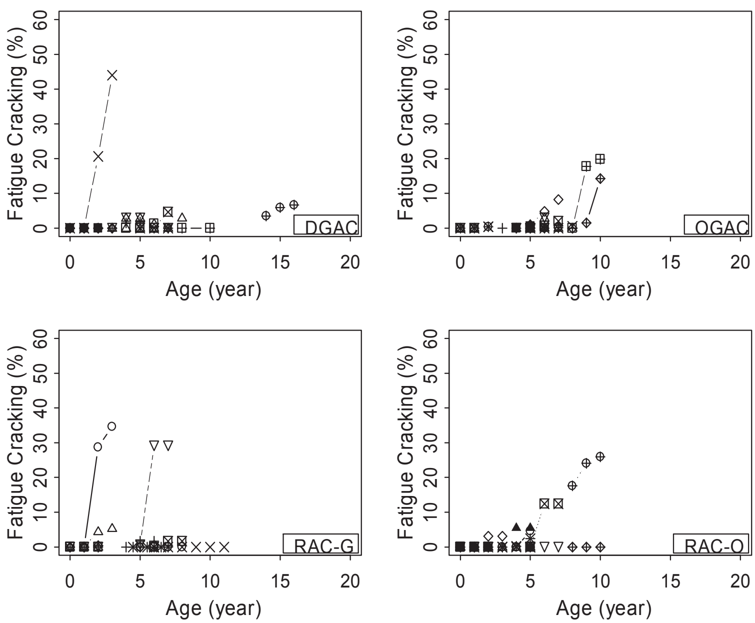

Figure 10. *Development trend of fatigue cracking in three years for each pavement section*

Acknowledgements

The authors would like to thank California Department of Transportation (Caltrans) for providing funding for this research. The opinions and conclusions expressed in this paper, however, are those of the authors and do not necessarily represent those of Caltrans.

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Environmental, Energy Consumption and CO_2 Aspects of Recycled Waste Tires Used in Asphalt-Rubber

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ABSTRACT: Waste tires are recognized as one of the most difficult waste products to manage in a modern society. They are not difficult individually, but are difficult collectively; particularly when world wide almost a billion such tires are disposed of annually. Waste tires are generated in industrialized societies at an annual rate equal to the human population which discards them, one scrap tire per person per year. The lack of adequate disposal methods and management systems in years past had lead to wide spread, cumbersome collection of waste tires in unmanaged or poorly managed waste tire piles. Problems associated with waste tire piles typically are: threat of fire and related environmental damage from a tire pile fire and the potential increase in vectors and pests. Secondary problems are that tire piles require substantial volume or space prior to any type of processing and are an eyesore. One of the uses of waste tires is to recycle them into asphalt-rubber. In this paper, the environmental, energy consumption and CO2 aspects of using ground tire rubber from waste tires in asphalt-rubber is considered and evaluated. Various environmental studies of asphalt-rubber are reviewed that demonstrate that asphalt-rubber is an environmentally acceptable paving material. Also, the end uses of ground tire rubber in terms of energy consumption from waste tires are analyzed in terms of: shredding for use in landfills as Alternate Daily Cover, shredding for use as tire derived fuel, and crumb rubber production with an end use in asphalt-rubber concrete pavements. This paper also touches upon a new aspect of environmental concern recognized in the Kyoto treaty and recently ruled upon by the United States Supreme Court namely CO2 emissions. The major goal of this paper is to investigate the overall benefit to Society for each aspect of the use of asphalt-rubber.

KEYWORDS: scrap tires, fuel, energy, asphalt rubber, Kyoto treaty

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1. Foreword

This paper is a continuation and updating of a paper presented at the AR2006 Conference (Sousa, 2006). Since 2006 there has become a greater environmental concern about green house gases namely CO_2. This was first recognized in the Kyoto treaty and more recently ruled upon by the United States Supreme Court in a decision where CO_2 was judged to be a pollutant.

2. Introduction

The disposal of scrap tires continues to be a major waste management issue. Approximately 300 million tires are disposed of annually each year in the USA. Tire piles can be a nuisance as well as health hazard because they are a breeding ground for mosquitoes and harbor various vermin. From time to time tire piles catch on fire and cause considerable environmental damage, Figure 1.

Figure 1. *The Wrong Type of Tire EnergyConsumption*

Because of the huge number of tires that accumulate annually they must be managed and processed in some way to prevent the build up of scrap tire piles, Figure 2. Many methods of disposal or end uses of scrap tires have evolved over the years, Figure 3. There are many broad categories of utilization. The burning of tires or derived fuel or tire derived fuel represents the single greatest usage of about 40 percent of all the waste tires. Landfill (Alternate Daily Cover) and civil engineering combined represent the next greatest amount representing about 24 percent of the waste tires. Ground rubber is about 10 percent of the use and is commonly used in AR.

Figure 2. *Scrap tires before processing*

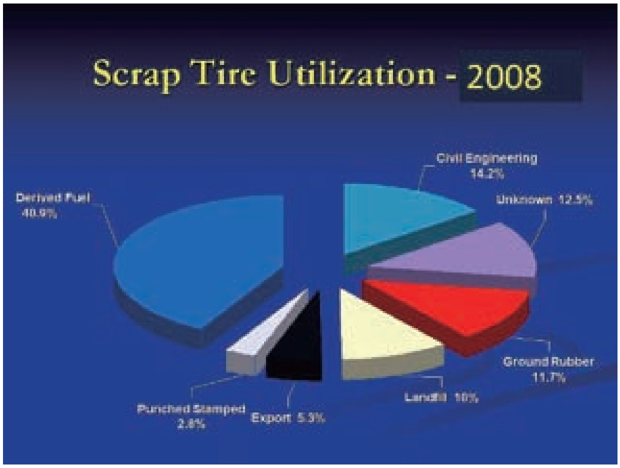

Figure 3. *Utilization of scrap tires in the USA*

3. Study Objectives

The objectives of this paper are to compare the energy consumption or saving of three common end uses of scrap tires and to estimate the degree of CO_2 reduction associated with use of asphalt rubber (AR). The three common end uses include shredding for use as Alternate Daily Cover (ADC) in landfills, shredding for use as a Tire Derived Fuel (TDF) in a combustion process and crumb rubber production with an end use in AR pavements. The scope of the energy consumption examination is to discuss the potential energy use or recovery benefits of each method. It should be noted that all three methods are currently in use and serve the intended purpose of removing scrap tires from the waste stream and represent about 74 percent of the consumption of waste tires. There are many methods of scrap tire disposal that can be used; these three were chosen to represent the range of options. Which method or a mix of methods used by a governmental entity to dispose of scrap tires is a function of many factors not necessarily just the potential energy recovery benefits. Nevertheless, using energy recovery benefits is a first start in judging the overall value of each method to society in general.

4. Background of Analysis

The technical approach taken in this paper is consistent with a study conducted by the Argonne National Laboratory in 1979 for the United States Department of Energy entitled "Discarded Tires: Energy Conservation Through Alternative Uses," (Gaines, 1979). At that time there was an energy crisis and the usefulness of tires as a fuel source was carefully examined. Also at that time waste disposal of tires was not an issue and air pollution regulations were not as strict as they are today. In light of these changes, and others such as potential global warming and green house gases namely CO_2 and the future of the Kyoto Treaty, which occurred over the past 21 years it seemed appropriate to again review this somewhat controversial topic in some detail.

5. Analysis

For each of the three disposal methods a combustion heat fuel scorecard was created. Many of the values were derived from the Argonne Laboratory study. Other values were obtained from various industry sources for aggregate, steel, hauling (trucking) and tire shredding and grinding. Table 1 is a list of typical heat combustion values for common fuels.

Table 1. Combustion Heat of Fuel

Fuel	kJ/kg heat value	BTU/lb heat value
Coal	26000	11000
Tire	35000	15000
Asphalt	35000	15000
Natural Gas	172000	74000
Propane	214000	92000
Gasoline	233000	100000
Diesel	319000	137000

In this study scrap tires and asphalt have the same heat of combustion value which is slightly greater than the heat of combustion value of coal. As can be seen coal, scrap rubber and asphalt are all at the low end of heat value. Presently, modern power generating plants typically use natural gas as much as possible to generate electricity and meet very demanding air pollution requirements.

6. Alternate Daily Cover

The first disposal method for scrap tires that is analyzed is called Alternate Daily Cover (ADC). ADC involves the placement of rubber tire shreds generally about 150 mm (6 in) square or larger being placed in a landfill to cover the daily refuse pile or layer to a depth of 150 mm (6 in), or as a light weight civil engineering fill, Figure 4 and 5.

Figure 4. *Tire shreds for alternate daily cover*

Figure 5. *Tire shreds being placed as cover or fill*

This process requires the least amount of energy of the three options. Table 2 is the heat of combustion values for ADC.

Table 2. *Utilization for Alternate Daily Cover*

Process	kJ/kg heat value	BTU/lb heat value
Tire Shredding	-93	-40
Shred Transportation	-1744	-750
Gain/Loss	-1837	-790

ADC is composed of scrap tires that have been shredded into approximately 100-150 mm (4-6 in.) square tiles that are spread to a depth of 150 mm (6 in.) atop a sanitation landfill pile at the end of each day. Regional specifications can vary on the shred size and layer depth. This lightweight cover keeps loose material from blowing away. There is no net energy benefit since it takes energy to shred the tires and transport the shreds and place them. The net negative use of energy is small and under the right circumstances may be an appropriate use of shredded tires.

7. Tire Derived Fuel

TDF is composed of whole scrap tires or shredded tires that are introduced into a coal fired furnace to add extra heat, Figure 6.

Figure 6. *Tire derived fuel whole tires combusted in coal fired cement kiln*

Table 3 is an example of the heat of combustion values for TDF. Tire chips can combust with fewer emissions than coal. In many locations, tire chips are used to help reduce the total emissions output. There is a net positive gain in energy for the rubber used. This is a good use of scrap tires and presently consumes about 120 million tires in the United States (Scrap, 2000).

Table 3. *Utilization for Tire Derived Fuel*

Process	kJ/kg heat value	BTU/lb heat value
Tire Shredding	-1740	-750
Shred Transportation	-1740	-750
Combustion Energy	+34900	+15000
Gain/Loss	+31420	+13500

8. Asphalt Rubber

AR is composed of crumb rubber derived from the commutation of scrap tires. Table 4 represents the heat of combustion values for crumb rubber modifier (CRM) used in A-R, Figure 7.

Figure7. *Typical asphalt rubber mix paving*

Table 4. *kJ/kg Utilization for Asphalt rubber*

Process	kJ/kg heat value	BTU/lb heat value
Tire Shredding	-1740	-750
Shred Transportation	-1740	-750
Granulation	-3600	-1540
Crumb Rubber Transport	-1740	-750
Steel Recovery	+1900	+820
Asphalt Saved	+209000 to +465000	+90000 to +200000
Aggregate Saved	+108000	+46000
Gain/Loss	+310080 to +566080	+133030 to +243030

The crumb rubber is the size of coffee grounds and is derived from either commutation by mechanical grinding, commonly called ambient grinding or from cryogenic commutation using liquid nitrogen, commonly called cryogenic grinding, Figure 7. Modern plants often employ a combination of both cryogenic and ambient technologies to obtain the most economical product. The crumb rubber is combined with liquid asphalt and then combined with aggregate materials and placed using conventional paving equipment. As this table shows there is a net positive gain in energy for the rubber used.

Figure 7. *Crumb Rubber after grinding to a 10 mesh or finer*

The 310080 kJ/kg (133030 BTU/lb) value is consistent with the previous Argonne Laboratory finding in 1979. In 1979 the Argonne Laboratory derived their value by examining the use of an AR chip seal and assigning its energy savings in terms of less asphalt concrete overlays would be needed over the life of the pavement. Since 1979, AR is now commonly used as a binder in hot mixes in the states of Arizona, California, Texas and Florida. Energy savings are now the result of using less than one half the thickness of routine paving material as reported by Arizona (Way, 2000) and California (Van, 2000). The 310080 kJ/kg (133030 BTU/lb) energy savings refers to a two inch A-R overlay being used in place of a normal four inch asphalt pavement overlay. The 566080 kJ/kg (243030 BTU/lb) energy savings refers to an one inch open graded AR mix being placed on top of a concrete pavement in place of a normal five inch asphalt pavement overlay.

Other energy savings that have occurred since 1979 include aggregate savings. In many parts of the United States and Europe good quality road building aggregate is in short supply and harder to obtain. The 108000 kJ/kg (46000 BTU/lb) energy savings refers to the mining energy and transport energy associated with using thicker pavements compared to the thinner AR pavements. The reclaiming of steel from tires also has considerable value not recognized in 1979. In all the energy savings by using AR is very impressive. These energy savings coupled with other AR benefits including less cracking, less maintenance and less noise (Bollard, 1999) make this a very attractive and beneficial end use of scrap tires in a highway environment. Life cycle cost analysis encouraged by the Federal Highway Administration has shown that a substantial dollar savings can be obtained over the expected life of a project when AR paving strategies are employed (Hicks, 2000).

9. Green House Gas CO2

Since 2006 there has become a greater environmental concern about green house gases namely CO_2. This was first recognized in the Kyoto treaty and more recently ruled upon by the United States Supreme Court in a decision where CO_2 was judged to be a pollutant, Figure 8 (US, 2007).

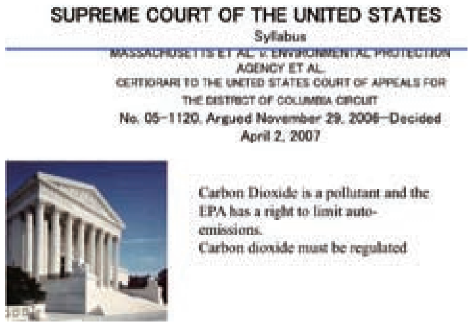

Figure 8. *US Supreme Court ruling on CO_2 as a pollutant*

In examining the previous savings in energy it is possible to estimate how much CO_2 can be saved by using a thinner AR pavement. Very large amounts of CO_2 can be saved by using AR pavements which could be useful in future years as the importance of reducing CO_2 becomes more significant.

Table5. *CO_2 Savings with AR 2 to 1 Reduced Thickness of Pavement*

Use of 50 mm (2 in.) overlay compared to 100 mm (4 in.) overlay	**SI Units Lane km**	**US Customary Lane mile**
Crumb Rubber kJ/kg or BTU/lb	310080 kJ/kg	133030 BTU/lb
Crumb Rubber/lane km or mile	5455 kg	20000 lbs
AR Energy/lane km or mile	1691362368 kJ	2660600000 BTU
Energy/ Liter or Gallon of Gasoline	0.0000076 kJ/Liter	0.000008 BTU/Gallon
Equivalent Liters or Gallons Saved	12800 Liters	21285 Gallons
CO_2 Saved/ kg or lbs	10 kg	22 lbs
Kg or Lbs of CO_2 Saved	128000 kg	468270 lbs
Tons of CO_2 Saved per lane km or mile	128 tonnes	234 tons

10. Possible long term energy and CO_2 savings over time

In addition to the energy and CO_2 savings associated with the construction of thinner AR pavements previous studies have shown that AR pavements can be smoother over time (Way, 2009) as shown in Figure 9. All other things being equal smoother pavements mean less energy consumption and therefore less CO_2 emissions. As can be seen in Figure 9 after the sixth year of service AR pavements are consistently smoother than conventional HMA pavements by about 200 mm per km. The World Bank has developed models to relate energy savings versus the smoothness of the pavement (World, 2001). Using their empirical relationship the average savings in fuel for a passenger car for the 6 to 15 year period would be approximately 0.08409 liters per 1000 km (0.03575 gallons per 1000 miles). Just as a theoretical example of what this means 100000 cars traveling 10000 miles a year over the nine year period of smoothness performance (year six to fifteen) the AR pavements would save a total of about 300000 gallons of gasoline. In addition the total CO_2 savings over that nine year period would be 3300 tons of CO_2.

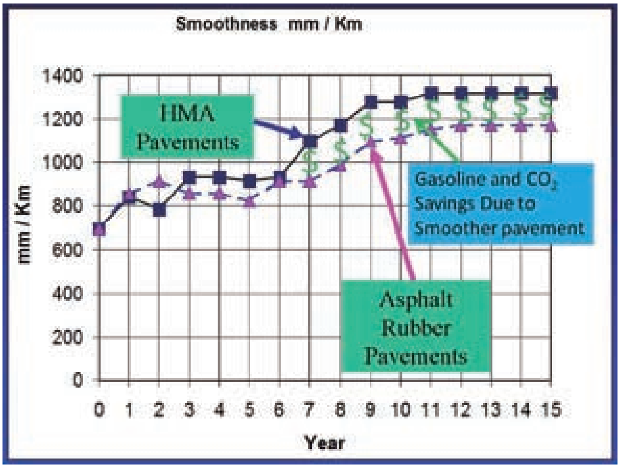

Figure 9. *Pavement smoothness in mm/Km versus years of service for HMA and asphalt-rubber pavements*

11. Discussion

The above academic exercise demonstrates the wide range of energy usefulness that scrap tires have to offer society in general but what about the practical side of the issue? With this in mind, experiences in the State of Arizona may be of value in discussing the pros and cons of uses of the methods previously discussed. In the mid 1970's tires were combusted in copper smelters in Arizona. As air pollution laws changed Copper smelters found it more difficult to operate in Arizona and thus by the mid 1980's all copper smelters were shut down in Arizona. Copper from Arizona mines is now smelted in Mexico. In the mid 1980's cement plants burned tires. Due to environmental concerns the cement plants decided to end the burning of tires even though such burning can be legally permitted. Thus by 1990 no tires were being burned in Arizona. Coincidentally the Arizona Department of Transportation (ADOT) along with cities and counties in the state began to routinely use AR as an engineered binder in pavements in 1988. Since that time ADOT alone has used over 10 million tires in pavements. There was and is no special program to reuse tires in pavements in Arizona, however the State law does encourage recycling of tires as the highest priority. Approximately 70 percent of the scrap tires in Arizona now go into pavements, with the remainder going into various commercial products.

11. Conclusion

Scrap tires can be used in many ways. It is important to Society to encourage their use and removal from the waste stream. The three processes discussed meet society's need of preventing tire piles from accumulating and exposing the ecosystem to unnecessary risks of increased pollution and pests. However, the potential energy used or saved in tire processing should also be examined. Table 6 summarizes the energy values of the three tire processes discussed in the paper.

Table 6. *Comparison for the three scrap tire disposal methods*

Tire Derived Material	kJ Gain/loss/kg Rubber	BTU Gain/loss/lb Rubber
Alternate Daily Cover	-1837	-790
Tire Derived Fuel	+31420	+13500
Crumb Rubber Modifier In Asphalt Rubber	+310080 to 566080	+133030 to +243030

Besides the potential energy savings gained by using granulated tire rubber as a modifier to asphalt pavement, it should be noted that this process can substantially improve the highway assets maintained by our communities.

12. Acknowledgment

The authors express their appreciation to the following for their input, contributions and research assistance:

Don Stout of FNF Construction, Inc.
Eric J. Stuart of the Steel Manufacturers Association
Michael Blumenthal of the Scrap Tire Management Council
Jim Anderson of Recovery Technologies Group
Muarry Quance of BAS Recycling

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Promoting Asphalt Rubber Application through Education

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ABSTRACT: There are about 300 millions waste tires generated each year in the United States. In the state of California alone, more than 40 million waste tires are generated per year. Waste tires take up valuable landfill space and cause significant negative environmental impact. Waste tires can be recycled as asphalt rubber to be used in rubberized hot mix asphalt concrete. Rubberized hot mix asphalt concrete has been used in California for more than 20 years and it has been proven to be a very cost effective paving material when applied properly. However, when using recycled materials in real applications, one faces many challenges, especially, if the knowledge of how to use the recycled materials such as waste tires has not been well disseminated. To overcome these challenges and barriers, teaching materials have been developed to educate university students and professionals about asphalt rubber usages. The teaching modules developed in this study have been taught in real university environments. Several professor training workshops on using asphalt rubber have been given in California of the United States and the training materials were stored on a website. Researchers can use the materials as a source of information while practitioners can use them as guidelines.

Keywords: Asphalt Rubber, Teaching Materials, Waste Tire

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1. Background

Waste tires can occupy valuable landfill space. Waste tires in stockpiles, legally or illegally dumped, may produce tire fires that are very hard to put out, and cause significant public health and environmental concerns. There are approximately 300 million waste tires generated each year in the United States. Today, most of the tires are reutilized as tire derived fuels, in civil engineering applications, or in rubberized asphalt concrete in roadway pavements. There are many benefits of using asphalt rubber in pavements. Asphalt rubber can reduce reflective cracking and improve fatigue resistance; increase resistance to bleeding, flushing, drain-down problems; allow higher binder content and increased film thickness. The open-graded rubberized asphalt can also reduce noise and splash and spray due to rain. Even though asphalt rubber is a good product for asphalt pavements, it doesn't mean that people can apply asphalt rubber freely. If engineers or contractors don't have the knowledge on how to utilize the material, it can fail rapidly. For example, a segment of freeway on Interstate highway 5 (I-5) in northern California used open graded rubberized hot mix asphalt inappropriately in 2007. The pavement failed within half year. Another example is a failed asphalt rubber overlay project in the city of Rohnert Park, California. Many segments of the project were replaced within one year. Therefore, it is extremely important to have engineers and contractors educated on using asphalt rubber. This is one of the roles of the California Integrated Waste Management Board (CIWMB) and this project in particular.

California has many different geologic features, including mountains, desert, valleys, and coastline, and the climates of these regions are quite different. These diverse geographic and climate conditions make California an ideal place to develop and test asphalt rubber paving materials. California has spent significant efforts to study the usage of asphalt rubber in pavements from laboratory performance tests, Heavy Vehicle Simulator studies, to numerous field investigations. California has developed useful guidelines and standards for asphalt rubber, such as Asphalt Rubber Design and Construction Guidelines (*Hicks 2002*), and Caltrans Asphalt Rubber Usage Guide (Caltrans 2003).

This study is part of a bigger project about develop teaching materials for university students on how to utilize waste tires in all areas of civil engineering applications, including transportation, structural, geotechnical and environmental engineering. In this study, training and teaching materials on utilizing asphalt rubber based on California experience have been developed. The developed teaching materials contained the history and long term performance of asphalt rubber, included case studies, and presented the ASTM standards, the latest manuals, and specifications used in California.

2. Objectives

The objectives of this research are to:

- synthesize the knowledge of utilizing waste tires in asphalt rubber applications
- develop effective teaching materials to educate university students about utilizing waste tire products in pavement

- promote sustainability by using waste tires in asphalt rubber applications through university education

The goal of this paper is to summarize the curricula of asphalt rubber civil engineering applications to make people aware of the existence of the course materials and resources developed on this project.

3. Approach and Proposed Curricula

Utilizing waste tires in civil engineering applications is a multi-disciplinary and complex subject. No single class currently available in civil engineering can cover all the aspects of it. To reach university students more effectively, we developed teaching modules for different grade levels for use in related civil engineering classes. As illustrated in Figure 1, it was proposed to develop waste tire application teaching modules for a variety of civil engineering courses from freshman level to senior level. Asphalt rubber applications are covered under introduction to civil engineering design, transportation engineering and asphalt paving materials classes, and contracts and specifications.

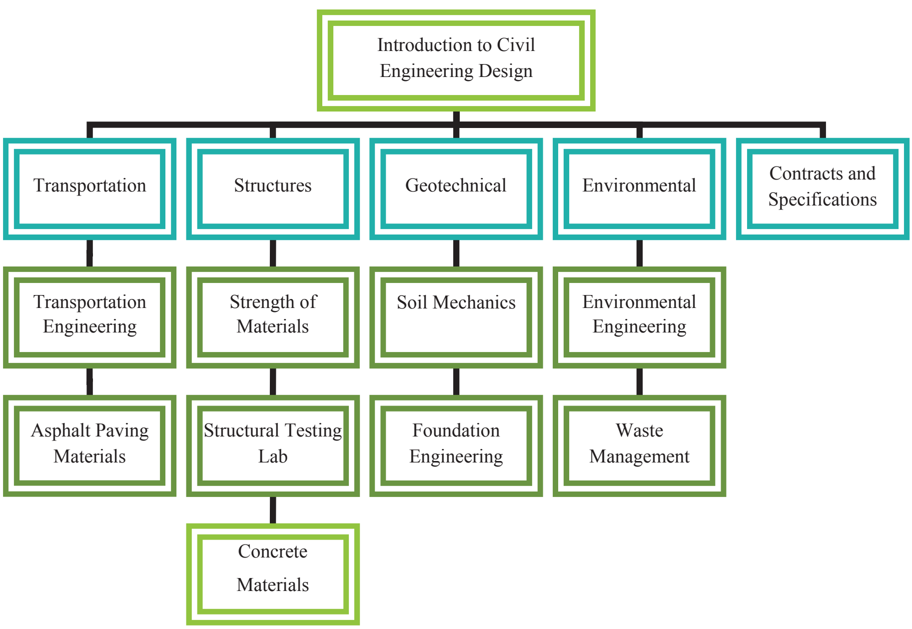

Figure 1. *Roadmap for Waste Tire Applications in University Curricula*

By offering teaching modules for waste tire applications at different class levels starting from freshmen level, more students can be reached than would be by a single elective class on utilizing asphalt rubber , for example, like a senior asphalt pavement class. By the end of university education, a student may be exposed to asphalt rubber applications multiple times.

It was deemed as a more effective way of teaching students about unconventional materials, such as waste tire derived materials.

4. Curricula Development

A series of course modules about asphalt rubber have been developed for a variety of undergraduate Civil Engineering courses, including Introduction to civil engineering design, transportation engineering, and asphalt pavement materials classes.

4.1 Introduction to Civil Engineering Design

The goal of this lecture is to introduce university students to waste tire materials and give them an overview of utilizing waste tire products in a variety of types of civil engineering applications. It is important for students to understand the significance of utilizing recycled materials to preserve valuable natural resources. Students should also understand the significance of protecting the environment and they should also learn to promote healthy and sustainable development of our society. Figure 2 shows a picture of the Westley tire fire in the State of California.

The lecture module also covers the benefits and challenges of using waste tire derived products in civil and transportation engineering applications. In civil engineering, it discusses physical properties of waste tire derived aggregate (TDA) such as TDA as backfill materials for retaining walls and bridge abutments, lightweight fill for embankments, insulation layer for roadway base, and vibration damping materials for rail lines.

In transportation, it introduces the rubberized hot mix asphalt for pavement. It describes the benefits of using asphalt rubber; including prolonging paving life, reducing maintenance costs and noise, improve safety, as well as environmental benefit of getting rid of waste tires. It also presents a roadmap of civil engineering classes that cover waste tire applications.

FIGURE 2. *Tire Fire in Westley California in September 1999 (Earth Link 2008)*

4.2 Transportation Engineering

The goal of the transportation engineering lectures is to inform students of the history, benefits, limitations and practice of using asphalt rubber (AR) as a paving material. The major references of this class are the Caltrans Asphalt Rubber Usage Guide (Caltrans 2003), Caltrans Highway Design Manual (Caltrans 2008), and Caltrans Maintenance Technical Advisory Guide (Caltrans 2008). These lectures are divided into four modules, each dealing with a different aspect of asphalt rubber applications.

The students are first introduced to the module of the history of using asphalt rubber as a paving material. Case studies of full scale AR overlay projects in California are presented. These studies outline the strategy of using AR as an overlay to repair existing distressed pavements, as well as discussing the design and results of the AR overlays. The benefits of using AR pavements as a replacement for conventional asphalt are also discussed.

The second module introduces the structural design of AR pavements. A 2005 Caltrans study is referenced in this module to review the revised practices of using AR in new pavements as well as an overlay (Caltrans 2005). Students are informed on the recommended design strategies for new pavements and overlays using AR. An overview of the revised practices for using AR in overlays and new pavement is also presented. This module also presents cost analysis comparing AR and conventional asphalt. RHMA can be a cost effective option when a reduce thickness design is warranted for preventing reflective cracking of the existing pavement. Table 1 illustrates the Caltrans practice on reduce thickness on overlay design based on reflective cracking.

Table 1. *Caltrans Practice on Reduce Overlay Thickness Conversion Table for crack retardation (Caltrans 2008)*

HMA (mm)	RHMA-G (mm)	RHMA-G over SAMI-R (mm)
45	30	N/A
60	30	N/A
75	45	N/A
90	45	N/A
105	45 if crack width < 3 mm 60 if crack width ≥ 3 mm or underlying material CTB, LCB, or rigid pavement	N/A for crack width < 3 mm 30 if crack width ≥ 3 mm and underlying material untreated 45 if crack width ≥ 3 mm and underlying material CTB, LCB, or rigid pavement
135	45 over 45 HMA	60

Students are next introduced to the manufacturing and construction process of AR in the third module. The module discusses the general paving process with an emphasis on the different practices between AR and conventional asphalt. An overview of the manufacturing process informs students how AR is produced and also highlights the operational differences when dealing with AR such as the laydown and compaction temperatures for successful placement of AR.

The last module of the lecture goes into detail about AR binder production, AR mix production, inspection of paving and troubleshooting. Some or all of these modules could be included in transportation and pavement engineering classes.

4.3 Asphalt Paving Materials

This module consists of asphalt rubber (AR) binder design, the different types of AR mixes and cautions for using AR. The lecture defines the different types of asphalt rubber binders and discusses how each type is produced. Crumb Rubber Modifiers (CRM) are the form of waste tires added to the binder. The interaction between the CRM, the asphalt and the affecting factors are explained. When designing an AR blend, it is necessary to develop a binder profile which evaluates the compatibility, interaction, and stability between materials over a period of time.

The students are introduced to the most commonly used types of rubberized hot mix asphalt concrete, including Rubberized Hot Mix Asphalt – Gap graded (RHMA-G), Rubberized Hot Mix Asphalt – Open graded (RHMA-O), and Rubberized Hot Mix Asphalt – Open graded – High Binder content (RHMA-O-HB). The mix design, advantages, and standard specifications are described for each rubberized asphalt mixture type. Figure 3 displays that advantage of RHMA-O to reduce water splashing effect over traditional Dense Graded Asphalt Concrete (DGAC). Both field blend asphalt rubber and terminal blended asphalt rubber are covered in this module.

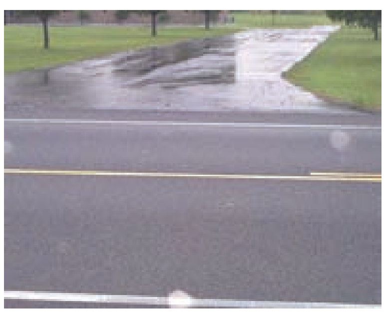

Figure 3. *Free-Draining RHMA-O Next To DGAC (Caltrans 2003)*

4.4 Contracts and Specifications

This lecture has two modules. One is on ASTM international standards; the other is for specifications on rubberized hot mix asphalt. A series of ASTM standards related to waste tire applications are covered. The major ones are: ASTM D8 (ASTM 2002), which defines the terminology of asphalt rubber; and ASTM D6114 (ASTM 2002), which gives the standard specifications of the asphalt rubber binder, and ASTM D6270 (ASTM 2004)., which has detailed definition on tire rubber, material characterization, usage, construction practices, guideline for fills, and leachate. The lecture also provides students the necessary background on the ASTM International.

The specification lecture starts with various types and aspects of specifications. As examples, standard specifications were illustrated using Caltrans standard specification on RHMA – O (open graded rubberized hot mix asphalt) and RHMA – G (gap graded rubberized hot mix asphalt) (Caltrans 2007).

5. Outcomes

Teaching materials for utilizing waste tire products in civil engineering applications have been developed for ten different civil engineering courses. The teaching materials are available on a website hosted by CSU, Chico. All these lectures have been taught at the undergraduate level at California State University, Chico in a variety of civil engineering courses. Students have greatly improved their knowledge on utilizing waste tire products in civil engineering applications. They were able to demonstrate their knowledge of, and interest in waste tire applications through their term projects, lab reports, presentations, and homework assignments.

A website was created to store the teaching materials, which includes PowerPoint presentations, lecture notes, sample assignments and sample solutions and student work. The link for the website is:

http://www.ecst.csuchico.edu/cp2c/dxcheng/Curricula/CIWMBEducation.php

Professors and instructors can easily access the teaching materials by logging onto the website. If you log in as a professor, you will be able to use all of the teaching materials. The public can also access part of the teaching materials by logging in as a guest. Generally, they can only access the PDF versions of the presentations. A snapshot of the webpage for professors is shown as Figure 4.

These course materials are available to be integrated into various courses in the undergraduate Civil Engineering curriculum and serve to introduce students to using asphalt rubber in pavement engineering.

Three professor training workshops have been conducted in California. Over 40 professors from 15 different universities attended the training. The general feedback from the professors was very positive. They found the teaching materials to be very interesting and useful. Utilizing waste tires as asphalt rubber in pavement fits into the sustainability and green construction practices. It is a much needed part of education for the 21st century engineers.

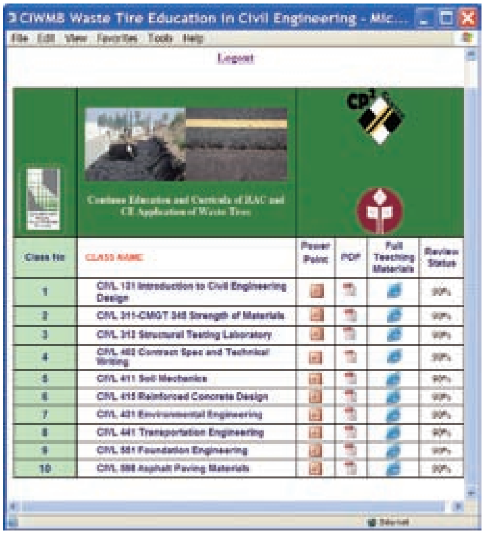

Figure 4. *Sample Webpage to Access Teaching Materials for Waste Tire Applications*

6. Conclusions

Each year, more than 60 millions waste tires are consumed by civil engineering applications and ground tire rubber usages, including asphalt rubber (RMA 2006). In order to promote the beneficial usage of waste tires in civil and transportation engineering, educational curricula, including a series of lectures for undergraduate courses were developed. The following conclusions can be drawn from the curricula development project:

- Waste tire applications, especially asphalt rubber, has beneficial properties that engineers can use. The asphalt rubber can improve the safety and performance roadway. It can be economical too when it is used at right conditions.

- Teaching modules or lecturing materials were developed to cover freshman level to senior level classes for asphalt rubber. The freshman class gives students an introduction and overview of asphalt rubber and their applications. Junior classes cover the material properties, testing, and standard specifications. Senior classes can cover asphalt rubber applications in pavement engineering.

- Outcomes show that introduction of curricula in a variety of courses is an effective way to teach asphalt rubber and can reach more students. Students have demonstrated a knowledge of and interest in the sustainable use of asphalt rubber through their school work.

Students in civil engineering are the future engineers and their knowledge of asphalt rubber will affect the sustainable usage of recycled materials such as waste tires. It was a good experience promoting the education of the sustainable usage of recycled waste tires in civil and transportation engineering by developing teaching materials. The education on the use of other recycled materials can follow a similar approach.

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Analysis of Environmental Sustainability in The Rehabilitation of Existing Pavements Using Asphalt Rubber Hot Mixes

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ABSTRACT. Asphalt Rubber is by definition a green technology, not only because it uses a recycled material but also because its mechanical performance allows pavement thickness reduction, saving natural resources and money and reducing CO_2 emissions.

This paper quantifies the main parameters connected to the study of the environmental sustainability related to the production and laying of AR hot mixes.

After a brief summary of the state of AR technology in Italy, results of a quantitative analysis of environmental benefits arising from the use of this technology are shown in terms of energy balance, highlighting the significant savings achieved under construction, in addition to the benefits of longer duration and less maintenance required for AR pavements.

Finally, it is reported, as a study case, the set of improvements solutions adopted for the rehabilitation of the Florence - Pisa – Livorno highway, where the choice of using AR technology as a rehabilitative solution was made after a detailed analysis of the environmental benefits.

KEYWORDS: Asphalt Rubber, Wet process, Energy saving, GHG saving, CO_2 emissions reduction.

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1. Introduction

Noise reduction, safety, durability, shorter construction time, lower maintenance costs, lower CO_2 emissions, higher cost benefit ratio and natural resources saving: the list of benefits connected to the use of Asphalt Rubber technology exceeds the most innovative material.

The present paper focuses the environmental sustainability of this technology, since nowadays environmental factors have an increasingly decisive role in the choice of road materials.

By re-using common waste products such as rubber from old tires it is undeniable that there are numerous environmental advantages in saving energy and natural resources. Also, experience has shown that by properly combining the waste product of ground tire rubber with asphalt at high temperatures the resultant binder will make a hot mix with superior engineering properties, including reduced fatigue and reflection cracking, greater resistance to rutting, improved aging and oxidation resistance and better chip retention due to thick films [1 - 3] . Plus, Asphalt Rubber pavements have demonstrated to have lower maintenance costs, higher noise absorption, reduced splash and spray and better night-time visibility due to contrast between pavement and striping [2, 5].

From above it can be deduced that environmental benefits of AR technology are connected to several aspects during the entire pavement life cycle:

- On one side, the use of AR allows significant reduce in thickness and increases pavement life (with lower maintenance), allowing a significant saving of raw materials, saving energy during hot mix production, transport and laying, and reducing emissions of pollutants and greenhouse gases;

- On the other side, there is the recycling of old tires, the fact that AR pavements present more regular surface, which reduces consumption of vehicles and related emissions. AR friction course and open graded mixes can actually reduce noise, which can avoid the current environmental impact of sound-absorbing panels. Moreover, AR is a recyclable hot mix, whose production does not produce more fumes than traditional solutions [3].

2. The state of Asphalt Rubber technology in Italy

While in other European countries crumb rubber modified hot mixes are now commonly used, in Italy, AR technology is still considered to all affects a new technology and most of old tires are destined for burning, especially in cement industry.

The fact that the European Community since 2006 prohibits the accumulation in landfill of used tires, has revived the interest for their possible recycling in road pavements and several agencies have started to recommend the use of AR as a sustainable engineering solution.

With regard to the regulatory developments that are following the technological evolution of AR in Italy, Asphalt Rubber binder (wet method) is part of the environmental directive: “Information concerning the recycled materials and goods and articles made with recycled materials from rubber”- as the DM 203/2003 - G.U. No 173, 27 July 2005, which encourage the reuse of recycled materials in new constructions.

Since the first applications in the Italian network, in the end of 2006, AR pavements have been studied and monitored and environmental expectations by increasing the application of AR in Italy confirm the aspects introduced in the preceding paragraph.

Regarding noise, tests have demonstrated that an AR Open graded surface leads to a reduction, depending on traffic speed, of: -3 dB for speeds <50 km/h, 4 dB for speeds between 50 and 90 km/h, 6 dB over the 90 km/h [11]. Since a noise barrier is able to obtain a reduction of about 1 dB for every 60 cm in height, it’s possible to reduce or even eliminate them, with a substantial reduction of environmental impact. These considerations, already tested and proven in the U.S. and Europe, have been measured in Italy: the CIRS (Centro Interuniversitario di Ricerca Sperimentale Stradale) measured a reduction of 4.5 dB for an average speed of 40 km/h. This result was confirmed by analysis conducted by local Environmental Agency (ARPA of Tuscany), which has led to define a sound absorption coefficient equal to 0.75 for AR Open graded hot mix [11].

3. Energy quantification of environmental benefits using Asphalt Rubber hot mixes for the rehabilitation of existing pavements

To actually quantify some of the environmental benefits listed in the introduction, it is presented the environmental improvements in terms of energy savings achievable through the application of AR hot mixes, compared to traditional hot mixes.

The AR design solutions allow an immediate benefit in terms of energy saving, due to the reduction of raw materials and related processes necessary during the construction phase, besides the advantage due to the longer life of AR pavements. Since this second type of benefit has already been examined in several studies, the present study only quantifies the savings during construction phase, in itself enough to convince of the environmental validity of AR solutions.

In order to illustrate energy and CO_2 emission savings, the study quantifies the energy balance of an entire rehabilitation process of a road pavement using AR hot mixes.

During the rehabilitation process it’s assumed that AR hot mixes (gap graded or open graded) are placed at approximate half the thickness of conventional hot mixes. This assumption is, anyway, conservative [1].

3.1 Overall energy balance during the rehabilitation process

In this paragraph it’s shown the analytical energy balance of the yard. The whole construction process can be divided into the following phases:

- AR hot mixes production;
- Hot mixes transport;
- Milling;
- Stabilization and recycling;
- Transport for disposal;
- New pavement construction.

For each of the steps listed, consumptions and energy resources and materials are detailed, taking as reference a pavement rehabilitation with 3 cm AR Open Graded (with nominal maximum aggregate size of 12.5 mm), 4 cm AR Gap Graded (with nominal maximum aggregate size of 14 mm), SAMI in Asphalt Rubber and 30 cm recycling with foamed bitumen. This type of pavement structure as been applied several times in Italy.

The following charts illustrate each aspect of the process, highlighting their energy consumption per m³ of AR hot mix.

- Hot mixes production: the known characteristics of the AR hot mixes plant, equipped with a two stage gas burner (580 kW), allows to determine the energy consumption associated with production;
- Hot mixes transport: it depends on the distance between production plant and yard; assuming an average distance equal to 50 km it's possible to define the energy consumption associated with transportation;
- Cold milling: knowing the consumption of a typical cold milling machine as a function of milling thickness (as shown in Figure 1), it's possible to determine the total consumption of fuel and, therefore, the equivalent energy consumption and CO_2 emissions;

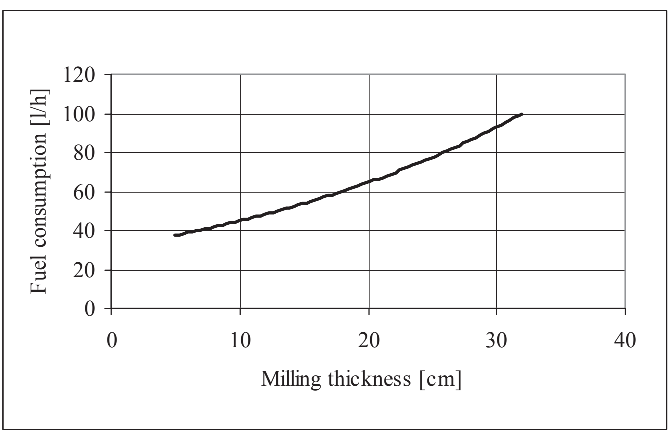


Figure 1. *Typical fuel consumption function of a cold milling machine*

- Stabilization and recycling: as for the milling machine, knowing the fuel consumption as a function of operating conditions, depending on the thickness to be recycled and stabilized it's possible to find the total energy consumption and associated CO_2 emissions;

- Disposal of resulting materials: materials milled and recycled are collected and transported away from the yard. Consumptions and emissions are calculated considering an average route equal to about 10 km, and taking into account the 15% increase in average volume that occurs between the stage of milling and the subsequent filling and stabilization;

- New pavement laying: in this phase, the energy balance has taken into account the power of every machine and the total use of each of them.

The results obtained are summarized in Table 1 and Figure 2.

Table 1. *Energy balance of the yard per unit of AR hot mixes*

Process	Energy consumption [kWh/m³]	CO_2 emissions [kg CO_2/m³]
AR hot mixes production	6.206	3.600
Hot mixes transport	13.185	7.647
Milling	17.651	10.237
Stabilization and recycling	19.214	11.144
Transport for disposal	1.953	1.133
Hot mixes laying	2.960	1.717
TOTAL	61.169	35.478

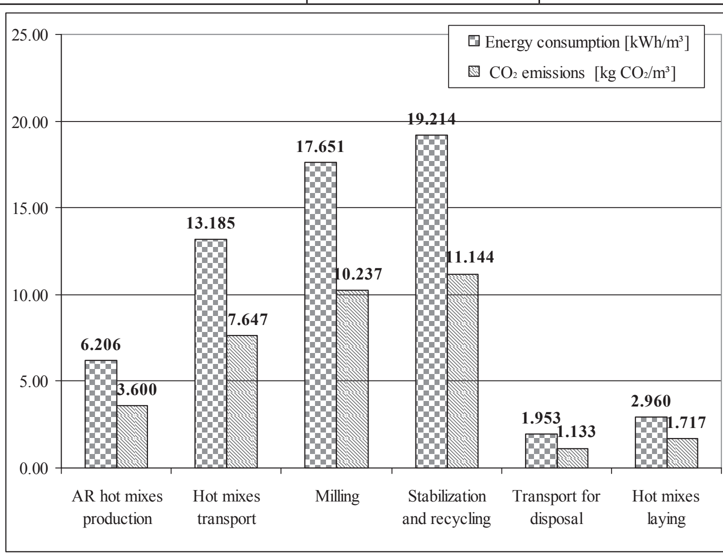


Figure 2. *Energy balance per unit of AR hot mixes*

For the entire rehabilitation process, therefore, energy consumption amount to 61.169 kWh for each m³ of AR hot mixes finally obtained, and the corresponding value of CO_2 emissions is equal to 35.478 kg CO_2/m³.

3.2 Energy saving associated to AR solutions for pavement rehabilitation

The main items that, during pavement rehabilitation, conduce to a substantial energy saving of AR applications compared with traditional solutions are:

- Reduced hot mixes production: this affects both in terms of production and transportation. Production was estimated assuming a plant with a 580 kW gas burner. Transport is estimated assuming, again, a 50 km average distance between plant and construction site;

- Reduction of milling thickness: the reduction of thickness presents the double advantage of reducing, at the same time, milling process and the amount of resulting materials. In order to calculate energy saving during milling, it was quantified the fuel saving analyzing the relative curve of consumption (as shown in figure 1). To taking into account the energy saving associated to disposal needing, it was considered a 10 km average distance from construction site and disposal landfill.

Results obtained are summarized in Table 2 and Figure 3.

Table 2. *Energy savings achieved for each m³ of AR hot mixes*

Process	Energy saving [kWh/m³]	CO_2 Emission saving [kg CO_2/m³]
Hot mixes production	7.378	4.279
Hot mixes transport	15.674	9.091
Hot mixes laying	2.960	1.717
Milling	1.873	1.086
Transport for disposal	0.766	0.444
TOTAL	28.651	16.617

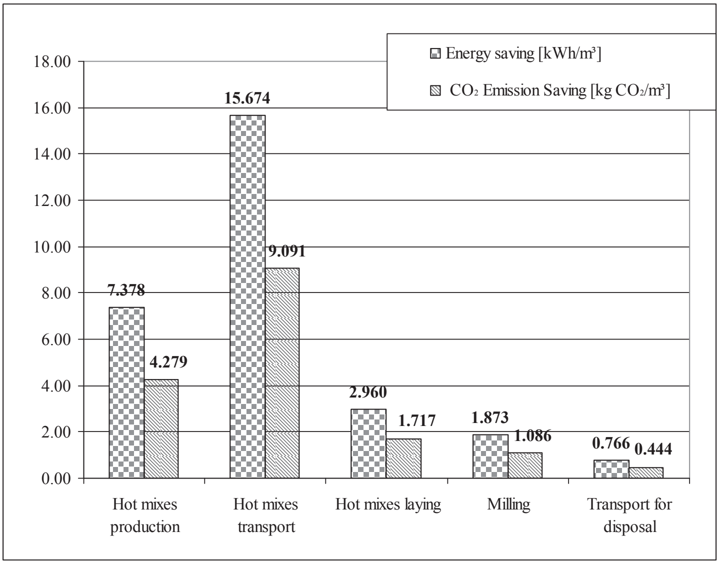


Figure 3. *Energy saving and CO_2 emission saving per unit of AR hot mixes*

Overall, using AR hot mixes instead of traditional design solutions, energy savings amount to 28.651 kWh for each m³ of AR hot mixes finally obtained, and the corresponding reduction in CO_2 emissions is equal to 16.617 kg CO_2/m³.

That savings, compared to overall energy consumption associated to the rehabilitation process, allow energy savings equal to 47%.

4. An Italian case study: the Florence-Pisa-Livorno Highway

The present case study regards the pavement rehabilitation along about 3+3 km of Florence –Pisa–Livorno highway in Tuscany.

Designed and built over many years, this highway doesn't present uniform road section characteristics along its path, but road surface width is sufficiently constant and equal to about 18.60 m.

The measures provided are aimed at improving road safety raising the performance of infrastructure. Interventions originally planned with traditional solutions consisted in:

- Removal of the existing pavement for a thickness equal about to 25-35 cm;
- Lime and/or cement stabilization;

- 30 cm of cold recycling (with foamed bitumen);

- 9 cm of traditional dense hot mix with modified bitumen (SBS medium)

- 5 cm of traditional binder hot mix with modified bitumen (SBS medium)

- 4 cm of traditional open graded hot mix with modified bitumen (SBS hard)

An improving AR solution was defined to increase performance and to implement, at the same time, a significant thickness reduction of pavement rehabilitation, with correspondent money saving.

Several researches [6 - 8] have demonstrated that, at intermediate and high temperatures, rubber stiffens the binder and increases elasticity (proportion of recoverable deformation) with the consequent reduction of temperature susceptibility and improvement in resistance to permanent deformation and fatigue.

The properties conferred to bitumen by the use of rubber as modifying agent clearly reflect on bituminous mixes manufactured with Asphalt Rubber in terms of rutting and cracking resistance compared with conventional bituminous mixtures, as showed by several studies [9, 10].

A structural design method was applied to calculate and verify AR solution, determining the state of tension and deformation (extension) caused by applied traffic loads, verifying the failure criteria for the pavement and taking into account the following issues:

- AR hot mixes contribution by decreasing the extent of vertical compression at the top of pavement foundation;

- AR hot mixes contribution by reducing traction stress at the base layers of bituminous layers.

The structural analysis was performed using BISAR® software by Shell Research. To find the optimal design, several possible solutions have been evaluated, choosing for the surface AR open graded hot mixes, which presents high noise-reduction characteristics, to be applied instead of the open graded hot mix originally planned. The chosen pavement solution also includes an AR SAMI that provides a durable waterproof membrane that has the necessary flexibility to withstand heavy traffic, foundation settling and climate changes. The best advantage is the increased resistance to the propagation of cracking: it allows the sealing of cracks on the existing pavement and prevents the spread to new surface layers, since it reduces the stress transmitted to the upper layers.

This SAMI membrane is formed by the application of the hot asphalt rubber mix (185º), at the rate of 2.5 Kg/m², and also by the application of chips at a rate of 10 to 12 Kg/m². The thickness of this membrane is about 1.2 to 1.5 cm, and can be considered effective for the pavement's design thickness calculation.

Considering experience with Italian road materials and characteristics of aggregates that are used in the present geographic zone and climatic database (max temperature oh 45°C and minimum of – 15°C), it was assumed for respective AR Gap-graded and Open-graded hot mixes an Elastic modulus of 4,000 and 1,700 MPa. SAMI was considered as a 15 mm layer with 3500 MPa of E modulus.

Granulometric specifications for AR materials are shown in Table 3 and Table 4.

Table 3. *Granulometric specifications for AR Gap Graded.*

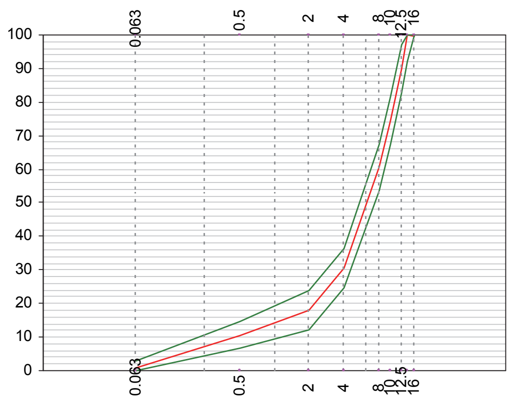
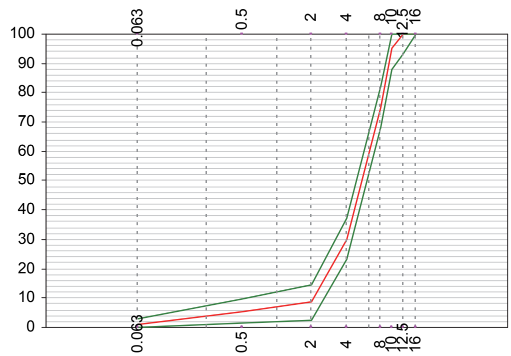
<table>
<tr><th colspan="3">Asphalt Rubber Gap Graded</th><th>MD</th><th>12.5</th><th>B</th><th>AR</th></tr>
<tr><td colspan="7">Mix (UNI EN 13108-1)</td></tr>
<tr><td colspan="3">Granulometric Composition (EN 12697-2)</td><td>Dimension (mm)</td><td>Limit (min)</td><td>Limit (max)</td><td>Regular Value</td></tr>
<tr><td colspan="3" rowspan="9">
</td><td>16</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>14</td><td>92</td><td>100</td><td>100</td></tr>
<tr><td>12.5</td><td>83</td><td>97</td><td>90</td></tr>
<tr><td>10</td><td>68</td><td>82</td><td>75</td></tr>
<tr><td>8</td><td>54</td><td>68</td><td>61</td></tr>
<tr><td>4</td><td>25</td><td>37</td><td>31</td></tr>
<tr><td>2</td><td>12</td><td>24</td><td>18</td></tr>
<tr><td>0.5</td><td>7</td><td>15</td><td>11</td></tr>
<tr><td>0.063</td><td>0</td><td>3</td><td>1</td></tr>
<tr><td>AR BINDER CONTENT</td><td>§ 5.3.1.3</td><td>UNI EN 12697-1/39</td><td>%</td><td>MIN</td><td>7.5-8.5</td><td>8</td></tr>
</table>

Table 4. *Granulometric specifications for AR Open Graded.*

<table>
<tr><th colspan="3">Asphalt Rubber Open Graded</th><th>MD</th><th>12.5</th><th>B</th><th>AR</th></tr>
<tr><td colspan="7">Mix (UNI EN 13108-1)</td></tr>
<tr><td colspan="3">Granulometric Composition (EN 12697-2)</td><td>Dimension (mm)</td><td>Limit (min)</td><td>Limit (max)</td><td>Regular Value</td></tr>
<tr><td colspan="3" rowspan="8">
</td><td>16</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>12.5</td><td>93</td><td>100</td><td>100</td></tr>
<tr><td>10</td><td>88</td><td>100</td><td>95</td></tr>
<tr><td>8</td><td>68</td><td>82</td><td>75</td></tr>
<tr><td>4</td><td>23</td><td>37</td><td>30</td></tr>
<tr><td>2</td><td>3</td><td>15</td><td>9</td></tr>
<tr><td>0.5</td><td>2</td><td>10</td><td>6</td></tr>
<tr><td>0.063</td><td>0</td><td>3</td><td>1</td></tr>
<tr><td>AR BINDER CONTENT</td><td>§ 5.3.1.3</td><td>UNI EN 12697-1/39</td><td>%</td><td>MIN</td><td>8.5-9.5</td><td>9</td></tr>
</table>

Design solution that delivers the best performance in terms of expected life, cost-benefit analysis and noise reduction is detailed below:

- Removal of the existing pavement for a thickness equal about to 30 cm;

- Lime and/or cement stabilization;

- 30 cm foamed bitumen cold recycling;

- AR SAMI;

- 3 cm of calcareous AR Gap Graded hot mix;

- 3 cm of basaltic AR Open Graded hot mix.

Performance comparison of original and alternative solutions considered is shown in Table 5.

Table 5. *Performance comparison of original and alternative pavement solutions performed using BISAR® software by Shell Research.*

Solutions	Total Thickness (m)	Fatigue		Permanent Deformation		Fail	Na (Shell)	Expected life [years]
		et	Nf	ev	Nv			
		*10e-6	Shell	*10e-6	Shell			
Original solution: 4 cm Open mix + 5 cm binder SBS + 9 cm dense SBS + 30 cm Recycling + 150 Mpa	0.430	52.7	1.16E+08	111.3	6.84E+08	Fatigue	1.16E+08	30
Alternative 1: 3 cm AR open + 3 cm AR gap + AR SAMI + 30 cm Recycling + 150 MPa	0.375	43.8	6.07E+09	149.0	2.13E+08	Perm. def.	2.13E+08	35
Alternative 2: 3 cm AR open + 4 cm AR gap + AR SAMI + 25 cm Recycling + 150 MPa	0.335	50.2	3.09E+09	169.8	1.26E+08	Perm. def.	1.26E+08	35
Alternative 3: 3 cm AR open +5 cm AR gap + AR SAMI + 25 cm Recycling + 150 MPa	0.345	55.5	1.87E+09	162.8	1.49E+08	Perm. def.	1.49E+08	35

As one can see from the table, applying the traffic forecasts provided, pavement expected life obtained using the same software, parameters and laws of fatigue to all solutions was:

- 30 years for the original solution;
- 35 years for all the three alternative solutions in AR.

Considering the first AR alternative solution shown above, overall energy balance of the yard was calculated assuming the following conditions.

- Hot mixes production plant has a capacity equivalent to 220,000 kg/hours and it's powered by a two stage burner with a thermal power up to 581 kW.
- Production plant is about 65 km far from rehabilitation yard. Considering the average consumption of each truck and the ratio of CO_2 emissions of fuel (diesel), it is possible to calculate energy savings and correlated CO_2 emissions.

Results are summarized in the Table 6 and Figure 4.

Table 6. *Overall energy balance for rehabilitation yard*

Process	Energy consumption [kWh]	CO_2 Emissions [kg CO_2]
AR hot mixes production	7,894	4,579
Hot mixes transport	16,771	9,727
Milling	28,402	16,473
Stabilization and recycling	31,663	18,365
Transport for disposal of milled material not recycled	2,580	1,496
Hot mixes laying	5,006	2,903
TOTAL	92,316	53,543

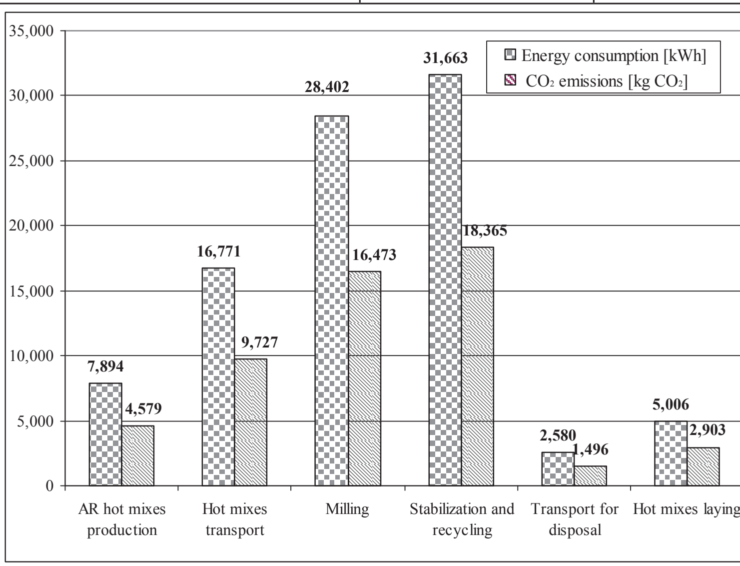


Figure 4. *Overall Energy balance for the yard*

Major savings in terms of materials quantities to be produced, transported, lay and/or disposed, are related to thickness reduction of the pavement, which allows a significant cutback in the conglomerate to be applied and in milling depth.

The respective reductions, compared with traditional solutions, of such materials quantities are:

- reduction of hot mixes produced, equal to 2,435 m³;

- reduction of milled material to be disposed of, equal to 1,793 m³.

Such production and disposal reductions lead to energy savings quantified as reported below. For example, considering the typical fuel consumption function of a cold milling machine, shown in Figure 1, it is possible to quantify energy savings resulting from the 3 cm reduction in milling thickness, as shown in Table 7.

Table 7. *Energy savings achievable by reducing the milling thickness.*

Thickness reduction (cm)	-3.0
Fuel savings per hour (l/hours)	8.0
Milling machine work capacity (m²/hours)	500
Total milling surface (m²)	21,200
Overal work time (hours)	42
Overall fuel savings (l)	340
Energy savings (kWh)	*1,551*
CO_2 reduction (kg CO_2)	*900*

Moreover, the relative reduction in the amount of milled material allows further energy savings on the transport for disposal, estimated in Table 8.

Table 8. *Energy savings achievable by reducing transport for disposal of milled material.*

Milled material reduction (m³)	636
Distance from yard to landfill [km]	10
Fuel consumption [km/l]	2.5
Overall fuel savings [l]	283
Energy savings (kWh)	*1,290*
CO_2 reduction (kg CO_2)	*748*

Overall energy savings, obtained using AR hot mixes as a more efficient alternative to traditional hot mixes, has led to the following results shown in Table 9 and Figure 5.

Table 9. *Overall energy savings achieved using AR hot mixes*

Process	Energy consumption [kWh]	CO_2 Emissions [kg CO_2]
Hot mixes production	10,526	6,105
Hot mixes transport	22,361	12,969
Hot mixes laying	5,006	2,903
Milling	1,551	900
Transport for disposal	1,290	748
TOTAL	40,734	23,626

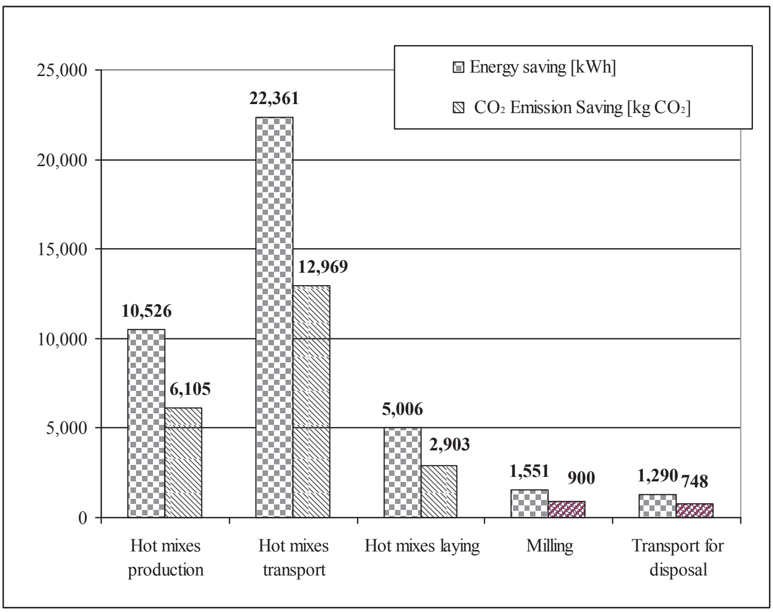


Figure 5. *Overall energy savings achieved using AR hot mixes*

Therefore, AR optimal design solutions allow energy savings equal to about half of total energy consumption resulting from the rehabilitation yard energy balance.

5. Conclusions

AR hot mixes in rehabilitation processes, as well as presenting better structural and functional performance, allow a significant reduction in environmental impact. Results of the analysis described in this article show energy savings that amount to 28.651 kWh for each m ³ of AR hot mixes finally obtained, and a corresponding reduction in CO_2 emissions equal to 16.617 kg CO_2/m^3.

Regarding the case-study presented, that is 3+3 km of an Italian highway, the overall energy balance for the rehabilitation yard shows a total comsumption of 92,316 kWh and a corresponding value of CO_2 emissions equal to 53,543 kg CO_2. The corresponding energy savings are equal to 40,734 kWh and the reduction in CO_2 emissions is equal to 23,626 kg CO_2, which is the equivalent of about 40,000 heavy (10%) and light (90%) vehicles emissions that travels in that highway

Therefore, the savings achievable using AR solutions instead of pavement rehabilitations with traditional hot mixes are equal to about half (44%) the overall energy consumption and carbon dioxide emissions resulting.

Moreover, the AR solutions present better structural and functional performance than the original solution. They may, indeed, support more traffic than expected and have a longer expected life.

The alternative solution also allows a significant noise reduction due to the AR open graded sound absorption coefficient equal to 0.75, in addition to improved adhesion characteristics.

Pavement maintenance expected in the medium to long term of this package will consist in the restoration of the surface layer (3cm) where necessary, and not earlier than 8 years.

All this features lead the pubblic autority that manages the Firenze-Pisa-Livorno highway to consider the AR solution the best technical alternative for the pavement rehabilitation.

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Chapter 6

Evaluation and Design of Chip Seals

Asphalt Rubber Chip Seal Construction Evaluation

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Abstract: The California Department of Transportation (CALTRANS) has used Asphalt Rubber Chip Seals (ARCS) as an effective preventative maintenance strategy for over 20 years, not only on low volume roads, but also on major roadways where high truck traffic, high ambient and pavement temperatures have been prevalent. Due to flushing/bleeding in the wheel paths, on one particular project in Imperial County, California, six test sections were placed to evaluate ARCS specification refinements. Key concepts developed are the minimum CRM content of twenty percent, use of PG 70–10 base stock asphalt, and recognition that two descending viscosity readings are necessary for proper asphalt rubber binder reaction. This paper will evaluate the effects of the ARCS specification refinements through the performance of six test sections, subjected to heavy traffic loads and high temperatures.

Key Words: reacted asphalt rubber binder, pavement preservation, preventive maintenance, asphalt rubber chip seals, descending viscosity readings.

1. Introduction

Six asphalt rubber chip seal (ARCS) test sections were placed on Route 86 from post mile 28.45 to 30.68 in Imperial County on May 12, 2005. The purpose of the test sections was to evaluate the recommendations set forth in the Review and Evaluation of Asphalt Rubber Chip Seals dated October 17, 2005 (ARCS 2005), which investigated several projects that experienced flushing/bleeding. One of the projects that was investigated was on Route 86 From Metric Station 609+70.40 to 708+19.00 (PM 37.3/43.3). It was decided in March of 2005 during the final revisions of the (ARCS 2005) that the recommendations needed to be evaluated. The best way to evaluate the recommendations and determine their soundness was to construct test sections implementing the recommendations and subject all the test sections to the same traffic and climatic loading experienced by one of the projects (Route 86, PM 37.3/43.3) that experienced flushing/bleeding. The test sections were laid out in the following manner, as illustrated in Table 1:

Table 1. *Test section layout*

MARTIN ROAD PM 28.45, SOUTH OF BUCK ROAD PM 30.68 NORTHBOUND HWY. 86 Imperial County, California		
Section	**Application Rate**	**Description Binder Type / Aggregate Type**
1	2.264 l/m^2	Hwy. 86 Binder / Hwy. 86 aggregate
2	2.264 l/m^2	Hwy. 86 Binder / New gradation
3	2.264 l/m^2	* PG Binder / New gradation
4	2.490 l/m^2	* PG Binder / New gradation
5	2.490 l/m^2	* PG Binder (no asphalt modifier) / New gradation
6	2.716 l/m^2	* PG Binder / New gradation
*All PG binders utilized were made from PG 70-10 non-polymer modified Hwy. 86 Binder utilized an AR 4000 paving asphalt		

The asphalt rubber binders utilized for the test sections were the standard California Department of Transportation Type 2 wet process asphalt rubber binder. All components were tested and found to be in conformance with the appropriate specifications. Table 2 below illustrates the percentage of components for each test section.

Table 2. *Asphalt rubber binder composition for test sections*

<table>
<tr><th colspan="2">Test Sections 1 and 2</th><th colspan="2">Test Sections 3, 4 & 6</th></tr>
<tr><td colspan="2">76% AR -4000 Paving grade asphalt</td><td colspan="2">76% PG 70-10 Paving grade asphalt</td></tr>
<tr><td colspan="2">17% Scrap Tire crumb rubber</td><td colspan="2">17% Scrap Tire crumb rubber</td></tr>
<tr><td colspan="2">5% High Natural crumb rubber</td><td colspan="2">5% High Natural crumb rubber</td></tr>
<tr><td colspan="2">2% Raffex 120 ACB- Asphalt modifier</td><td colspan="2">2% Raffex 120 ACB – Asphalt modifier</td></tr>
<tr><td></td><td colspan="2">Test Section 5</td><td></td></tr>
<tr><td></td><td colspan="2">76% PG 70-10 Paving grade asphalt</td><td></td></tr>
<tr><td></td><td colspan="2">17% Scrap Tire crumb rubber</td><td></td></tr>
<tr><td></td><td colspan="2">5% High Natural crumb rubber</td><td></td></tr>
<tr><td></td><td colspan="2">No Asphalt Modifier</td><td></td></tr>
</table>

The aggregates shown in Figure 1 are the actual aggregates utilized for hot pre-coated screenings, the specified gradations are shown in Table 3 and the actual aggregates for each test sections are shown in Table 4.

Grading used on Test Section 1 (current Section 37 Coarse 12.5mm Grading) | **New 12.5mm Medium Maximum Grading**

Figure 1. *Aggregate gradation comparison for 12.5 mm screenings*

Table 3. *Aggregate gradation specification comparison for 12.5 mm screenings*

Section 37 Coarse Grading		New 12.5mm Medium Maximum Grading	
Sieve Sizes	Percentage Passing	Sieve Sizes	Percentage Passing
19-mm	100	19-mm	100
12.5-mm	95-100	12.5-mm	75-90
9.5-mm	50-80	9.5-mm	0-20
4.75-mm	0-15	4.75-mm	0-2
2.36-mm	0-5	2.36-mm	-
75-um	0-2	75-um	0-1

Table 4. *Actual aggregate gradation for test sections*

Sieve Size	Test Section #1	Test Sections 2-6
	% Passing	% Passing (AVG)
19mm	100	100
12.5mm	96	81
9.5mm	54	16
4.75mm	0	1
2.36mm	0	0
75µm	0	0

(AVG) = Average of three test samples.

2. Background

Prior to the publishing of "The Review and Evaluation of Asphalt-Rubber Chip Seals" (ARCS 2005), dated October 17, 2005, it was decided between Caltrans and Industry that due to the binder flushing problems encountered on the three projects addressed in the report (ARCS 2005) and based on the recommendations listed in the report (ARCS 2005), six test sections would be placed south of the original project location in the #2 Lane of Imp-86. These test sections would vary in the asphalt binder type, binder application rate and cover aggregate size as shown in Table 1 in the introduction. The six test sections were placed in the northbound #2 Lane between Metric Stations 467+00 and 503+40 (PM 28.4 and 30.7) – North of Martin Road to North of Hoskins Road on May 12, 2005. On April 27, 2006 and again on April 26, 2007, the test sections were reviewed and visually evaluated. The review and evaluation of the performance of the ARCS test sections, at the approximate two year time period was based on the following criteria:

2.1 Binder Type

The existing specified type 2 asphalt rubber binder type of AR–4000 was used in test sections #1 and #2. Test sections #3, 4 and 6 used PG 70-10 base stock for the type 2 asphalt rubber binder and test section # 5 used the PG 70-10 base stock for the type 2 asphalt rubber binder without asphalt modifier. In the production process of asphalt rubber binder, the paving asphalt is modified with a resinous, aromatic hydrocarbon compound (asphalt modifier/ extender oil) which has a high flash point and which with the additional aromatic contents of the asphalt binder and high natural crumb rubber, reacts with the rubber portion of the crumb rubber modifier (CRM), which is more appropriately defined as a “polymer swell” rather than a chemical reaction. The crumb rubber absorbs the aromatic oils and releases some of the similar oils used in rubber production into the asphalt binder, as illustrated in Figure 2 (HEITZ 1992). The reaction of the asphalt binder and the crumb rubber blend is affected by:

- The type and amount of the mechanical mixing energy in the blending process to react the asphalt binder with the CRM.
- The size and texture of the CRM and the amount of the aromatic hydrocarbons in the asphalt binder and the extender oil.
- The blending time and temperature of the blended Asphalt-Rubber binder.
- The temperature at which the reaction of the blend occurs.
- The length of time the temperature remains elevated.

In order to promote the interaction between the paving grade asphalt binder and the asphalt rubber components, the mixture has to be reacted at high temperatures and with aggressive agitated mixing. Since the asphalt rubber mixture possesses a resistance to flow (high viscosity), heating the mixture to higher temperatures is necessary to keep the asphalt rubber particles suspended in the blend. Although the reaction will be gradual at fairly low temperatures, temperatures from 190.6ºC to 218.3ºC are necessary for the reaction to occur and to produce timely production of the material.

“The specified reaction time should be the minimum time (at a preset temperature) required to stabilize the binder viscosity”. (HEITZ 1992)

From SSP 37-030 (11-16-07) and NSSP 37-030
“The temperature of the blended paving asphalt and asphalt modified mixture shall not be less than 190.6° C nor more than 226.7° C when the CRM is added. The combined material shall be reacted for a minimum of 45 minutes after incorporation of the CRM at a temperature of not less than 190.6° F nor more than 218.3° C. The temperature shall not be higher than 10° below the actual flash point of the Asphalt-Rubber Binder”.

After the incorporation of the CRM into the asphalt binder, viscosity readings shall be taken at least every hour with not less than one reading for each batch of the asphalt rubber binder at a temperature of 218.3℃ and recorded when the reaction of the CRM with the asphalt binder occurs, in three stages (Huff 1980).

As illustrated in Figure 2, in **Stage 1**, where the CRM has been initially mixed with the asphalt binder, the CRM appears as an un-dissolved material. In **Stage 2**, a conversion occurs of the CRM and asphalt binder to a more soluble "gel" state due to the additional blending time and heating temperature. Stage 2 represents the four hour maximum time, after the 45 minute minimum reaction time period and the second descending viscosity reading has been achieved, that the asphalt rubber binder is required to be applied to the pavement surface. If any of the material is not used within the four hour time period after the reaction period, heating shall be discontinued. In Stage 3, where the CRM is somewhat overreacted, the asphalt rubber binder is returned to a material similar to the original paving grade asphalt and does not meet the physical property specifications in SSP 37-030. Once the material temperature cools below 218.3℃, it can be reheated no more than two times. If the material is reheated between 218.3°C and 226.7°C additional scrap tire CRM, not exceeding 10 percent of the total binder weight, can be added and reacted for a minimum of 45 minutes.

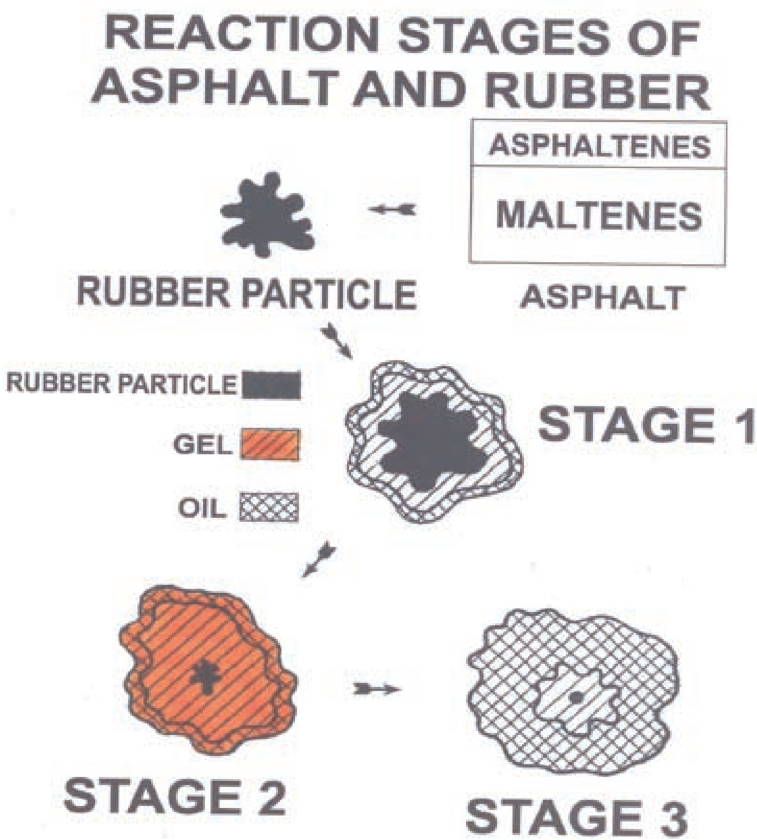

Figure 2. *Reaction stages of asphalt and rubber (HUFF 1980)*

Since the Time-Temperature-Viscosity curve, when plotted, generally depicts a bell shaped curve, it is imperative that when the reaction of the CRM and asphalt binder has reached the top of the curve, that the viscosity of the blend is descending on the curve. To verify this state of the reaction, two descending viscosity readings are necessary to account for any variation in the asphalt rubber binder during production and to verify that the viscosity meets the required specifications. Currently, SSP 37-030 adopted 11-16-07, specifies a viscosity of **1,500 to 3,000 cP** at 218.3°C. The viscosity limits need to be revised to between **1,500 and 2,500 cP** as it relates to ARCS only.

It is important to understand that the design parameters of the binder and testing procedures are necessary to produce a satisfactory formulation. Just mixing together proportions of arbitrarily selected asphalt binder, CRM and extender oil, even within the specified ranges, will not necessarily produce a binder that possesses the physical properties required in the specifications (CTARUG 2003).

2.2 Aggregate Gradation

Test section #1 used the Standard Specification – Section 37-1.02 -12.5 mm coarse gradation with the percent passing the 9.5 mm sieve, revised by a Contract Change Order (CCO), from 70-85 % to 50-80 %. This was the original grading used on the asphalt rubber chip seal portion of project 11-241104 which resulted in bleeding/rutting in the wheel paths and subsequent preparation of the October 17, 2005 report (ARCS 2005). Test sections # 2 through #6 used a revised 12.5 mm coarse gradation as recommended by the ARCS (2005) report. This gradation results in a more one - size aggregate with no intermediate aggregate particles occupying voids in the more uniform mat. The actual aggregates, gradations used are illustrated in Figure 1, Table 3 and Table 3.

3. Asphalt Rubber Chip Seal Test Section Construction

3.1 Binder Application

The current asphalt rubber seal coat specifications (Standard Special Provision 37-030 - last modified on 11-16-2007) require a binder spread rate of 2.490 liters to 2.943 liters per square meter. A lower planned spread rate of 2.264 liters per square meters was placed on test sections # 1 through # 3. A planned spread rate of 2.490 liters per square meter was to be placed on test sections # 4 and # 5 while a planned spread rate of 2.716 liters per square meter was to be placed on test section # 6. The actual spread rates were verified by certified weigh back tickets and found to be either right on target or slightly lower than target by 0.091 l/m^2.

3.2 Aggregate Spread Rate

The current asphalt rubber chip seal specifications (Standard Special Provision 37-030 – last modified on 11-16-2007) require an aggregate spread rate of 14.65 kilograms to 21.70 kilograms per square meter. The old metric designation used on the 11-241101 project was 15 to 22 kilograms per square meter. The actual aggregate spread rate ranged from 20.61 $kg/m^2$to 21.65 kg/m^2.

3.3 Flush Coat Application

The flush coat spread rate was 0.18 l/m^2 of CQS1 emulsion and 3.58 kg/m^2 of sand.

4. Asphalt Rubber Chip Seal Test Section Evaluation

Reference to the control section represents the material components of the ARCS placed on the northbound #2 lane of project 11-241104 (portion) on Imp–86 PM 37.3/43.3 on September 17, 2003 (Figure 3). The ARCS test sections were reviewed on April 27, 2006 and April 26, 2007, which represent the one year and two year construction anniversaries. The test sections were reviewed for aesthetics, crack mitigation and were also tested for coefficient of friction by California test method 342 and skid testing by ASTM E274-06. All test sections with the exception of test section # 6 (0.27) were greater than the minimum coefficient of friction of 0.30 (CTM 342) and all test sections achieved skid numbers above 30 at SN 40 (ASTM E274-06).

Figure 3. *Control section IMP 86 northbound number 2 lane*

Test Section #1

As illustrated in Figures 4 and Figure 5 lowering the application rate of the asphalt rubber binder from 2.58 l/m^2 to 2.264 l/ m^2 improved the aesthetics as compared to the control section.

Figure 4. *Test Section # 1 - 7-13-05*

Figure 5. *Test Section # 1 - 4-26-07 – 11:55 PM - Station 467+67*

Test Section #2

As illustrated in Figures 6 and Figure 7, the wheel path flushing was less than in test section #1 due to the larger 12.5mm coarse aggregate gradation.

Figure 6. *Test Section #2 – 7-13-05*

Figure 7. *Test Section #2 – 4-26-07 – 12:18 PM – Station 473+80*

Test Section #3

Figures 8 and Figure 9 illustrate that test section #3, was rated the best for aesthetics due to the larger sized aggregate (New 12.5 mm coarse), the use of a harder based binder (PG70-10) and a reduced binder spread rate (2.264 l/m^2).

Figure 8. *Test Section #3 – 7-13-05*

Figure 9. *Test Section #3 – 4-26-07 – 12:27 PM – Station 482+80*

Test Section #4

Figure 10 and Figure 11 illustrate that test section #4, had slightly more flushing in the wheel paths as compared to test section # 3. However, test section # 4 was rated the best when considering crack mitigation and aesthetics.

Figure 10. *Test Section #4 – 7-13-05*

Figure 11. *Test Section #4 – 4-26-07 – 12:39 PM – Station 488+00*

Test Section #5

Figure 12 and Figure 13 illustrate that test section#5, had significantly more flushing in the wheel paths as compared to test section # 4 even though all the application components were the same except for elimination of the asphalt modifier. Without the asphalt modifier, the binder formulation had to be heated to 221.1℃, which is above the maximum specified temperature of 218.3℃, in order to achieve a reaction within a reasonable time period for application.

Figure 12. *Test Section #5 – 4-26-07*

Figure 13. *Test Section #5 – 4-26-07 – 12:53PM – Station 494+40*

Test Section #6

Figure 14 and Figure 15 illustrate that test section #6, as compared to test section #5 exhibited a significant improvement in crack mitigation and aesthetics. Although the binder application rate in test section #6 was slightly higher (0.14 l/m^2) than the binder application rate in test section #5, without the asphalt modifier, the flushing was still slightly less.

Figure 14. *Test Section #6 – 7-13-05*

Figure 15. *Test Section #6 – 4-26-07 – 1:03 PM – Station 501+20*

4.1 General

- The six test sections validated the upper limits of the Caltrans maintenance selection criteria (MTAG 2003).

- The changes implemented for all the test sections, excluding test section #5, resulted in greater performance criteria, as compared to the control section based on the two year review of the test sections.

- Coefficient of friction numbers (California TM 342) were increased in each test section as compared to the Route 86 control section.

Satisfactory performance can de defined as the ability of the ARCS to perform under high desert temperatures, to sustain heavy truck traffic loading without flushing/bleeding in the wheel paths and to have a coefficient of friction value of at least 0.30 as determined by California Test Method 342.

Changes that were implemented for all the test sections, except test section #5, resulted in higher quality seal coats (less bleeding) as compared to the "control section". The binder for test section #5 eliminated the asphalt modifier, which resulted in a less than acceptable aesthetic condition. The coefficient of friction value determined by California TM 342 was greater than the 0.30 specified for all the test sections, except test section #6, which was 0.27.

5. Conclusions

5.1 Asphalt-Rubber Binder

- The minimum reaction time currently specified of 45 minutes is not adequate to provide a stable binder viscosity and will not produce the desired binder properties.

- Reduction of the asphalt rubber binder spread rate from the control section spread rate reduces flushing but can affect the mitigation of reflective cracks.

- PG 70-10 asphalt rubber binder does not mitigate reflective cracking as well as AR-4000 but yields better results to mitigate flushing.

- The elimination of the asphalt modifier results in an increase in production time due to the slower reaction time with the rubberized binder.

- With the elimination of the asphalt modifier from the asphalt rubber binder in test section #5, moderate to heavy flushing was observed in the wheel paths.

- Tests validated that asphalt rubber binder can be spread less than 2.490 l/m^2, under suitable field and climatic conditions,

- Test validated that PG 70-10 non-polymer modified binder can be reacted to produce asphalt rubber binder, but requires a longer reaction time.

5.2 Aggregate Screenings

- New 12.5 mm coarse aggregate gradation reduced flushing.

- New 12.5 mm coarse aggregate gradation, with flush coat, did not increase liability for windshield damage.

- Reduction of the aggregate spread rate does not decrease flushing.

Use of the new 12.5mm coarse aggregate gradation and the PG 70-10 base stock resulted in a more aesthetically acceptable seal coat for test sections 2, 3, 4 and 6 as compared to the control section seal coat.

Note:

It should be noted that the untreated pavement areas between each of the test sections have significantly deteriorated in the last two years with more cracking and pavement rutting compared to the treated test sections. This confirms a well-established fact that a timely placed preventative maintenance treatment will greatly extend the service life of a pavement.

6. Recommendations

- Implementation of the Time-Temperature-Viscosity curve with two descending viscosity readings to account for any variation in the asphalt-rubber binder during production.

- Increase the crumb rubber content from the current specified 20±2 % to 21±1 %. This revision would result in a minimum crumb rubber content of 20%. Based on previous projects, the lower specified percentage of 18% has been supplied. This change will extend the performance and assist in reflective crack mitigation.

- Specify that the new 12.5 mm gradation, which is a more uniform, one-sized aggregate, be used for the aggregate screenings for surface treatments.

- Specify a harder higher performance grade paving asphalt (PG 70-10 non-polymer modified for the desert area/truck routes) for use in asphalt rubber binder formulations.

- Implementation of Non-Standard Special Provision 37-030, as presented in ARCS 2008 in Appendix I, will result in increased performance and aesthetics for asphalt rubber chip seals throughout the State of California.

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The Development of a Design Procedure and Usage Criteria for Hot Applied Chip Seal Applications

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ABSTRACT. This paper will discuss the various hot applied and highly modified field blended binder materials that are currently being manufactured in the United States and utilized in chip and cape seal pavement preservation applications. These pavement preservation systems serve as alternatives to expensive construction or re-construction techniques.

Further discussion will focus on a specific chip seal design method which includes binder design and aggregate retention testing to determine materials compatibility prior to a project being constructed. Binder and aggregate application rates, proper aggregate quality and gradation, along with other construction issues and procedures; including traffic volume and climatic region are variables that will also be considered as part of this "design" process.

KEYWORDS: AR/PMAR/RAB, Aggregate Retention, Pavement Preservation, PCI.

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1. Introduction

Asphalt-rubber binder was formulated and developed by Mr. Charles H. McDonald in the early to mid 1960's. Initially the usage criterion for asphalt-rubber binder was limited to chip seal construction also known as Stress Absorbing Membrane (SAM) and Stress Absorbing Membrane Interlayer (SAMI) applications. This asphalt-rubber material has since gone through major evolvement concerning binder design procedures, required testing parameters, laboratory testing equipment, and variations of formulation. Concerning actual chip seal design procedures, what we have learned through the research and development of asphalt-rubber binder, has translated into a focus on the quality of the cover aggregate, further testing and strict quality requirements of said aggregate, along with a proposed new testing protocol to account for many of the asphalt-rubber chip seal design variables, resulting in an overall comfort level of individual project quality and long term performance. Further evolvement of blending and reaction equipment, construction equipment and improved construction procedures has also contributed to the overall predictability and success of SAM and SAMI chip seal projects.

Today there are many variations concerning the use of hot applied binder materials, their formulations and intended use. Some of these binder materials are considered second generation A-R (Asphalt-Rubber) and others are considered RAB (Rubberized Asphalt Binder). The basic difference is the percentage of total modifier(s) that is (are) field blended with the specified PG asphalt cement and the presence of particulate CRM in the binder which relates directly to the viscosity that is tested in the field at the end of recommended interaction time(s). This resulting viscosity that is achieved in the field, regardless of binder type, directly relates to the improved physical properties associated with each individual binder material. These physical properties are determined through the initial binder design profile and proper blending procedures in the field, including vast improvements concerning field QC/QA.

2. Binder Design Effort

When considering binder design, Tables 1 and 2 demonstrate the consistency of viscosity range and this relationship to physical properties of an A-R and PMAR (Polymer Modified Asphalt-Rubber) binder types. A variety of interaction periods are normally conducted to evaluate stability and consistency of the physical properties over a specific timeframe. Table 3 demonstrates the consistency of the physical properties concerning a RAB or PG 76-22TR Binder.

Table 1. *(A-R) Asphalt-Rubber Binder*

Minutes of Reaction

Test Performed	45	90	240	360	1440	Specified Limits
Viscosity, Haake @ 190ºC, Pa's, (10^3), or cP	2,200	2,500	2,500	2,400	2,100	1,500 – 4,000
Resilience @ 25ºC, % Rebound (ASTM D3407)	48	----	48	----	49	18 Minimum
Ring and Ball Softening Point, ºC (ASTM D36)	72	72	72	73	73	52 – 74
Cone Penetration @ 25ºC, 150g, 5 sec., 1/10 mm (ASTM D217)	21	----	22	----	21	20 – 70

Table 2. *(PMAR) - Polymer Modified Asphalt-Rubber*

Minutes of Reaction

Test Performed	45	90	240	360	1,440	Specified Limits
Viscosity, Haake @ 190ºC Pa's, (10^3), or cP	1,900	2,100	2,200	2,000	1,800	1,500 – 4,000
Resilience @ 25ºC, % Rebound (ASTM D3407)	52	----	53	----	55	25 Minimum
Ring and Ball Softening Point, ºC (ASTM D36)	74	75	74	76	75	55 – 80
Cone Penetration @ 25ºC, 150g, 5 sec., 1/10 mm (ASTM D217)	24	----	25	----	25	20 – 65

Table 3. *(RAB) - Rubberized Asphalt Binder or PG 76-22TR (California "modified binder")*

Tests on Original Asphalt	Test Method	Spec. Limits	Results	Results
Apparent Viscosity @ 135ºC, Pa's	AASHTO T316	3.00 max.	2.795	
Dynamic Shear, $G^*/\sin\delta$, kPa	AASHTO T315	1.00 min.	----	
76ºC			2.28	
82ºC			1.25	
Predicted Pass/Fail Temp., ºC			83.9	
Solubility in Trichloroethylene, %	AASHTO T44	93 min.	Report	
Tests on Residue from RTFO	AASHTO T240			
Dynamic Shear, $G^*/\sin\delta$, kPa	AASHTO T315	2.20 min.		

Tests on Original Asphalt	Test Method	Spec. Limits	Results	Results
76ºC			4.76	
82ºC			2.64	
Predicted Pass/Fail Temp., ºC			83.9	
Delta (δ) at 2.20kPa, ºC		80 max.	75.54	
Mass Loss, %	AASHTO T240	1.00 max.	0.343	
Elastic recovery, 25ºC, %		65 min.	72.0	
Tests on Residue from PAV	AASHTO K28		100ºC	110ºC
Dynamic Shear, G*/sinδ, kPa	AASHTO T315	5000 max.		
34ºC (Spec. Temp. -16 Grade)			----	1,112
31ºC (Spec. Temp. -22 Grade)			1,533	1,589
28ºC (Spec. Temp. -28 Grade)			----	2,162
Creep Stiffness, S, @ 60s, MPa	AASHTO T313	300 max.		
-12ºC			130	140
-18ºC			263	273
Slope, m-value	AASHTO T313	0.300 min.		
-12ºC			0.315	0.307
-18ºC			0.272	0.260
Performance Grade	AASHTO M320		PG 76-22TR	PG 76-22TR

The design effort, regardless of the type of hot applied binder that is specified, can be very similar, but at the same time can differ based on the total percentage of modifier that is required and the overall testing regiment that is specified. The design profile concerning A-R and PMAR binder materials has remained fairly consistent over the years (see Tables 1 and 2) and is typically performed for each individual project to evaluate the compatibility and interaction between component materials and to verify stability of the binder over time.

Apparent Viscosity – monitors the fluid consistency of the specified binder to ensure pumpability and to identify binder changes which might affect chip seal or hot mix placement. Measurement is achieved by the use of a rotational viscometer and the results are presented in centipoise (cP) or Pascal Seconds (Pa-s). The required testing can be performed by either using the Brookfield Viscometer or a handheld Haake type Viscometer. If the Brookfield Viscometer is the required method for acceptance the Haake Viscometer should be calibrated and correlated to the Brookfield measurement prior to use.

Penetration (ASTM D5 or AASHTO T49) – modified binder consistency can be measured via the use of a penetrometer at low, moderate and high temperatures. Needle penetration is usually the standard, performed at 39.2º F and 77.0º F. Cone penetration is typically used with A-R and PMAR binder materials and is performed at the same temperatures as needle penetration but is only utilized if the gradation of the crumb rubber modifier (CRM) is 14 mesh or larger. Results are presented in tenths of millimeter units.

Softening Point (ASTM D36 or AASHTO T53) – is an indicator of materials stiffness and will show a tendency of the binder to flow at elevated temperatures. Measurement is achieved by the "ring and ball" method and results are presented in Degrees Fahrenheit or Degrees Centigrade.

Resilience (ASTM D5329) – measures the elastic properties of the modified binder and the results are presented as percentage of rebound for the binder. Resilience is one of the most important properties in the specifications and is considered a reliable and consistent measure of elasticity.

3. Aggregate Requirements

Current specifications recommend the cover aggregate meet specific gradation requirements, usually in accordance with ASTM C136 test method. The crushed coarse aggregate shall have a percentage of wear not more than 7 percent at 100 revolutions and not more that 30 percent at 500 revolutions, as determined by ASTM C131 test method. Ninety percent of the crushed stone or crushed gravel should have a minimum of two fractured faces on three sieves (#4 (4.75 mm), ⅜ inch (9 mm) and ½ inch (12 mm) and should not contain more than 8 percent by weight of flat or elongated particles. Further aggregate testing can be specified such as the sodium sulfate accelerated soundness test which does not allow a total loss greater than 12 percent when the aggregate is subjected to 5 cycles in accordance with ASTM C88 test method. Aggregate gradation should be determined by the application rate of the specified binder which in turn should be based on the existing condition of the pavement receiving the recommended pavement preservation treatment. Table 4 demonstrates common gradation requirements for ⅜ inch (9 mm) and ½ inch (12 mm) aggregate materials utilized in A-R or PMAR SAM and SAMI applications and Table 5 demonstrates common gradation requirements for ¼ inch (4.75 mm) and ⅜ inch (9mm) aggregate materials utilized in hot applied RAB field blend binder materials.

Table 4. *Aggregate Gradation for A-R and PMAR (SAM and SAMI)*

3/8 inch SAM & SAMI Gradation		**1/2 inch SAM & SAMI Gradation**	
Sieve Size	Percent Passing	Sieve Size	Percent Passing
½ inch (12 mm)	100	¾ inch (15 mm)	100
⅜ inch (9mm)	70 – 85	½ inch (12 mm)	95 – 100
¼ inch (4.75 mm)	0 – 15	⅜ inch (9mm)	70 – 85
#8 (2.36 mm)	0 - 5	¼ inch (4.75 mm)	0 – 15
#200 (75 µm)	0 – 1	#8 (2.36 mm)	0 – 5
----	----	#200 (75 µm)	0 - 1

Table 5. *Aggregate Gradation for RAB (PG 76-22TR – "Modified Binder")*

1/4 inch RAB Gradation		3/8 inch RAB Gradation	
Sieve Size	Percent Passing	Sieve Size	Percent Passing
½ inch (12 mm)	100	¾ inch (15 mm)	100
⅜ inch (9 mm)	97 – 100	½ inch (12 mm)	97 – 100
¼ inch (4.75 mm)	65 – 100	⅜ inch (9 mm)	70 – 100
#8 (2.36 mm)	0 – 10	¼ inch (4.75 mm)	0 – 15
#200 (75 µm)	0 – 1	#8 (2.36 mm)	0 – 5
----	----	#200 (75 µm)	0 - 1

Regardless of the aggregate gradation this material is usually specified to be pre-coated or hot pre-coated with 0.5 ± 0.25 percent emulsified binder or PG asphalt cement. When used hot the temperature of the aggregate should be a minimum of 250ºF (121ºC) and should not exceed 325ºF (162ºC).

4. Hot Applied Chip Seal Design

Once the specified binder has been determined and a binder design profile has been completed, along with the aggregate being graded and confirmed to be of desirable quality, the actual chip seal design can begin. The testing has been specified per Caltrans Materials Laboratory "Vialit Test for Aggregate Retention in Chip Seals" (French Chip) as modified from EN 12272-3 and further modified to accommodate hot applied binder materials, A-R, PMAR and RAB (See Tables 1, 2 and 3). Table 6 demonstrates the procedure and test results for this proposed test method.

Table 6. *Modified Vialit Aggregate Retention*

Mineral Aggregate Preparation	
Aggregate Size, mm	6.3 – 9.5
#200 Washed? Yes/No	Yes
Conditioning Prior to Application	Dried @ 122ºF (50ºC) for 24 hours
Vialit Specimen Preparation	
Binder Application Rate, g	79.0
Number of Aggregate Chips Applied	100

Binder Temperature @ Application	325ºF (163ºC)
Aggregate Temperature @ Application	Ambient
Curing Condition of Specimen Plates, Phase 1	Cured @ 325ºF (163ºC) for 15 minutes
Curing Condition of Specimen Plates, Phase 2	Cured @ Ambient, 4 hours minimum
Condition of Specimen Plates @ Testing	Cured @ Test Temperature for 30 minutes
Vialit Test Results **Spec. Limit**	
Aggregate Retention, % 90 min.	
Test Temperature @ 41ºF (5ºC)	
Test Temperature @ 23ºF (-5ºC)	
Test Temperature @ 5ºF (-15ºC)	
Test Temperature @ -8ºF (-22ºC) (Optional)	

Once the specimen plates have cured at the various test temperatures then individual plates are placed into the testing apparatus (see Figure 1) and each plate is tested according to the following criteria. A 500 gram steel ball is dropped from a specific height, three times and the number of stones that are retained on each plate is reported as percent aggregate retention at the various test temperatures (see Figure 2). Ninety percent retention is the lowest value that is acceptable at any one of the four test temperatures.

It is important to note that with each binder type that is tested for aggregate retention value, the weight of the binder (application rate) is currently specified at 79 grams (approximately .43 gallons per square yard). However this rate is associated for only a 3/8 inch (9 mm) maximum size aggregate. There is now consideration to increase the weight of the binder (application rate) to more accurately represent the actual field application rate(s) of the A-R and PMAR binder materials and the potential use of a ½ inch (12 mm) maximum size aggregate. Recommended minimum field application rates for RAB binder type is .40 gallons per square yard (1.32 liters per square meter), for A-R binder type is .60 gallons per square yard (2.86 liters per square meter) and for PMAR binder type is .65 gallons per square yard (3.26 liters per square meter).

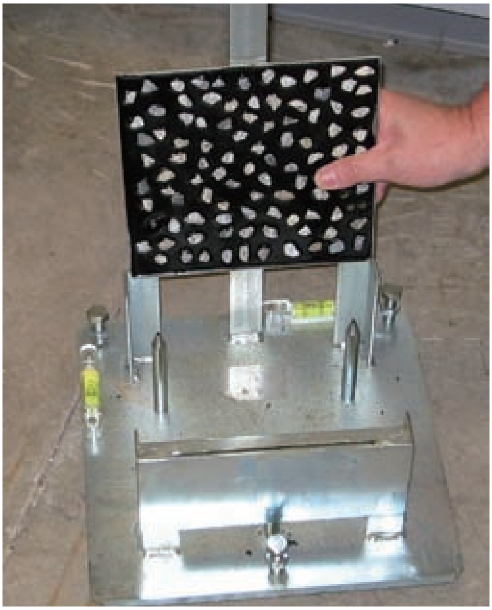

Figure 1. *Modified Vialit Testing Apparatus*

Figure 2. *Modified Vialit Test*

If the aggregate retention has been verified as acceptable the chip seal design data is documented and submitted to the specifying agency prior to the actual project beginning. This should be required as part of the project specifications, which ensures the agency that, the binder and aggregate materials that have been specified are compatible. This type of design work should also be considered a base line concerning the testing of samples taken from the project during construction, which in turn can verify the initial design work that was done.

5. Project Selection Criteria

There is much interest and a definite need concerning criteria by which the proper binder materials and aggregate gradation can be selected as part of a pavement preservation project. As a result of this interest there are many different concepts as to how this might be accomplished, how the resulting data can be confirmed "repeatable" and applied to pavement preservation projects that will potentially experience a wide range of variables, such as differences in climate, topography, traffic volume, traffic type, etc.

Since many specifying agencies have functioning Pavement Management Systems (PMS) this seems to be the most logical place to start the effort. A procedure needed to be identified to accomplish this and establish a method which would potentially match existing pavement condition with best case rehabilitation. This in turn would potentially give the agency a good feel for what to expect in terms of constructability, cost effectiveness (initial and long term), performance (initial and long term) and the potential for reduced maintenance costs over the predicted life of the pavement preservation strategy being considered.

With this in mind it is then logical to look at Pavement Condition Index (PCI) to initially determine if pavement preservation is the correct strategy. As each specifying agency is different and conditions vary widely, these decisions should be made according to what the agencies history is and what the expectations are to meet the end result. It is certain that PCI is not going to give comprehensive data regarding underlying base condition, overall pavement stability or account for other issues that may come into play during evaluation. However, a range of PCI can give the agency an indication as to what type of pre-maintenance should be required and potential selection criteria regarding a pavement preservation strategy that will have the highest potential for success. As this evaluation criterion is adopted the various agencies need to utilize their past history of maintenance activity and performance of their chosen pavement preservation treatments to aid in the decision making process prior to establishing "new" specifications and updated patterns of use for the various "improved" pavement preservation strategies that may be selected.

6. Pavement Condition and Materials/System Choice

When an agency begins to review and categorize the condition of their pavements and establish an overall PCI, the process to decide what type of rehabilitation that will be done on these pavements can be confusing at best. There are a myriad of materials and systems to utilize and most pavement engineers, public works directors and decision makers rely on materials experts and sales people to guide them in the use of the specific materials and systems that are available in the marketplace. Based on experience and performance history the timing of any materials application is critical for that material to perform as expected.

When dealing with conventional emulsified binder, modified emulsified binder, hot applied binder, modified hot applied binder, asphalt-rubber binder and polymer modified asphalt-rubber binder, each material can be part of or included in a specific pavement preservation strategy. The information in Table 7 suggests criterion of usage regarding specific pavement preservation treatments, each having different life expectancies based on the many variables previously discussed.

Table 7. *Suggested Usage Criteria*

PCI Range	Recommended Pavement Preservation Method
75 - 90	Fog Seal, Scrub Seal, Slurry Seal, Conventional Emulsion Chip Seal, Polymer Modified Emulsion (PME) Chip Seal, or Rejuvenating Emulsion Chip Seal
50 - 75	Micro-Surface, PG 76-22PM and PG 76-22TR (field or terminal blend) Chip Seal, Scrub Seal in Conjunction with PG 76-22 PM / TR Chip Seal or PG 76-22 PM or TR Cape Seal
35 - 50	A-R (Asphalt-Rubber) Chip Seal, PMAR (Polymer Modified Asphalt-Rubber) Chip Seal, RASS (Rubberized Asphalt Scrub Seal) in Conjunction with A-R / PMAR Chip Seal, A-R / PMAR Cape Seal, or A-R / PMAR Three Layer Cape Seal System
20 - 35	A-R / PMAR Three Layer Cape Seal System, A-R / PMAR Two Layer Overlay System or A-R / PMAR Three Layer Overlay System

It is very important to recognize that the various binder materials mentioned in Table 7, which may be utilized in a chip seal, cape seal or multi-layered system(s) have limitations concerning application rate. This will in turn effect the recommendation of proper aggregate gradation and application rate of the aggregate, which will in turn affect initial and long term performance of the pavement preservation system being utilized. When specifying the type of slurry seal to use as the wearing course in a cape seal pavement preservation system it is very important to evaluate the street type (residential, collector or major arterial) and choose accordingly. When dealing with residential street type a Type II Slurry Seal is normally utilized. When dealing with collector or major arterial streets with traffic counts higher than 3000 vehicles per day then the Type II Slurry Seal should be upgraded to a Micro-Surface. The following photographs (figures 3 to 7) could be considered representative of certain pavement conditions indicated by PCI.

Figure 3. *PCI of 90 (Indicates Fog Sealing as Proper Pavement Preservation Technique*

Figure 4. *PCI – 75 (Indicates Slurry Seal, Microsurface, Conventional Emulsion/PME Chip Seal or Rejuvenating Emulsion Chip Seal as Proper Pavement Preservation Techniques)*

Figure 5. *PCI – 50 (PG 76-22TR/PM Chip Seal or Cape Seal as Proper Pavement Preservation Techniques)*

Figure 6. *PCI – 35 (Indicates A-R/PMAR Chip Seal or Cape Seal or Three Layer Cape Seal System as Proper Pavement Preservation Technique)*

Figure 7. *PCI – 20 (Indicates A-R/PMAR Three Layer Cape Seal, A-R/PMAR Two Layer Overlay System, A-R/PMAR Three Layer Overlay System or Reconstruction)*

7. Individual Project Variables

Each pavement preservation project has its own specific set of variables to consider. Not only from the standpoint of existing surface condition, pre-maintenance requirements, materials recommendations and appropriate system to specify but other variables regarding construction issues, traffic conditions (current and future), ambient and surface temperatures, climate and topography. Taking into consideration and accounting for as many of these and other variables as possible will result in a pavement preservation project that will not only be cost effective, but will also give the specifying agency the expected long term performance.

Construction variables can be overwhelming as so many different issues need to be considered and accounted for through drafting and enforcement of proper specifications. Construction variables range from proper binder application rate (.08 to .75 gallons per square yard depending on the specified binder and pavement preservation technique), proper aggregate gradation, aggregate quality and application rate (if cover aggregate is required, based on the selection of the pavement preservation system), including the requirement of pre-coated or hot pre-coated aggregate (pre-coating is normally associated with the use of "hot applied binder materials"). Concerning a chip seal application proper and timely rolling with the appropriate number of rollers is critical to the overall embedment of the aggregate into the specified binder and end result aggregate retention value. Other variables such as ambient temperature, surface temperature and project topography will affect the speed of the chip seal "train", so understanding of these variables is also critical as to how construction procedure adjustments need to be made in the field.

8. Conclusion

Utilizing proper pavement preservation techniques is not an exact science. Accounting for as many variables as possible through proper design, specifications, specification enforcement, construction procedures and inspection will consistently result in a cost effective, high quality, long lasting projects. As the specifying agency looks at maintaining pavements with any level of PCI there is a chance for continued surface deviations due simply to the existing surface condition, pre-maintenance that was accomplished (if any), and the pavement preservation system that was chosen. This is certainly the case if the existing pavement condition is below 40 on the PCI. Choosing the proper pavement preservation strategy that is initially cost effective while giving the specifying agency the longest life cycle with minimal or maintenance free performance should be the ultimate goal, this can only be done if focus is placed on the variables discussed in this paper and by understanding the limitations of the various component materials utilized as part of the many pavement preservation strategies available today.

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Chapter 7

Terminal Blend Binders

(Papers on this section contain contributions with terminal blend binders that do not satisfy ASTM ??? specifications. However they have been added to these Proceedings not only to bring to Nanjing the authors so that later they can be the champions of change in their respective countries but also they provide in-site on the relative benefits of terminal blends to other binders.)

Properties of HMA Mixtures Produced with Polymer-Modified and Tire Rubber-Modified Asphalt Binders

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ABSTRACT. This paper summarizes the impact of rubber modified binders on the laboratory performance of hot mix asphalt mixtures (HMA). The purpose of this study is to compare the laboratory performance of the mixtures manufactured with the PG64-28TR and PG76-22TR rubber modified binders to the performance of mixtures manufactured with the Nevada Department of Transportation (NDOT) currently used PG64-28NV and PG76-22NV polymer modified binders, respectively. The PG64-28TR and the PG76-22TR are terminal blend binders modified with 10% ground tire rubber. The mixtures were evaluated in terms of their resistance to rutting, fatigue cracking, thermal cracking, dynamic modulus, and moisture sensitivity. The PG64-28TR mix showed lower rutting resistance than the PG64-28NV mix and lower laboratory fatigue resistance than the PG64-28NV mix at a strain level higher than 500 microns. On the other hand, the PG76-22TR mix showed higher rutting resistance than the PG76-22NV mix and lower laboratory fatigue resistance than the PG76-22NV mix at all strain levels. The PG64-28TR mix and the PG64-28NV mix showed good resistance to thermal cracking, and approximately similar dynamic modulus at 21.1℃. On the other hand, the PG76-22TR mix showed higher dynamic modulus than the PG76-22NV mix at 21.1℃. All mixtures exhibited good resistance to moisture damage.

KEYWORDS: Terminal blend, rutting, fatigue, thermal cracking, dynamic modulus, moisture sensitivity.

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1. Introduction

In the United States and Europe, approximately 300 million used tires are generated each year (Amirkhanian *et al*., 2004). A typical tire contains 60% rubber, 20% steel, and 20% fiber (Putman *et al*., 2004). Therefore, in many countries this has been a serious problem due to the lack of landfill space, and environmental pollution caused by burning the rubber. There are many applications in civil engineering to solve this problem and one them could be the utilization of the tires rubber in pavement by including them in the asphalt mixture. The concept of using tires rubber in asphalt mixtures follows three processes: 1) the dry process where the crumb tire rubber is used as an aggregate in the mix, 2) the wet process where the crumb rubber is blended into the asphalt binder on the job site and 3) the terminal blend, introduced in the 1980s, where the tire rubber is blended into the asphalt binder at the terminal or refinery and shipped to the hot plant.

Over the past few decades, the wet process has been more commonly used than the dry process. The traditional method of producing the wet process is to blend the crumb rubber into the asphalt binder on the job site at elevated temperatures and allow it to react prior to mixing with the aggregate. The rubber modified asphalt binder produced using this technique is referred to as "crumb rubber modified (CRM) binder". The terminal blend technology has been used since the mid 1980s in many states. The tire rubber is blended into the asphalt binder at the terminal or refinery and shipped to the hot plant as a finished product (Sebaaly *et al*., 1998). This technology eliminates the need for the blending unit at the job site. In addition, this technology allows the product to be engineered specifically to meet the climatic conditions defined by the Superpave performance graded system "PG". The rubber modified asphalt binder produced using this technique is referred to in this paper as "tire rubber-modified asphalt binder". Since the CRM binder technology has been around for quiet a long time, several research studies have evaluated the performance of CRM mixtures. The terminal blend technology is relatively new and therefore, has not been extensively evaluated.

The objective of this research effort is to compare the laboratory performance of Nevada' s HMA mixtures made with rubber-modified asphalt binders to the performance of HMA mixtures made with currently used polymer modified binders (Sebaaly *et al*., 2007). Two Nevada HMA mixtures were used in the study: one mix from the south and one mix from the north. Each mix was designed with the two types of binders which resulted in a total of four HMA mixtures that were evaluated in this study. The laboratory performances of the mixtures were evaluated in terms of the following properties:

- Moisture sensitivity
- Dynamic modulus
- Resistance to rutting
- Resistance to fatigue cracking
- Resistance to thermal cracking

2. Materials and mixtures characteristics

Two aggregate sources were identified, one source in northern Nevada (Lockwood) and one source in southern Nevada (Sloan). Currently the Nevada Department of Transportation

(NDOT) specifies a PG64-28NV and a PG76-22NV polymer-modified binders for the northern and southern parts of the state of Nevada, respectively. The NV extension indicates that the binder is graded with the PG-special system which includes the Superpave binder system plus the following properties: toughness, tenacity, and ductility on original and RTFO binder at 4.4℃.

The PG64-28TR and the PG76-22TR binders are modified with 10% ground tire rubber, and were graded following the Superpave PG system plus the elastic recovery test at 25℃ for the original binder. All asphalt binders were supplied by the Paramount Petroleum Company, Nevada, except for the PG76-22NV asphalt binder that was supplied by Ergon Asphalt, Nevada. A total of four HMA mixtures were evaluated in the laboratory.

- A northern Nevada mixture designed with a polymer-modified PG64-28NV binder and aggregate from the Lockwood source.
- A northern Nevada mixture designed with a tire rubber-modified PG64-28TR binder and aggregates from the Lockwood source.
- A southern Nevada mixture designed with a polymer-modified PG76-22NV binder and aggregate from the Sloan source.
- A southern Nevada mixture designed with a tire rubber-modified PG76-22TR binder and aggregates from the Sloan source.

Figure 1 shows the gradation of the Lockwood and Sloan aggregates, respectively. Both gradations meet the specifications for NDOT type 2C gradation. All four mixtures were treated with 1.5% of hydrated lime by weight of aggregates according to the NDOT specifications.

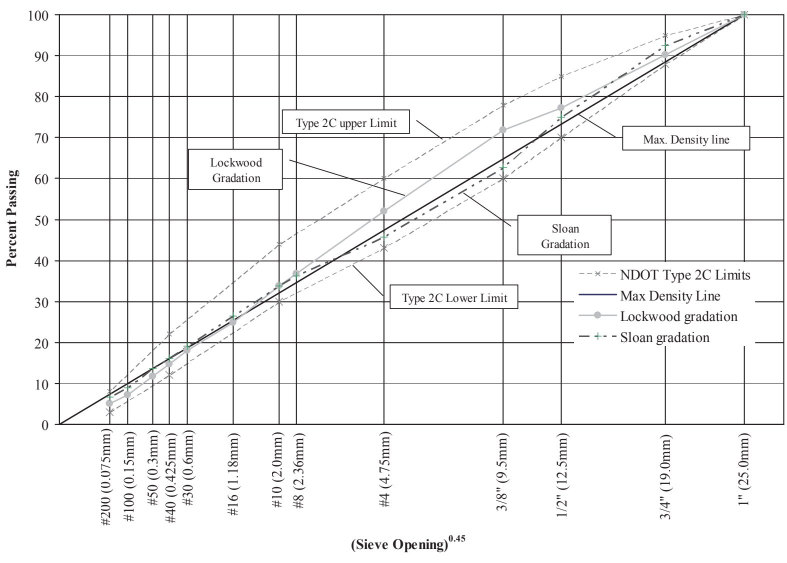


Figure 1. *Gradation of the Lockwood and Sloan aggregates blend*

2.1. Mix designs

The NDOT Hveem Mix Design Method as outlined in the NDOT Testing Manual was used as a guide to design the mixtures. The heated aggregate samples were mixed with various amounts of asphalt binder so that at least two were above and at least two were below the expected optimum asphalt content. The optimum binder content was determined by identifying the asphalt contents which provide the specified minimum Hveem stability, then the highest binder content that has air voids between 4 and 7%, and VMA between 12 and 22%. The mixture and volumetric properties along with the corresponding NDOT specifications at the design binder contents are summarized in Table 1.

Table 1. *Mix Design Summary and Moisture Sensitivity Properties*

Property		PG64-28NV	PG64-28TR	PG76-22NV	PG76-22TR	SSRBC1
Rotational viscosity at 135℃, Pa.s		0.61	0.60	1.98	1.50	3.00 Max.
Optimum Binder Content (%DWA[2])		4.8	5.0	3.8	3.9	-
Air Voids in Total Mix (%)		5.9	6.0	4.7	5.7	4 - 7
Void in Mineral Aggregate (%VMA)		14.1	14.8	13.2	14.3	12 - 22
Hveem Stability		40	39	42	39	37 Min.
AASHTO T-283 Moisture Sensitivity Test						
Air Voids, %	Average	6.8	6.6	7.2	7.1	-
Tensile Strength at 25 ℃, kPa (Dry)	Average	807	786	910	1124	448 kPa4 690 kPa5
	CV[3] (%)	2	10	10	10	
Tensile Strength at 25 ℃, kPa (Wet)	Average	765	752	772	958	-
	CV[3] (%)	6	4	8	10	
Tensile Strength Ratio, %		95	95	85	85	70%

1. Denotes “Standard Specification for Road and Bridge Construction”.
2. Denotes “dry weight of aggregate”.
3. Denotes “coefficient of variation” defined as the ratio of the standard deviation over the average times 100.
4. Minimum tensile strength for the PG64-28NV mix and the PG64-28TR mix.
5. Minimum tensile strength for the PG76-22NV mix and the PG76-22TR mix.

3. Laboratory evaluation

The laboratory produced mixtures were evaluated in terms of the following properties: moisture sensitivity, dynamic modulus, resistance to rutting, resistance to fatigue cracking, and resistance to thermal cracking. The laboratory performances of mixtures manufactured with tire rubber modified binders were compared to the corresponding laboratory performance of the mixtures manufactured with polymer modified binders.

3.1. Moisture sensitivity

According to the NDOT Hveem mix design method, the dry tensile strength (TS) should meet a minimum of 448 kPa for the PG64-28NV mix and the PG64-28TR mix, and a minimum of 690 kPa for the PG76-22NV mix and the PG76-22TR mix. The tensile strength ratio (TSR) should be at least 70%. The TSR is measured as the ratio of the moisture conditioned tensile strength over the unconditioned TS of the HMA. The moisture conditioning process followed the procedure described in AASHTO T-283 test with the exception of using five samples at both the conditioned and unconditioned stages.

Table 1 summarizes the results of the mixtures TS and TSR at 25℃. The coefficients of variations represent the variability of the data among the replicates for each test. The data in Table 1 shows that all mixtures met the NDOT Hveem mix design criteria for moisture damage. In the case of the northern mixtures, the PG64-28NV mix and the PG64-28TR mix exhibited similar TS and TSR ratio. In the case of the southern mixtures, the PG76-22NV mix exhibited lower TS values than the PG76-22TR mix but similar TSR value.

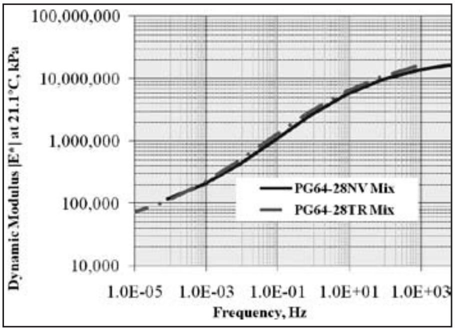
3.2. Dynamic modulus

Stiffness is an important property to consider for the design of any structure. In the case of HMA mixtures the stiffness changes with temperature and frequency. Thus, when designing a pavement structure, the moduli values must be accurately represented at all desired temperature-frequency combinations. The dynamic modulus represents the stiffness property of HMA mixtures. The dynamic modulus test as standardized in AASHTO TP62 was used to develop the dynamic modulus master curve of the HMA mixtures. For linear viscoelastic materials such as HMA mixtures, the stress-strain relationship under a continuous sinusoidal loading is defined by its complex dynamic modulus (E*). Mathematically, the "dynamic modulus" is defined as the absolute value of the complex modulus.

The recommended test series for the development of the E* master curve for use in pavement response and performance analysis consists of testing at temperatures of -10, 4.4, 21.1, 37.8 and 54℃ and loading frequencies of 0.1, 0.5, 1.0, 5, 10, and 25 Hz at each temperature. The development of a master curve allows the modulus to be determined at any reasonable temperature-frequency combination.

The dynamic modulus test was performed on all four mixtures. Master curves were constructed using the principle of time-temperature superposition. First, a standard reference temperature was selected (21.1℃), and then data at various temperatures were shifted with respect to loading time until the curves merge into a single smooth function. The 150 mm diameter cylindrical samples were compacted in the Superpave Gyratory Compactor (SGC) to an air voids content of 7±1%. A total of three replicates were tested for each mixture.

The loading frequency simulates the speed of traffic loads on the pavement. A higher loading frequency represents a fast moving load and a lower loading frequency represents a slow moving load. Figure 2 shows the dynamic modulus master curves of the various mixtures at 21.1℃. Figure 2 shows that the PG64-28TR mix and the PG64-28NV mix have similar dynamic modulus property while the PG76-22TR mix has a higher dynamic modulus property than the PG76-22NV mix at all frequencies. This indicates that the PG76-22TR mix is stiffer than the PG76-22NV mix.



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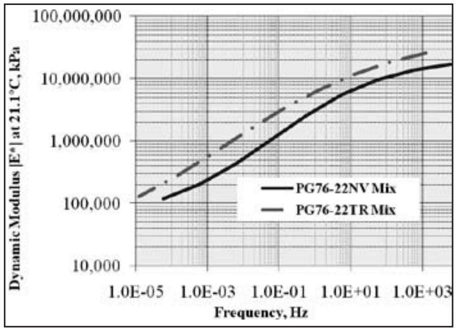


b

Figure 2. *(a) Dynamic modulus of the PG64-28NV mix and the PG64- 28TR mix at 21.1℃*
(b) Dynamic modulus of the PG76-22NV mix and the PG76-22TR mix at 21.1℃

3.3. Resistance to rutting

The rutting resistance of the various mixtures was evaluated in the laboratory using the repeated load triaxial (RLT) test and the asphalt pavement analyzer (APA) test.

3.3.1 Rutting behavior of mixtures in the repeated load triaxial test

The repeated load triaxial (RLT) test measures the axial permanent deformation in the HMA mixture as it is subjected to triaxial stress conditions. The test specimen was cored from the center of a 150 mm by 180 mm gyratory compacted sample. The tested samples had air voids of 7±1%. The triaxial condition was achieved by applying a static radial confining pressure of 207 kPa using compressed air and a repeated deviatoric vertical stress. The repeated deviatoric stress was applied for 0.1 seconds followed by a 0.6 seconds rest period. The test was conducted at three different temperatures and on three replicates from each mixture at each temperature for a total of 12,000 cycles with continuous measurements of the vertical deformation along the middle 100 mm of the sample. This enabled the evaluation of the permanent vertical strain in the sample as a function of load cycles.

The RLT test results were used to characterize the rutting behavior of the various mixtures in the form of the performance model provided by the new mechanistic-empirical pavement design guide (MEPDG). Equation 1 shows the performance model suggested in the MEPDG to assess rutting in the HMA layer relates the ratio of axial permanent strain (ε_p) and the resilient axial strain (ε_r) to the number of loading cycles (N) and temperature (T).

$$\frac{\varepsilon_p}{\varepsilon_r} = aN^b T^c \qquad [1]$$

In this equation, a, b, and c are experimentally determined coefficients for the permanent deformation behavior of HMA mixtures in the secondary stage shown in Figure 3. The primary and tertiary stages are not taken into account for developing the rutting model. The secondary stage corresponds to small rate of rutting exhibiting a constant rate of change of rutting that is also associated with volumetric changes; however, shear deformations also increase.

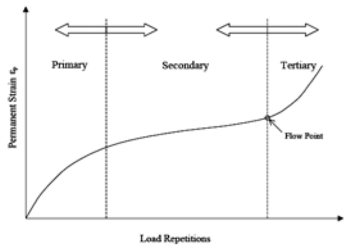


Figure 3. *Typical repeated load permanent deformation behavior of pavement materials*

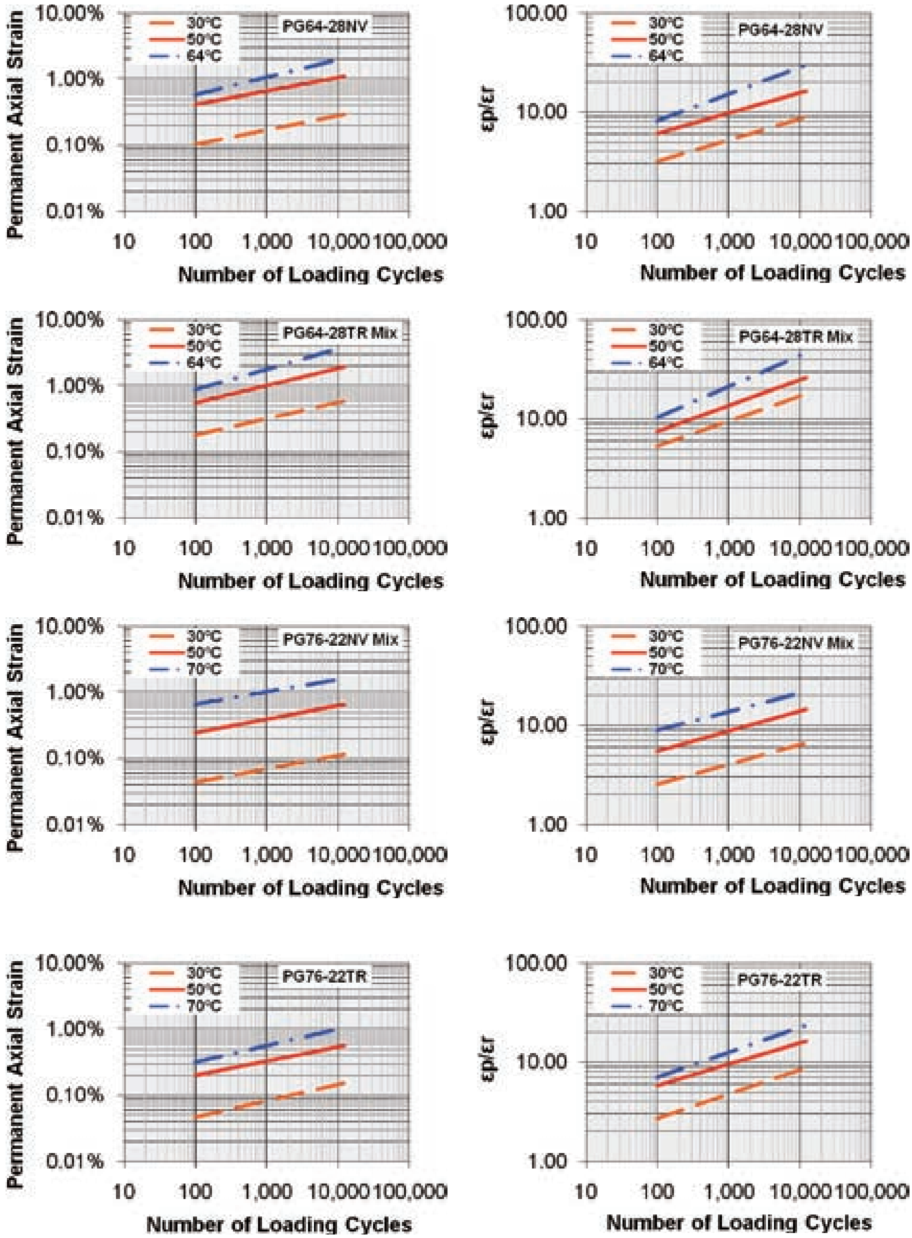
The data from the RLT were plotted in the form of permanent strains versus loading cycles and the ratio of permanent strain over the resilient strain versus loading cycles as shown in Figure 4. Each curve represents the average response of three replicate samples tested at each of the specified testing temperatures listed on the individual plots in Figure 4. Equations 2 to 5 describe the rutting models developed by fitting a multiple linear regression for the test data at the various temperatures for each mixture. The SAS macro called REGDIAG was used to fit the multiple linear regressions (MLR).

$$\text{PG64-28NV mix: } \frac{\varepsilon_p}{\varepsilon_r} = -2.10154 N^{0.222908} T^{1.432324} \quad (R^2 = 98.3\%) \qquad [2]$$

$$\text{PG64-28TR mix: } \frac{\varepsilon_p}{\varepsilon_r} = -1.46011 N^{0.257669} T^{1.071886} \quad (R^2 = 96.0\%) \qquad [3]$$

$$\text{PG76-22NV mix: } \frac{\varepsilon_p}{\varepsilon_r} = -2.10406 N^{0.197635} T^{1.437127} \quad (R^2 = 99.8\%) \qquad [4]$$

$$\text{PG76-22TR mix: } \frac{\varepsilon_p}{\varepsilon_r} = -1.74528 N^{0.238186} T^{1.165057} \quad (R^2 = 99.3\%) \qquad [5]$$



(a) (b)

Figure 4. *(a) Permanent axial strain versus number of loading cycles*
(b) Ratio of permanent axial strain to resilient axial strain versus number of loading cycles

The rutting behavior of the tire rubber-modified asphalt mixtures were statistically compared to the polymer modified asphalt mixtures using the least square mean statistical technique. The ratio of the permanent axial strain to the resilient axial strain ($\varepsilon_p/\varepsilon_r$) after 12,000 loading cycles were determined for each mixture using Equations 2 – 5 and compared for any statistically significant difference. Table 2 summarizes the $\varepsilon_p/\varepsilon_r$ values along with the significance difference after 12,000 loading cycles. The following nomenclature is used in Table 2 and throughout the paper:

- H: The $\varepsilon_p/\varepsilon_r$ of the tire rubber-modified mixture is statistically significantly higher than the corresponding polymer-modified mixture.

- L: The $\varepsilon_p/\varepsilon_r$ of the tire rubber-modified mixture is statistically significantly lower than the corresponding polymer-modified mixture.

- NS: No statistical significant different exist between the tire rubber-modified mixture and the corresponding polymer-modified mixture.

Table 2. *RLT Test Results at 12,000 cycles*

Temperature, ℃	Mix	$\varepsilon_p/\varepsilon_r$ after 12,000 cycles	Significance Level*
30	PG64-28NV	9.7	H+
	PG64-28TR	16.1	
50	PG64-28NV	17.5	H+
	PG64-28TR	29.2	
64	PG64-28NV	23.4	H+
	PG64-28TR	38.8	
30	PG76-22NV	6.9	L#
	PG76-22TR	6.2	
50	PG76-22NV	17.6	L#
	PG76-22TR	15.5	
70	PG76-22NV	27.5	L#
	PG76-22TR	24.3	

* Comparing the NV mixtures to the TR mixtures.

+ $\varepsilon_p/\varepsilon_r$ of the TR mixture is statistically significantly higher than the εp/εr of the NV mixture.

$\varepsilon_p/\varepsilon_r$ of the TR mixture is statistically significantly lower than the εp/εr of the NV mixture.

Table 2 shows that the PG64-28TR mix has significantly higher εp/εr than the PG64-28NV mix at all three test temperatures of 30℃, 50℃, and 64℃. On the other hand, the PG76-22TR mix has significantly lower εp/εr than the PG76-22NV mix at 30℃, 64℃, and 70℃. For the same resilient strain the RLT test results shows that the PG64-28TR mix will exhibit a higher permanent strain than the PG64-28NV mix resulting in a lower resistance to rutting for the PG64-28TR mix compared to the PG64-28NV mix. On the other hand, for the same resilient strain the PG76-22TR mix will exhibit a lower permanent strain than the PG76-22NV mix resulting in a higher resistance to rutting for the PG76-22TR mix compared to the PG76-22NV mix. However, a better laboratory rutting resistance in the RLT will not necessarily translate to a better rutting performance in the field as the rutting performance of an asphalt pavement is highly dependent on pavement structure, the HMA stiffness, and the rutting characteristics of the HMA mixture. In a mechanistic pavement analysis, an HMA layer with a higher stiffness will produce a lower vertical resilient strain in the HMA layer and hence a better rutting life. Therefore, a mechanistic analysis needs to be conducted to correctly evaluate rutting performance of the various mixtures.

3.3.2. Rutting behavior of mixtures in the asphalt pavement analyzer

The asphalt pavement analyzer (APA) test was used to empirically evaluate the resistance of the mixtures to rutting. The APA test is standardized under AASHTO TP63-03, where a loaded concave wheel travels along a pressurized rubber hose that rests upon the HMA sample. Four 150 mm diameter cylindrical samples were compacted from each mix using the SGC to a height of 76 mm and air voids of 7±1%. Samples are secured within form-fitting acrylic blocks during testing. The APA wheel load is 45 kg and the hose pressure is 690 kPa. The samples were conditioned for six hours before being tested in the dry condition at 60℃ to 8,000 cycles. A data acquisition program records rut depths at 2 points within each sample and their average is reported.

Table 3 summarizes the rut depth data from the APA test at 60℃. The coefficients of variations represent the variability of the data among the four replicates for each mix. A maximum of 8 mm rut depth after 8,000 cycles at 60℃ has been used by NDOT as a general failure criterion. The APA data in Table 3 indicate that all mixtures meet and exceed the NDOT APA criterion. No significant difference was observed between the TR and the NV mixtures at 60℃ indicating similar resistance to rutting. These results could be an indication that the APA test is not sensitive enough to test for any significant difference in the rutting resistance of the mixtures when tested at a temperature lower than the high performance temperature of the PG binder grades specifically as is the case for the PG76-22NV and PG76-22TR mixtures.

Following the above observations, the PG64-28NV mix and PG64-28TR mix were tested under the APA at 64℃ and the PG76-22NV mix and PG76-22TR mix were tested under the APA at 76℃. Table 3 shows that there is no significant difference in the rut depths between the PG64-28NV mix and the PG64-28TR mix at 64℃, whereas the PG76-22TR mix exhibited a slightly higher rut depth than the PG76-22NV mix at 76℃.

It should be noted that the contradiction between the APA and RLT results is due to the fact that the APA is an empirical tests while the RLT is a mechanistic test. Therefore, the final impact of the RLT results on the rutting performance of the mixtures must be coupled with the dynamic modulus properties through an extensive mechanistic analysis. While the results of the APA test are directly and empirically correlated to the rutting performance of the mixtures. Due to length limitations, this information is not included in this paper and has been published in other reports (Sebaaly, 2006).

Table 3. *APA Rut Depths for All Mixtures*

Temperature, ℃	Mix	Mean Rut Depth (mm)	CVfour samples rut depth, %
60	PG64-28NV	1.7	4.6
	PG64-28TR	1.2	9.5
	PG76-22NV	1.5	8.6
	PG76-22TR	1.2	5.9
64	PG64-28NV	2.1	4.0
	PG64-28TR	2.0	6.7
76	PG76-22NV	2.7	7.2
	PG76-22TR	3.4	3.5

4. Fatigue characteristics of mixtures

The resistance of the HMA mixtures to fatigue cracking was evaluated using the flexural beam fatigue test in accordance with the AASHTO T321-03. Fatigue life or failure was defined as the number of cycles corresponding to a 50% reduction in the initial stiffness. The fatigue behaviors of the four HMA mixtures were characterized by the following relationship.

$$N_f = k_1 \left(\frac{1}{\varepsilon} \right)^{k_2} \qquad [6]$$

where N_f is the fatigue life (number of load repetitions to fatigue damage), ε_t is the applied tensile strain, and k_1 and k_2 are experimentally determined coefficients. The beam fatigue samples were compacted using the kneading compactor. The compacted beams were 405 mm long by 76 mm thick and 76 mm. Since fatigue cracking is more critical during the in-service life of the pavement, the compacted beams were subjected to long-term oven aging following the Superpave procedure of 120±3 hours at a temperature of 85℃, then cut using a diamond blade saw to 381 mm long by 51 mm thick by 64 mm wide. The tested specimen air voids were 7±1%. It is clearly shown from the beam fatigue relationship that the higher the k_1 and k_2 values, the more fatigue resistant the mix will be. Table 4 shows the beam fatigue results of each mix at 22.2 ℃, and the fatigue regression coefficients for all mixtures. Figure 5 shows the fatigue characteristics of the various mixtures.

Table 4. *Summary of the Fatigue Test Results and Regression Coefficients for All Mixtures*

Mixture	Binder Content, % DWA*	Average Air Voids, %	Initial Flexural Stiffness, MPa	k_1+	k_2+	R^2, %
PG64-28NV	4.8	6.5	1,848	1.047E-04	2.769	98.6
PG64-28TR	5.0	7.0	1,862	3.171E-17	6.541	98.7
PG76-22NV	3.8	6.8	1,924	5.656E-08	3.722	98.8
PG76-22TR	3.9	7.0	3,254	6.455E-05	3.262	97.6

* Denotes "Dry Weight of Aggregates."

+ k_1 and k_2 are experimentally determined coefficients using the following relationship:

$$N_f = k_1 (1/\varepsilon)^{k_2}$$

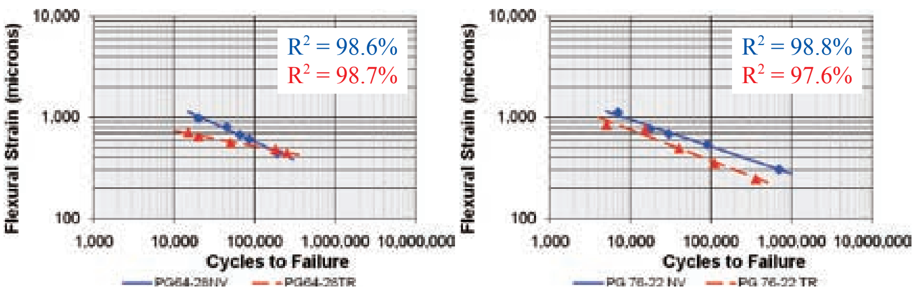


Figure 5. *Relationships between strain levels and number of cycles to failure of the various mixtures at 22.2 ℃*

Table 5 summarizes the fatigue data of the various mixtures in terms of the number of cycles to failures under tensile strains of 300, 500, and 800 microns, representing low, medium, and high levels of tensile strains at the bottom of the HMA layer of a flexible pavement, respectively. The effect of tire rubber-modified binder on the fatigue behavior was evaluated in terms of the ratio of fatigue life of the tire rubber-modified mixtures over the corresponding polymer-modified mixture. A fatigue life ratio greater than one indicates a better life for the tire rubber-modified mixtures at the same strain level. Table 5 shows a better laboratory fatigue behavior for the polymer-modified mixtures compared to the tire rubber-modified mixtures at all three strain levels.

Table 5. *Laboratory Fatigue Life for All Mixtures at 22.2℃*

Mix	Air Voids,%		Number of cycles to failure, Nf		
	average (%)	CV[-] (%)	Strain Level		
			300 microns	500 microns	800 microns
PG64-28NV	6.5	5.9	NA	144,925	39,435
PG64-28TR	7.0	1.6	NA	124,220	5,740
PG76-22NV	6.8	10	733,435	109,550	19,050
PG76-22TR	7.0	5.6	200,040	37,800	8,160
Fatigue life ratio of PG64-28TR mix to PG64-28NV mix			NA	0.86	0.15
Fatigue life ratio of PG76-22TR mix to PG76-22NV mix			0.28	0.35	0.43
PG64-28TR mix vs PG64-28NV mix			NA[+]	NS[#]	L[*]
PG76-22TR mix vs PG76-22NV mix			L[*]	L[*]	L[*]

[-] Total of 6 specimens

[+] Results not reported because the test was not conducted at this strain level.

[#] No significantly difference between the NV and TR mixtures.

[*] N_f of the TR mixtures is significantly lower than the N_f of the NV mixtures.

Additionally, Table 5 summarizes the results of the statistical analyses for the mixtures comparison at all three strain levels. The PG64-28TR mix has significantly less fatigue resistance than the PG64-28NV mix at 800 microns but there is no significantly difference between these two mixtures at 500 microns. The two mixtures are not compared at the 300 microns since they were not actually tested at this strain level. Additionally, the PG76-22TR mix has significantly less fatigue resistance than the PG76-22NV mix at 300, 500, and 800 microns.

However, a better laboratory fatigue resistance will not necessarily translate to a better fatigue performance in the field as the fatigue life of an asphalt pavement is highly dependent on both the modulus and the fatigue characteristics of the HMA mixture and their interaction. In a mechanistic pavement analysis, an HMA layer with a higher stiffness will show a lower laboratory fatigue life but on the other hand it will produce a lower tensile strain under field loading. Therefore, depending on the magnitude of strain reduction, the HMA layer with the higher stiffness may result in a longer fatigue life in the field or vice versa. Therefore, a full mechanistic analysis is needed to effectively evaluate the impact of tire rubber on the fatigue performance of an HMA pavement.

5. Thermal cracking resistance in the thermal stress restrained specimen

The thermal stress restrained specimen test (TSRST) was used to determine the low temperature cracking resistance of the various mixtures. The test cools down a 51 by 51 by 254 mm beam specimen at a rate of 10 ℃/hour while restraining it from contracting. While the beam is being cooled down, tensile stresses are generated due to the ends being restrained. The HMA mixture fractures as the internally generated stresses exceed its tensile strength. The temperature at which fracture occurs is referred to as "fracture temperature" and represents the field temperature under which the pavement will experience thermal cracking. The TSRST samples were compacted using the kneading compactor to air voids of 7±1%. Since thermal cracking is more critical during the in-service life of the pavement, the compacted beams were subjected to long-term oven aging for 120 ± 3 hours at a temperature of 85 ℃.

It should be note that thermal cracking distress is only a concern in the northern part of the state of Nevada. Hence, only the northern mixtures (the PG64-28NV mix and the PG64-28TR mix) were tested in the laboratory for thermal cracking resistance.

Table 6 summarizes the fracture temperatures for the PG64-28NV mix and the PG64-28TR mix. The coefficients of variations represent the variability of the data among the three replicates for each test. The fracture temperatures of both mixtures were colder than the low performance temperature of the binders of -28 ℃. Both mixtures had similar fracture temperatures within 2 ℃ which resulted in a non significant difference at the 5% significance level.

Table 6. *Thermal Cracking Fracture Temperatures of the PG64-28NV and the PG64-28TR mixtures*

Mixture	Air Voids, %	CV, %	Fracture Temperature ℃
PG64-28NV	7.0	3.1	-33
PG64-28TR	7.0	3.9	-31

6. Conclusion and recommendations

The objective of this research effort was to compare the performance of Nevada's HMA mixtures made with terminal blend (TR) rubber modified asphalt binders with Nevada's HMA mixtures made with polymer modified binders (Sebaaly *et al*., 2007). This objective was achieved through a laboratory-based experiment that evaluated the resistance of the two types of mixtures to rutting, fatigue, thermal cracking, and moisture damage. In addition, the dynamic moduli of the mixtures were evaluated. Based on the data generated from the laboratory experiment, the following conclusions can be made.

- The moisture sensitivity of the terminal blend (TR) and the polymer modified (NV) mixtures are well above the minimum required tensile strength ratio of 70%. This indicates that both mixtures exhibit good resistance to moisture damage.

- When the laboratory rutting resistances of the mixtures were compared in the RLT test, the PG64-28TR mix showed lower rutting resistance than the PG64-28NV mix. On the other hand, the PG76-22TR mix showed higher rutting resistance than the PG76-22NV mix. Therefore, it may be advantageous to use the PG76-22TR mix in southern Nevada where rutting is a more prevalent failure mode.

- All mixtures met the NDOT APA criterion of 8 mm at 60℃ after 8,000 loading cycles.

- When the laboratory fatigue resistances of the mixtures were compared, the PG64-28TR mix showed lower fatigue resistance than the PG64-28NV mix at strain levels higher than 500 microns, but the same at 500 microns, and the PG64-28TR mix showed higher fatigue resistance than the PG64-28NV mix at strain levels lower than 500 microns. On the other hand, the laboratory fatigue resistance of the PG76-22TR mix is lower than the laboratory fatigue resistance of the PG76-22NV mix at all strain levels.

- The dynamic modulus of the PG64-28TR mix is approximately equal to the dynamic modulus of the PG64-28NV mix at 21.1℃, and the dynamic modulus of the PG76-22TR mix is higher than the dynamic modulus of the PG76-22NV mix at 21.1℃.

- The PG64-28TR mix and the PG64-28NV mix showed good resistance to thermal cracking since the fracture temperatures of both mixtures were colder than the low performance temperature of the binders (-28℃). Both mixtures had similar fracture temperatures (within 2℃) which resulted in a non-significant difference.

- It is recommended that a full mechanistic analysis be performed to evaluate the combined impacts of mixtures properties and their resistance to the various failure modes along with pavement structure on the responses of HMA pavements to traffic loads.

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Pavement Monitoring Results After Seven Years of Using Crumb Rubber Modified Asphalt in Brazil

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ABSTRACT: After the year 2000, when the first AR conference in Vilamoura Portugal took place, the first trial sections began to be built using crumb rubber modified binders in major highways in Brazil. Since then, the use of the technology has increased very much and there are already over 2.000 km of roads using some kind of crumb rubber modified asphalt in Brazil. Almost all of the Brazilian applications use Terminal Blend binders.

This paper is the result of the continuous monitoring of these pavements since then. Some of these applications were done using around 15% of crumb rubber in dense graded mixes and some of them using at least 20% in gap graded mixes. Our focus in this paper is on these gap graded applications which have been used on some important Brazilian highways now.

The paper is pointing out the problems and difficulties presented during the asphalt mix production and the results achieved, in terms of pavement performance. The paper shows the properties of the binders used, such as viscosity, elastic recovery, penetration, flow etc. It also presents the mix characteristics and all of the pavement performance related data ever since its application, such as Traffic, FWD deflections, skid resistance and ride quality.

The majority of the applications were done within the private sector in toll concession companies. As they are obliged by contract to survey the pavement every year, the paper shows these monitoring surveys qualifying the pavement performance since its application. The paper also presents some comparisons between the performance of the sections rehabilitated with crumb rubber modified binder using thickness reduction factors to the performance assessed in sections rehabilitated with conventional binders.

With the arrival of the asphalt rubber "in situ" mixer to Brazil, it will be possible to produce higher viscosity binders and have even better pavement performances.

KEYWORDS: Pavement Performance, Crumb Rubber Modified Applications, Brazil

1. Introduction

In 2001 the first experiences with asphalt modification with crumb rubber took place in Brazil, mainly in the center-southern States. At that time all the applications in the field were made by the private sector in toll road networks. The asphalt used was, in most cases, processed in terminal blends using at least 15% of crumb rubber. Still today 90% of the applications are done using dense graded mixes. In some cases, where gap graded mixes were applied, the amount of crumb rubber in the asphalt reaches up to 20%.

This paper will focus on the work performed by two big Concession Companies in the state of São Paulo who believed in the technology and applied it on most of their network pavement extension. They are **SPVias – Rodovias Integradas do Oeste S.A. and Ecovias**.

Spvias is a toll road company located the southern west region of the state of São Paulo and has approximately 516 km of highway with almost 1.500 km in traffic lane in its network. Their roads have a traffic varying from 5.000 to 25.000 vehicles a day in single lanes and double lanes per direction.

Ecovias belongs to the toll Company Group **Ecorodovias**, which operates three more concession companies in Brazil. It is one of the most important highway management company in the State of São Paulo. The major roads, Via Anchieta and Imigrantes, link the city of São Paulo to the coast, where are the most beautiful beaches of São Paulo state. These roads also connect a large portion of the southern road network to the biggest port in South America, the port of Santos. They hold over 30.000.000 vehicles per year.

Copavel is both responsible for the pavement design and for the yearly pavement monitoring campaigns. This has enabled us to present this paper on behalf of the collected data and assessed results which confirmed the performances of the roads rehabilitated using crumb rubber binders.

2. Historical Data

The first application happened in 2001 after the first AR 2000 in Villamoura – Portugal. It began with trial sections in the south of Brazil in two Concession Companies: **Univias** in **Rio Grande** do Sul state and Rodonorte in Paraná State. In 2002 many trial sections were built on many other Brazilian States and it was after 2003 that the technology was applied in larger scale for road rehabilitation throughout the country.

In the beginning of 2002 the first gap graded mix application took place in São Paulo in Via Anhanguera, operated by **Intervias**. This road links São Paulo to the northern states of Brazil. With this gap graded mix a modified binder with 20% of crumb rubber was used. After 2003 the technology left the trial sections applications to reach large scale road rehabilitation projects and, since then, 15.000 tons of crumb rubber modified binder have been applied on almost 4.000 km of roads in Brazil.

The first large scale gap graded mix application was done in 2003 on **Intervias** concessionaire's roads, with almost 100 km of roads rehabilitated using a 20% crumb rubber asphalt mix. Although they have achieved very good results and best pavement performance ever, the Concession was sold to a different group which has changed the maintenance plan stopping the use of crumb rubber binders.

In 2003 **Spvias** started the use of crumb rubber binder and has not stopped since then. Over 23.000 tons of crumb rubber binders have been applied on almost 600 km of single lane roads and they have plans to overlay their whole network with it. Nowadays this is the biggest private user of crumb rubber binders in Brazil.

Ecovias started their trial sections in 2004 and today are the biggest user of gap graded mix in Brazil. The major application have taken place after 2006 on the main roads of their network, Via Anchieta and Rodovia dos Imigrantes. Both roads link São Paulo city to the coastal city of Santos, where the port is. The table below presents an estimate of crumb rubber binder consumed since the year of 2001 and also an estimate of the number of tires removed from the environment.

Table 1. *Information provided by Greca Asfaltos do Brazil crumb rubber terminal blend producer*

Year	Quantity of Crumb Rubber Asphalt (ton)	Approximately Road Lengh in km	Tires removed from environment
2001	152.42	3.92	3,918.25
2002	732.25	18.82	18,823.78
2003	15,304.68	393.44	393,436.50
2004	8,024.70	206.29	206,290.49
2005	16,550.02	425.45	425,450.39
2006	28,214.09	725.30	725,297.94
2007	17,657.15	453.91	453,911.31
2008 (Jan. to Nov.)	21,246.67	546.19	546,186.89
TOTAL	107,881.98	2,773.32	2,773,315.55
Considerations:	Asphalt Pavement mat: 4 cm thick, 7m wide, Asphalt content 5,9% (density 2,7 ton/m^3)		

3. Problems and Difficulties During Crumb Rubber Mixes Applications

Some of the problems encountered on the crumb rubber applications help explain the direction that the technology has followed in Brazil in these years.

First of all and the most important, there is the fact that the Brazilian road building industry was not well prepared to deal with such a high viscosity binder. With the addition of 20% of crumb rubber to the binder its viscosity reached 2.000 cP. at 175 °C or more. The asphalt plants found it difficult to pump the binder continuously and homogeneously. The asphalt mix production was always stopping due to blocking in the binder pumping system.

This fact led the crumb rubber asphalt producers to lowering the viscosity of the binder in order to facilitate the production of the asphalt mix. Even though the plants were later adapted with more powerful pumps and stronger tubes, the lower viscosity binders remained as the preferred option to reduce the production problems in the field. For that reason the use of 15% of crumb rubber in the binder modification became almost a standard in Brazil.

The 20% crumb rubber content in binders is used only in the cases when the wearing courses has to be built with gap graded mixes to improve macro-texture and friction coefficients. In such cases the problem turns to be the available aggregates. In São Paulo State, where environmental issues are of major concern nowadays, the opening of new quarries was not always possible and the existing ones usually sell aggregates for very different purposes such as Portland Cement Concrete, for instance.

A specific gradation and a good shape aggregate was not always that easy to find and usually was more expensive than the regularly available aggregate. There are some regions where, even though the rock has good mineralogical nature, the produced aggregate is not very good. In the interior of the State of São Paulo there are good basalt rock quarries but the shape of the produced aggregates is not so good. They are usually flat and not adequate for the use in discontinuous grading asphalt mixes. That is also why dense graded mixes are the most popular in our Country.

Due to such particularities the crumb rubber asphalt mixes have had lower asphalt content when compared to traditional asphalt rubber mixes. These facts changed the original concept of the Asphalt Rubber in Brazil, which is based on very high viscosity binders with high asphalt content in the mixes. With the arrival of the in situ crumb rubber mixes very high viscosity binders will be produced in even larger scales. This will force the industry to re-adapt themselves to work with binders with viscosities around 4.000 CP. to 5.000 CP at 175°C.

4. Pavement Performance In Spvias Network

After more than 600 km of equivalent single lane roads have been rehabilitated, this paper will focus on the results obtained from two particular applications; the first and older one is on SP-255 highway, where a dense graded mix was applied over a cracked cement treated base. The second application was on SP-270 highway where a gap graded mix was applied following the California Department of Transportation (Caltrans) gradation.

4.1 Asphalt Crumb Rubber application in SP-255 Highway.

In 2002 the pavement design was conceived and it defined that the extension from Km 279,3 to Km 288,2 should have an 8 cm thick overlay for the 2,63 x 10 6 Aashto ESAL's after milling out the existing asphalt surface layer.

In March 2003 the section was rehabilitated using a Dense graded mix with 15% of crumb rubber binder. The mix characteristics are shown below.

SPVIAS - SP-255 Highway - Crumb Rubber Asphalt Mix										
Asphalt Institute " Mix 4B "										
Sieves	Sieves mm	Min	Max	MIX	Sand	Filler	3/4"	3/8"	Crushed Dust	P%Talbot 3/4
		4B	4B		15	0	18	25	42	
# 200	0.075	4	10	8.4	0.4	97	0	1	14.7	8.4
# 80	0.18	10	18	12.6	1.5	99.1	0	1	22.8	12.4
# 40	0.42	15	25	22.1	17.7	100	0	1.4	32.3	18.1
# 10	2.4	33	48	43.7	89.9	100	0	1.7	61.5	39.6
# 4	4.8	50	70	61.2	99.4	100	0	4.8	93.6	53.9
1/2"	12.7	80	100	97.9	100	100	36.7	100	100	83.3
3/4"	19.1	100	100	100.0	100	100	100	100	100	100.0
1"	25.4	100	100	100.0	100	100	100	100	100	100.0
1 1/2"	38.1	100	100	100.0	100	100	100	100	100	100.0
2"	50.8	100	100	100.0	100	100	100	100	100	100.0

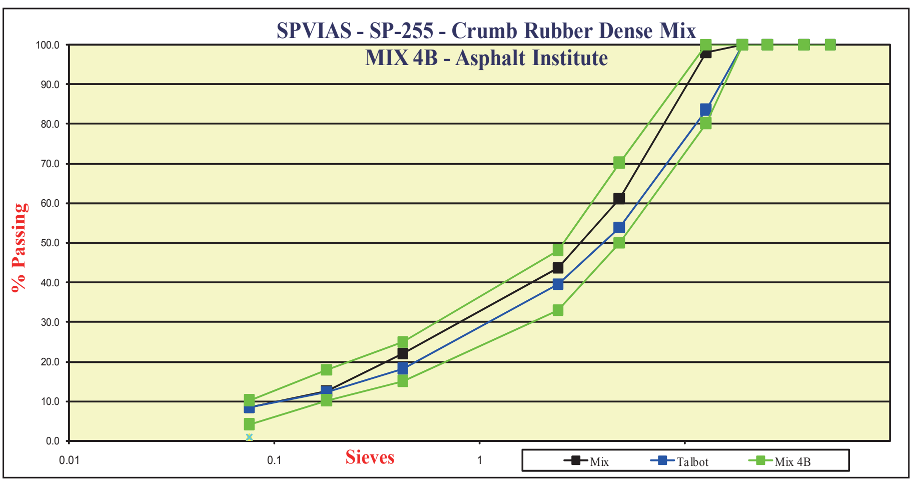

Figure 1. *Dense Graded Mix used in SPVias SP-255 Overlay Project in 2003*

Table 2. *Dense Graded Marshall Characteristics*

Marshall Characteristics of Crumb Rubber Modified Binder Dense Graded Mix used in SP-255 Rehabilitation Project		
Description	Unit	Values
Density	ton/m^3	2,495
Asphalt Content	%	5.90%
Stability	Kgf	1050 kgf/cm^2
Flow	mm	14.8
% Air Voids	%	3.80%
% Asphalt Voids Ratio	-	78
Compaction Temperature °C	°C	160 °C to 165°C
Crumb Rubber Content in Asphalt	%	15%

Table 3. *Crumb Rubber Binder Characteristics*

Crumb Rubber Modified Binder Characteristics in DENSE GRADED MIX SP-255 Rehabilitation Project		
Crumb Rubber Content in Asphalt	%	15%
Rotational Viscosity - Haake Viscometer - Sabita (BR 5 T)	Pa.s - 175°C	1.20
Ball Penetration and Resilience Sabita (BR2 T)	(0,1) mm	19.0
Compression Recovery - Sabita (BR3 T)	% - 5 min	91
	% - 1 hour	88
	% - 4 days	54
Flow Test - Sabita (BR4 T)	mm	58.3
Softening Point - (Astm - D 36)	°C	54

Below are some pictures of the Rehabilitation Project.

Figure 2. *Pavement surface after milling*

Figure 3. *Asphalt mix being placed*

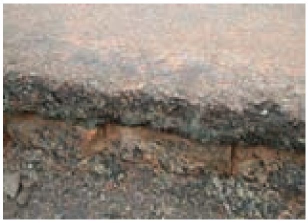

Figure 4. *Severe cracking in the CTB*

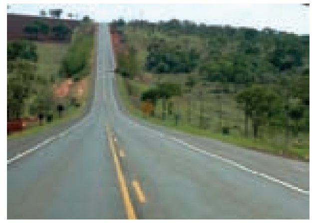

Figure 5. *Good results after 5 years*

4.1.1 Pavement Monitoring Results of SP-255 Project

The pavement has been frequently evaluated since its rehabilitation in 2003. Even though block cracking of the cement stabilized base was very intense, it has not migrated to the surface so far. The pavement structural capacity and ride quality have improved a lot after the rehabilitation, as can be seen in the graphs below.

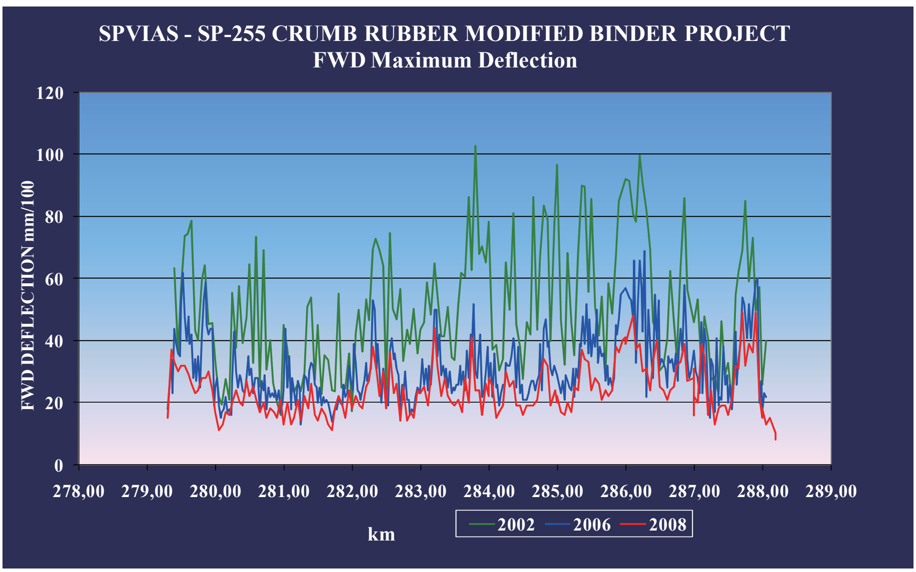

Figure 6. *FWD Deflections*

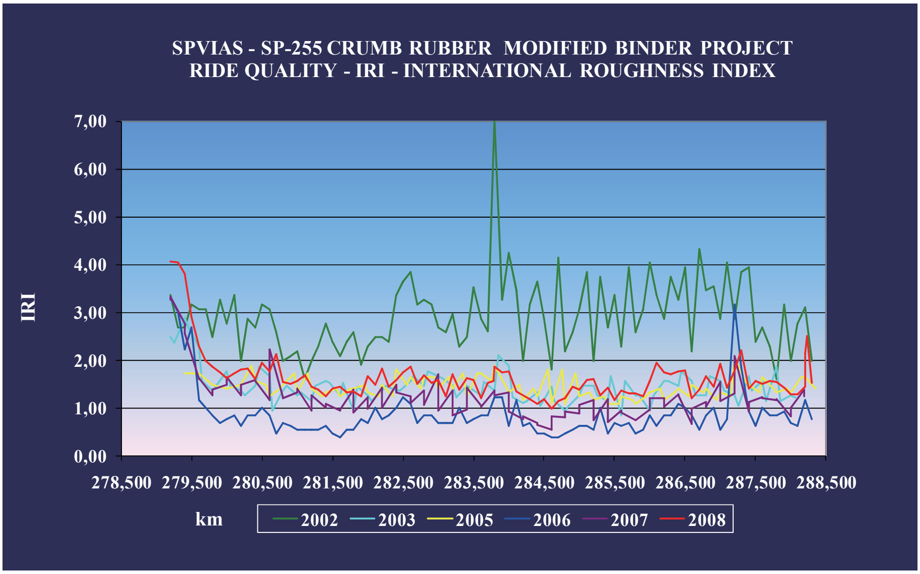

Figure 7. *IRI Measurements*

4.2. Asphalt Crumb Rubber application in SP-270 Highway.

It was possible to find good aggregates for this project and the mix was designed as a gap graded one according to the California Department of Transportation - Caltrans guidelines.

The pavement design was done in 2004 for an estimated traffic of for 8,86 x 10 6 AASHTO ESAL's, an 8 year service life. The pavement condition was bad with alligator cracks and permanent deformation in localized areas. These areas were repaired with a mill and fill technique reaching a depth of 50 mm using conventional dense graded mix. After that, the overlay was applied with a varying thickness from 30 mm to 40 mm in the critical sectors.

The mix was applied in the end of 2004 and its properties are presented in Figure 8 below.

SPVIAS - SP-270 Highway - Crumb Rubber Modified Binder Mix CALTRANS GRADING								
Sieves	Sieves mm	Min	Max	MIX	Filler	5/8"	3/8"	Crushed Dust
		CALTRANS	CALTRANS		2	15	61	22
# 200	0,075	2	7	5,1	97	0,7	1,4	9,8
# 100	0,149	5	10	6,3	99,1	0,8	1,6	14,8
# 50	0,297	7	15	9,1	100	0,9	1,9	26,4
# 30	0,595	10	20	12,2	100	0,9	2,1	39,8
# 8	2,38	15	25	21,8	100	1,1	3	80,9
# 4	4,76	28	42	34,6	100	1,6	17,9	97,4
3/8"	9,51	78	92	90,0	100	35,5	99,5	100
1/2"	12,7	90	100	94,3	100	61,8	100	100
5/8"	15,9	100	100	100,0	100	100	100	100
3/4"	19	100	100	100,0	100	100	100	100

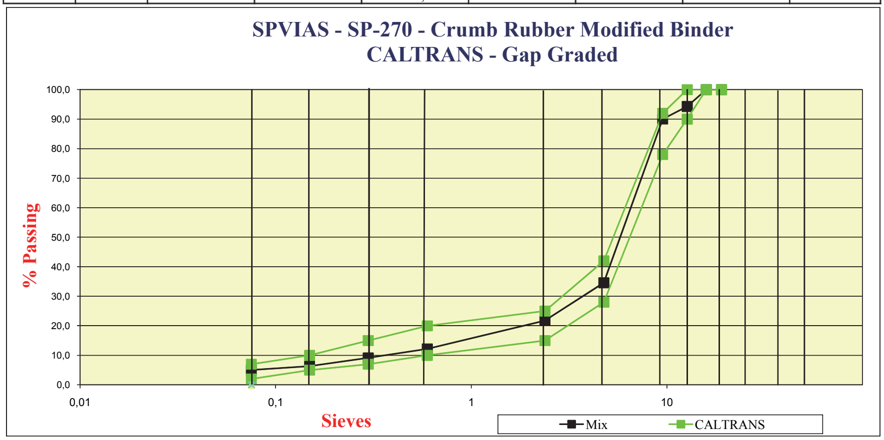

Figure 8. *Caltrans Grading used in SP-270 Rehabilitation Project*

Table 4. Gap Graded Marshall Characteristics

Marshall Characteristics of Crumb Rubber Modified Binder Mix used in GAP GRADED SP-270 Rehabilitation Project		
Density	ton/m^3	2.293
Asphalt Content	%	6.20%
Stability	Kgf	780
Flow	mm	13.6
% Air Voids	%	5.0%
Asphalt Voids Ratio	-	75
Compaction Temperature °C	°C	165 °C to 175°C
Crumb Rubber Content in Asphalt	%	20%

Table 5. *Crumb Rubber Binder Characteristics on Gap Graded Mix*

Crumb Rubber Modified Binder Characteristics in GAP GRADED SP-270 Rehabilitation Project		
Crumb Rubber Content in Asphalt	%	20%
Rotational Viscosity - Haake Viscometer - (Sabita BR 5 T)	Pa.s - 175°C	2.56
Ball Penetration and Resilience (Sabita BR2 T)	(0,1) mm	27.0
Compression Recovery - (Sabita BR3 T)	% - 5 min	93
	% - 1 hour	84
	% - 4 days	48
Flow Test - (Sabita BR4 T)	mm	30
Softening Point - (Astm - D 36)	°C	60

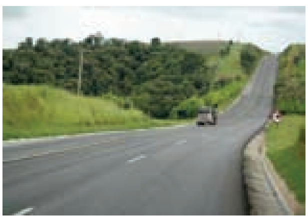

Figure 9. *SP-270 Highway with Gap Graded Mix in Place*

Figure 10. *Macro-texture detail of the mix applied*

4.2.1 Pavement Monitoring Results of SP-270 Project

The pavement has been evaluated since its rehabilitation in 2004. After the rehabilitation, the structural capacity, the riding quality and the skid resistance have all improved on the road. There are some friction measurements using the Grip Tester which are presented in the graphs below.

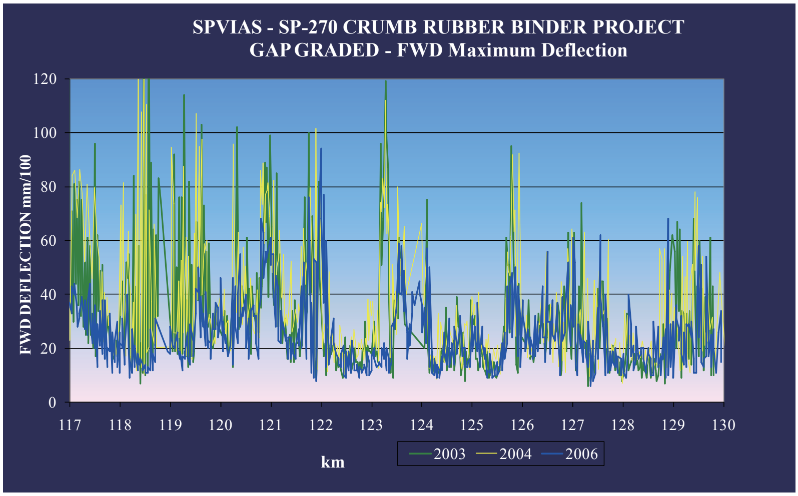

Figure 11. *FWD Maximum Deflection*

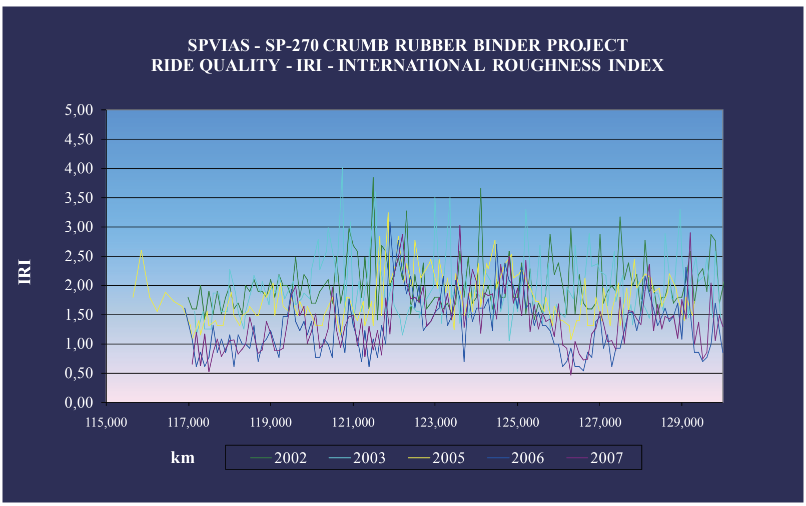

Figure 12. *IRI – International Roughness Index*

There was a good structural gain in the first four kilometers of the section but around the km 122 the deflections remained high due to drainage localized problem. This was the reason why the pavement presented some premature failure in 2006, over this precise extension. After solving the drainage problems the pavement was patched and it has been performing well since then.

It is important to point out that the first three IRI surveys from 2002 to 2005 were done with a Roughness Response Type Meter and the new ones after 2006 were done using a Laser Profiler which, in this case, presented lower IRI values.

It was also possible to verify the levels of skid resistance provided by a gap graded mix; the improvement is remarkable if compared with a dense graded mix. The latest results are shown below.

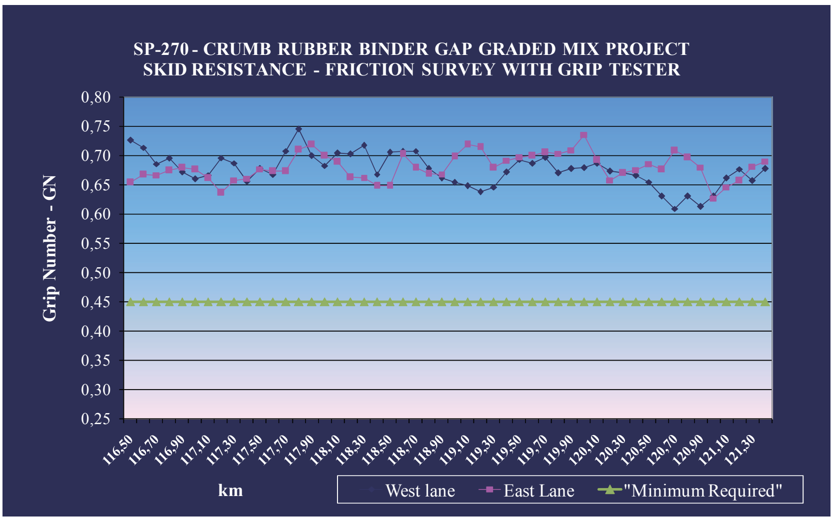

Figure 13. *Friction Testing with Grip Tester device in 2005 after 6 months.*

5. Pavement Performance In Ecovias Network

The other big project that is taking place in São Paulo is in Ecovias concessionnaire network. The major roads of their network, Via Anchieta and Rodovia dos Imigrantes are the link between the city of São Paulo which has an altitude of 800 m above sea level, and the coast and both of them are being rehabilitated using crumb rubber modified binder mixes. After doing the first trial section in 2004, they started implementing crumb rubber modified asphalt in all new flexible pavement rehabilitation projects in their network.

5.1. Crumb Rubber Modified Binder Projects in Ecovias.

In Anchieta highway, where the first big crumb rubber asphalt mix application took place, the road geometry is very sinuous and the traffic is one of the heaviest in São Paulo State. In 2005 the pavement design, considering a period of 4 years determined that the section from the km 40 to 55 should have an 3 cm overlay for 2,65 x 10 7 Aashto ESAL's, after some patching in the existing asphalt surface layer.

The pavement structure is composed of a rigid pavement built in the sixties overlayed by multiple dense graded asphalt mixes. Due to the stiffness of the pavement, the deflections are usually very low and the major concerns were the crack reflection from the slab joints and also a design a very rigid mix designed to be rut resistant and cope with all the heavy traffic. The pavement deflections were not evaluated in this case. Table 14 below presents the mix Marshall Characteristics.

Table 14. *Gap Graded Marshall Characteristics of Ecovias's projects*

Marshall Characteristics of Crumb Modified Rubber Gap Graded Mix used in SP-150 Rehabilitation Project - Via Anhanguera		
Description	Unit	Values
Density	ton/m^3	2,326
Asphalt Content	%	6.0%
Stability	Kgf	1065 kgf/cm^2
Flow	mm	10
% Air Voids	%	5.0%
% Asphalt Voids Ratio	-	72.5
Compaction Temperature °C	°C	165 °C to 170°C
Crumb Rubber Content in Asphalt	%	18%

During the mix production they had difficulties pumping the high viscosity binder in the asphalt plant and decided to limit the viscosity to 1,8 Pa.s – 175 °C. The crumb rubber binder was then modified to meet the contractor specifications.

In this case the amount of crumb rubber in the asphalt rubber binder was reduced to 18%. Normally we use a minimum of 20% of crumb rubber in Brazil when applying in a gap graded mix. The following table presents the crumb rubber binder characteristics.

Table 15. *Crumb rubber binder Average Characteristics used in Ecovias's Mixes*

Crumb Rubber Binder Characteristics in GAP GRADED Via Anhanguera - SP-150 Rehabilitation Project		
Crumb Rubber Content in Asphalt	%	18%
Rotational Viscosity - Haake Viscometer - (Sabita BR 5 T)	Pa.s - 175°C	1.80
Ball Penetration and Resilience (Sabita BR2 T)	(0,1) mm	24.0
Compression Recovery - (Sabita BR3 T)	% - 5 min	93
	% - 1 hour	85
	% - 4 days	51
Flow Test - (Sabita BR4 T)	mm	60
Softening Point - (Astm - D 36)	°C	56

The Anchieta highway section is located in a very humid forest area, and the mix grading was adjusted to reduce the permeability of the surface layer. Due to the very heavy traffic, the officials from concessionaire did not want a high content of asphalt in the mix to reduce the risk of bleeding and rutting.

During the mix design, the mixes were tested in The São Paulo State University in an accelerated rutting device to evaluate the rutting versus asphalt content. The results are presented below.

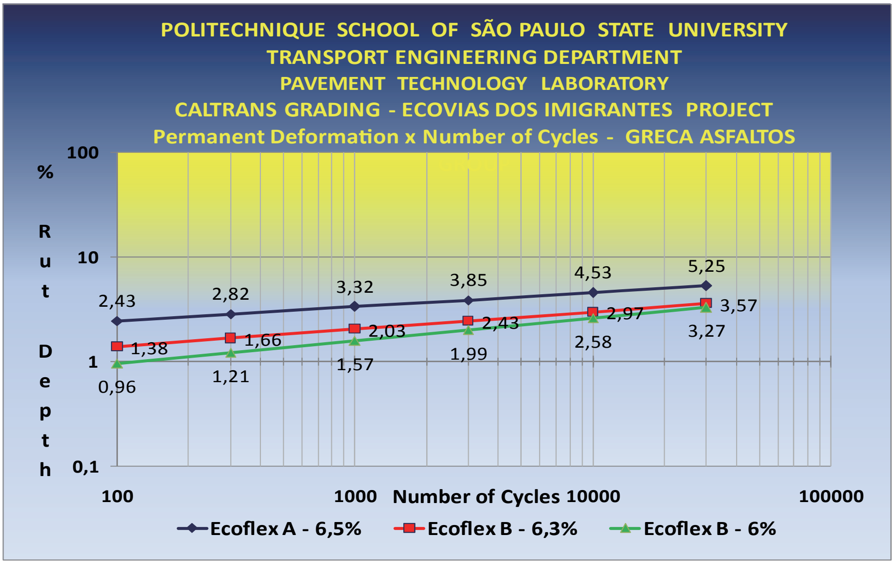

Figure 14 – Rutting accelerated testing Results

The best results were achieved for 6% of binder content with Ecoflex B (18% of crumb rubber) which was selected as the optimum value for the mix design. This asphalt content generated 5% air voids in the mix. In 2006 the section was rehabilitated using a Caltrans Gap grading mix with characteristics as shown below.

ECOVIAS - SP-150 and SP-160 Highways									
Via Anchieta and Rodovia dos Imigrantes									
Crumb Rubber Asphalt Mix -Caltrans Gap Graded									
Sieves	Sieves mm	Min	Max	MIX	Sand	Filler	5/8"	3/8"	Crushed Dust
		CALTRANS	CALTRANS		0	2	15	61	22
# 200	0,075	2	7	5,5	0,4	97	0,7	1,4	9,8
# 100	0,149	5	10	8,4	1,5	99,1	0,8	1,6	14,8
# 50	0,297	7	15	11,2	17,7	100	0,9	1,9	26,4
# 30	0,595	10	20	14,3	89,9	100	0,9	2,1	39,8
# 8	2,38	15	25	20,5	99,4	100	1,1	3	80,9
# 4	4,76	28	42	35,8	100	100	1,6	17,9	97,4
3/8"	9,51	78	92	86,3	100	100	35,5	99,5	100
1/2"	12,7	90	100	98,1	100	100	61,8	100	100
5/8"	15,9	100	100	100,0	100	100	100	100	100
3/4"	19	100	100	100,0	100	100	100	100	100

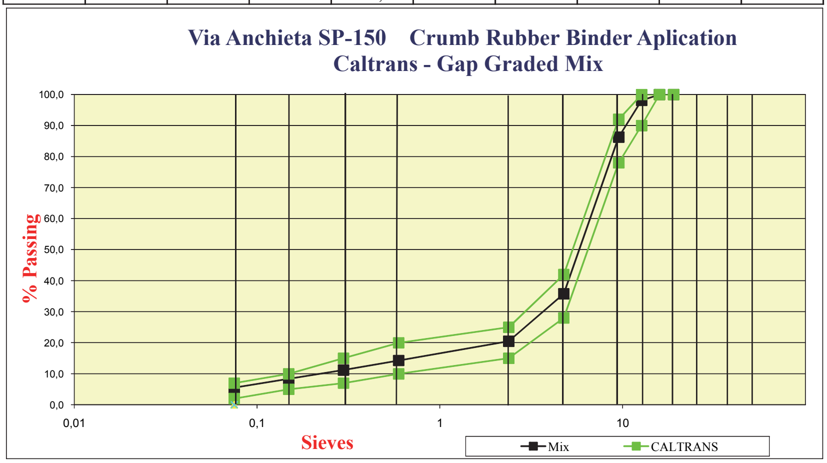

Figure 15. *Caltrans Grading used in SP-150- Via Anchieta Rehabilitation Project*

After this first application in 2006 there has been a lot more applications in Ecovias network in their main highways, other sections of Anchieta and Imigrantes. We are presenting some pictures of the sections that have been rehabilitated since the year of 2006.

Figures. *16 -17 – SP-150- Via Anchieta Gap Graded 2006 Rehabilitation Project*

Figures 18 -19. *SP-160- Imigrantes Gap Graded 2007 Rehabilitation Project*

Figures 20-21. *SP-150- Via Anchieta Gap Graded 2008 Rehabilitation Project*

6. Conclusions

After these seven years of experience we can conclude that crumb rubber modified binders represent a great contribution to the pavement rehabilitation technology. Even though we cannot obtain the same traditional "asphalt rubber" properties using the terminal blend binders, all the applications have presented outstanding performances so far, when compared with conventional asphalt mix applications.

From the beginning, the crumb rubber binder use was restricted to the private sector in toll concession companies, but now they are reaching several Highway State Departments as well as the Federal Highway Department in Brazil. In this period Brazilian specifications were developed to control the crumb rubber binder characteristics and its applications.

The so called "new" technology barrier has been broken and the road community is more comfortable using these binders. Pavement designers are recommending its use more often, and the solutions are being easily accepted by the decision makers based on good results presented.

A lot of research has been done since the first application in 2001 with good results using both 15% or 20% crumb rubber modified binders. The results have proven that the use of crumb rubber in asphalt mixes is a good pavement rehabilitation alternative to make the pavements last longer and reduce the maintenance costs. The examples presented in this paper are from Spvias and Ecovias, road concession companies who decided to have their entire flexible pavement network rehabilitated with crumb rubber binder mixes. It shows that the benefit cost ratio is very positive for them no matter the amount of traffic they have on their roads.

The crumb rubber modified binder is a powerful tool to reduce reflective cracking but, due to the Brazilian tropical weather with a lot of rain, prior milling is imperative to guarantee a good pavement performance. Recent research (Ceratti, 2006) has indicated that reflective cracking on 15% crumb rubber modified dense graded mixes overlay can be 5 to 6 times slower than the reflective cracking when using conventional dense mixes.

From now on, after the terminal blend has been accepted and the road industry has adapted itself to deal with high viscosity binders, it is a good time to start doing the crumb rubber mixing on the job site to have the traditional asphalt rubber, which will give us even better results. The in situ mixers have just arrived in Brazil and we hope to present in the near future, new results of "asphalt rubber" applications.

7. Acknowledgements

The author would like to thank Mr. Helvecio Soares and Mr. Paulo Pimenta, Spvias President and Engineering Director for supporting the crumb rubber binder applications and the preparation of the paper. Thanks to Mr. Humberto Gomes, Ecovias President, for allowing the presentation of their results, and also thanks to Marcos Greca who supported and supplied important information to the preparation of the paper. A special thanks to Savio Vasconcellos who has helped on special advices and English revision.

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Studies on Adhesive Performance of Waste Crumb Rubber Modified Asphalt Mixtures

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ABSTRACT. At present, material recycling has become one of the most attractive science alternatives, and different recycling methods are now available to specific pavement capability and structural needs. The method that incorporates waste crumb rubber into asphalt concrete was studied. The objective of this study was to investigate and evaluate the fatigue properties in two of the most important factors, that is crumb rubber modifier size and temperature which influence the performance of asphalt concrete greatly, were discussed by Direct Tensile Test in different temperature. Using UTM25 electro-hydraulic servo-universal testing machine, Uniaxial Direct Tensile Test was conducted on cylindrical specimens with different linearly strain loading rate at different temperature. Through Direct Tensile Test, the result showed the adhesive strengths were significantly enhanced. The recycling of existing waste crumb rubber produces new pavements with considerable savings in material, money, and energy. They have been used for many years, proving to be both economical and effective in protecting the environment. Furthermore, waste crumb rubber modified asphalt concrete have been found, for the most part, to perform as well as other modified asphalt.

KEYWORDS: Waste crumb rubber; modified asphalt; Direct Tensile Test; adhesive strength

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1. Introduction

The use of various CRM in asphalt mixtures has increased significantly in recent years around the world.[1-2] Research has shown that the addition of crumb rubber to base asphalt produces binders with improved resistance to rutting, fatigue cracking, and thermal cracking as well as reducing the thickness of asphalt overlays and reflective cracking potential. Fatigue behavior of rubberized mixtures was found to be significantly improved compared to conventional mixtures. [3-6]The previous research indicated that the rubber particle size is important factor in affecting the rheological and engineering properties of mixtures because of the interaction of CRM with the base binders. This interaction changes the physical properties, viscosity and rheological properties of the CRM modified binder, leading to a high resistance to the rutting of pavements [7-8]. The interaction process of CRM with binder is, essentially, rather complicated, depending on the variables of CRM and binder (i.e., type, percentage, size, grade, etc.) in addition to the mixing condition (temperature, mixing time, etc.). There have been a few of researches identifying the influence of these individual variables on the interaction. From these literatures, it was concluded that swelling of the rubber particles due to the absorption of light fractions into the rubber particles, and stiffening of the residual binder phase are the main mechanism of the interaction [9].

The objective of this study is to evaluate the anti-drawing properties of CRM asphalt mixtures as a function of different size and temperatures using different methods in the laboratory and suggest amount, size and temperature to satisfy the properties required in the mix designs.

2. Experimental

2.1 Materials

Fine crumb rubber is made from waste wheel through crushing, which main component is natural rubber (NR) and styrene-butadiene rubber (SBR) and rubber content is almost 55%. And classified by grain size, pulverizing crumb rubber (12-30 mesh), comminution pulverizing crumb rubber h(30-47 mesh), fine crumb rubber(47-200 mesh)and super fine crumb rubber(more than 200 mesh). The asphalt binder tested were base asphalt and CRM asphalt prepared by ourselves with fine crumb rubber(CRM Asphalt-1) and super fine crumb rubber(CRM Asphalt-2) at 142℃, whose specifications was showed in table 1.

Table 1. *Specifications of asphalt binder*

	Item	Results	Index
Base asphalt	Penetration/0.1mm	64	60-80(25℃)
	Softening point/℃	48.5	44-54
	Ductility/cm	＞100	≥100(15℃)
CRM Asphalt-1	Penetration/0.1mm	78	60-80(25℃)
	Softening point/℃	56.8	≥55
	Ductility/cm	27.4	≥20(5℃)
CRM Asphalt-2	Penetration/0.1mm	82	60-80(25℃)
	Softening point/℃	60.4	≥55
	Ductility/cm	32.8	≥20(5℃)

Gneiss coarse aggregates and fine aggregates (Bulk specific gravity of 2.68 and 2.72, respectively) were used for the asphalt mixture. Limestone was selected as mineral filler. The gradation type of AC 25 was selected for asphalt mixture. Mineral filler content is 2% by weight, and preliminary asphalt content is 4.0%. Figure 1 shows the selected mixture gradation curve.

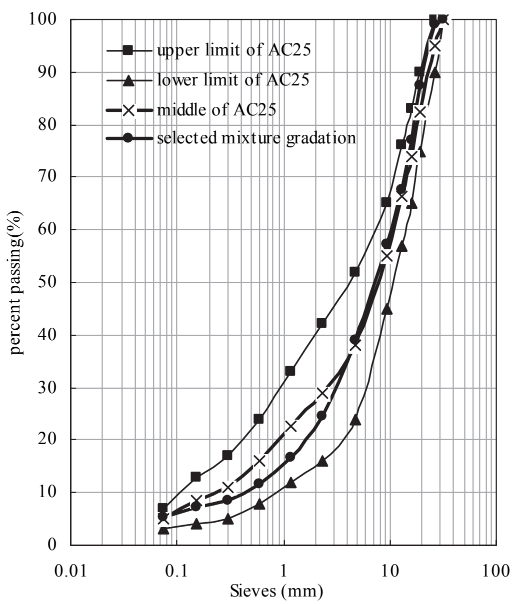

Figure1. *Selected mixture gradation curve*

2.2 Direct Tensile Test

The test used to evaluate the adhesion properties of the asphalt mixtures was a simple Direct Tensile Test. The type of loading is like the one used to evaluate the asphalt-aggregate interaction. In Figure 2 it can be seen a specimen being tested in simple tension.

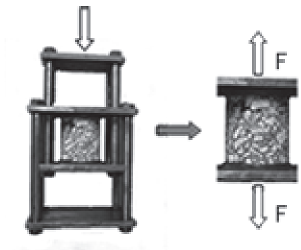

Figure 2. *Clamp used for DTT test*

In Figure 3 is the typical result of a tensile test. The specimens were tested in simple tension on base asphalt mixture sample at a constant deformation rate of 0.5mm/min at 15℃, quickly reaching rupture and the maximum value of resistance. The overall results of the tensile tests regarding the influence of temperature and strain loading rate are presented in Figure 5-7.

Based on the Strategic Highway Research Program (SHRP), Direct Tensile Test was conducted at constant strain loading rate in this research [10,11]. The height/diameter ratio and the maximum aggregate particle size are related and affect the final results, so the height and diameter were both chosen 100mm at this research. The strain loading rate is 0.1, 0.5,1 and 18 mm/min. the experimental temperature is controlled at 15 and 25 ℃.

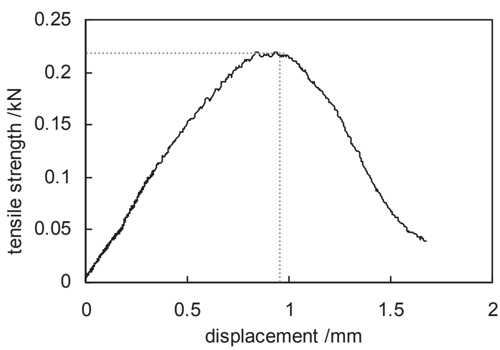

Figure 3. *A typical result of the DTT test (15 ℃, 0.5 mm/min)*

The test system used was a UTM25 electro-hydraulic servo-universal testing machine made by Instron LTD, England. The clamp used for DTT test was constructed by self, as noted in Figure 2.

2.3 Sample preparation

In this experiment, AC25 asphalt mixes was compacted into cylindrical samples. Optimum asphalt content was 4 percent by weight of mix, and Air voids ranged between 3 and 4 percent.

Asphalt concrete specimens were prepared by gyratory compaction using 130 mm diameter by approximately 120 mm high cylindrical specimens. After compaction, specimens were cored and then the ends were sawed, taking special care in the cutting of the faces in order to obtain fiat parallel faces. After this, we got specimens with 100 mm diameter and 100 mm high. The ends were cleaned before gluing them to the end caps with the epoxy resin.

Figure 4. *Process of sample preparation*

The sample preparation technique is importance in order to ensure that the failure plane occurs far enough from the end caps and perpendicular to the sample axis. During the final gluing step, we have pay especial attention to ensure proper alignment between the caps of clamp and the specimen axis. Otherwise, shear stress will occur in the test and influence the results.

Temperature control and experiment were all carried out in a controlled temperature chamber. Specimens shall be placed in this chamber before test more than 3 hours and monitored by a precise thermometer until its inner part has attained the test temperature.

3. Results and discussion

3.1 Direct Tensile Strength

Direct Tensile Strength (DTS) is considered to be a very important performance characteristic of asphalt pavement. The DTT test is carried out to define the tensile characteristics of the asphalt concrete which can be further related to the cracking and adhesion properties of the pavement [12].

According to the maximum load carried by a specimen at failure, the following equation is used to compute the DTS:

$$\mathrm{DTS} = \frac{P_{\max}}{\pi \times r^2} \tag{1}$$

Where:
P_{max} is the maximum applied load of each stress-elongation curve, as Figure 2 shows;
r is the diameter of specimen.

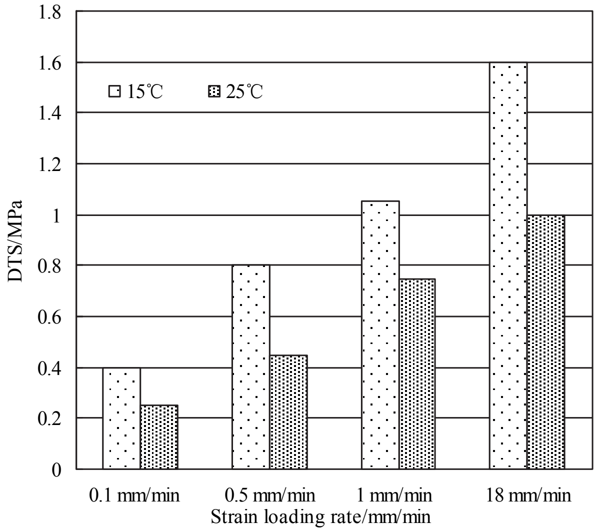

Figure 5. *DTS of base asphalt mixture*

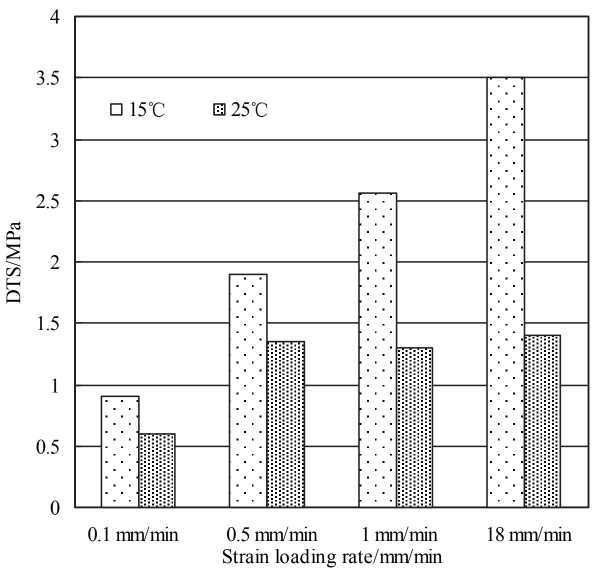

Figure 6. *DTS of CRM asphalt-1*

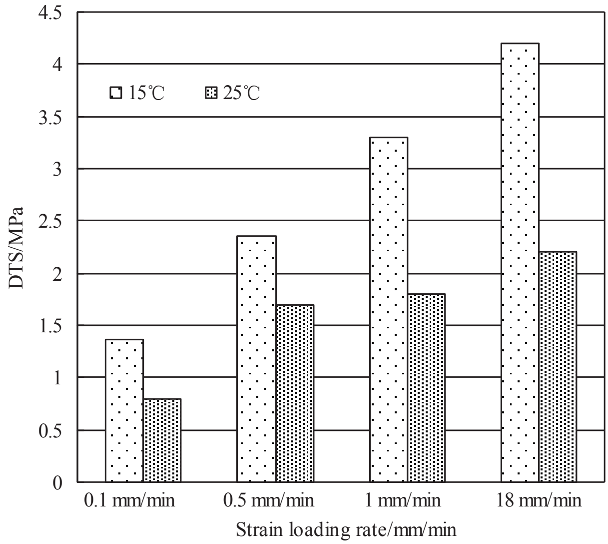

Figure 7. *DTS of CRM asphalt-2*

Figure 5-7 show the comparison of DTS of each group of asphalt mixtures. The results indicated that DTS decreases sharply with the temperature increase from 15℃ to 25℃ under the same strain loading rate. Both the base asphalt mixture and the CRM asphalt mixture have the same relationship with the temperature. Higher strain rate seems to increase the possibility of adhesive failure. At the same temperature, the percentage of adhesive failure at higher strain loading rate is much more than the percentage at lower strain loading rate. And the finer the crumb rubber size, the higher the adhesive strength of CRM asphalt.

4. Conclusions

Utilization of crumb rubber modification asphalt is effective and economical. Testing results were analyzed through Direct Tensile Strength by UTM25 to evaluate the adhesion properties of asphalt mixture. Conclusion can been drawn as follows:

1. The DTS decreases with the temperature increase from 15℃ to 25℃ at the same strain loading rate.

2. At the same temperature, the percentage of adhesive failure at higher strain loading rate is much more than the percentage at lower strain loading rate.

3. The finer the crumb rubber size, the higher the adhesive strength of CRM asphalt.

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Rheological and Engineering Properties of Rubberized Asphalt Concrete Mixtures Containing Warm Mix Asphalt Additive

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abstract: In recent years, warm mix asphalt (WMA) is widely used for reducing energy requirements and emissions in hot mix asphalt (HMA) industry. In addition, the use of rubberized asphalt in the past has proven to be economical, environmentally sound and effective in improving the performance of pavements across the U.S. and the world. WMA additives reduce the mixing and compaction temperatures and achieve ideal workability of HMA without significantly affecting the engineering properties of the mixtures. The objective of this research was to investigate the performance characteristics (e.g. rheological and engineering properties) of rubberized warm asphalt mixtures through a series of laboratory tests on binders (viscometer, dynamic shear rheometer (DSR), and bending beam rheometer (BBR) on the binders), and indirect tensile strength, rutting resistance, resilient modulus, and fatigue behavior of mixtures. The experimental materials included one virgin binder (PG 64-22), one crumb rubber modified (CRM) binder (PG 64-22 + 10% -40 mesh rubber), two aggregate sources, and two WMA additives (Aspha-min® and Sasobit®), and a total of twelve mixtures were used in this study. The results of the experiments indicated that the uses of WMA additives in rubberized HMA do not significantly reduce the rheological and engineering properties of these mixes at lower mixing and compacting temperatures than the conventional rubberized asphalt mixture.

*keywords: Crumb rubber, Warm mix asphalt, Viscosity, G*sinδ, ITS, Rutting, Stiffness, Fatigue life*

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1. Introduction

In recent years, the "warm mix asphalt" (WMA) is widely being promoted and used in the hot mix asphalt (HMA) industry as a mean of reducing energy requirements and lowering emissions. WMA can significantly reduce the mixing and compacting temperatures of asphalt mixtures, by either lowering the viscosity of asphalt binders, or causing foaming in the binders. Reduced mixing and paving temperatures would decrease the energy required to produce HMA, reduce emissions and odors from plants, and make better working conditions at the plant and the paving site (Kristjansdottir *et al*. 2007; Gandhi and Amirkhanian 2007; Wasiuddin *et al*. 2007; Prowell *et al*. 2007; Xiao *et al* 2009a).

At the same time, recycling of scrap tires has also been of interest in the asphalt industry throughout the world for over 40 years. Currently, approximately 82% of scrap tires are utilized for such applications as tire-derived fuel, molded products, crumb rubber, and other applications in the United States (RMA 2006). The mixing of crumb rubber with conventional binders results in an improvement in the resistance to rutting, fatigue cracking and thermal cracking (Button 1987; Way 2003; Xiao 2006). Many researchers have found that utilizing crumb rubber in pavement construction is effective and economical (Airey *et al*. 2003; Huang *et al*. 2004; Xiao *et al*. 2007; Xiao and Amirkhanian 2008; Xiao *et al*. 2009b).

However, the influence of crumb rubber and WMA additives mixed with virgin mixtures together has not yet been identified clearly. The interaction of modified mixtures with WMA additives is not well understood from the standpoint of binder properties and field performance. It has been shown that the WMA additives reduce the mixing and compaction temperatures and achieve ideal workability of HMA without significantly affecting the engineering properties of the mixtures (Hurley and Prowell 2006; Prowell *et al*. 2007; Gandhi and Amirkhanian 2007). While the addition of crumb rubber increases the demand of asphalt binder and significantly increases the mixing and compacting temperatures, the modified binder is helpful in resisting the high temperature deformation and extending the long-term performance of the HMA. Because of the complicated relationships of the crumb rubber and WMA in the modified mixtures, detailed information will be beneficial to help obtain an optimum balance in the use of these materials.

The objective of this study was to investigate the performance characteristics of the rubberized asphalt binder containing WMA additives and the rubberized asphalt concrete mixtures containing the WMA additives. Experiments were carried out to evaluate rheological properties of modified binder (i.e., viscosity, $G^*/\sin\delta$, $G^*\sin\delta$, and stiffness) as well as mechanical properties of the mixtures (i.e., indirect tensile strength, rutting susceptibility, and fatigue behavior).

2. Experimental design and procedures

2.1 Materials

One virgin binder (PG 64-22) and one crumb rubber modified (CRM) binder (PG 64-22 + 10% -40 mesh rubber) were used in this study. The rheological properties of CRM binder was discussed in results analysis paragraph. The PG 64-22 binder was a mixture of several sources that could not be identified by the supplier. One type of rubber, -40 mesh ambient rubber, was produced by mechanical shredding at ambient temperature. To ensure that the consistency of the rubber was maintained throughout the study, only one batch of crumb rubber was used in this study (Table 1). Previous research and field projects conducted in South Carolina indicated that the -40 mesh ambient rubber is effective in improving the engineering properties of rubberized mixtures. Therefore, the -40 mesh rubber was employed in this study. To prepare the modified binders, a reaction time of 30 minutes was considered suitable based on previous studies indicating that the mixing time did not significantly influence the binder properties (Putman 2005; Xiao *et al*. 2006).

Aspha-min® and Sasobit® were used in this study as the two WMA additives. Aspha-min ® is a Sodium–Aluminum–Silicate zeolite, which is hydro thermally crystallized as a very fine powder. It contains approximately 21% crystalline water by weight. By adding it to an asphalt mix, the fine water spray is created as all the crystalline water is released, which results in volume expansion in the binder, therefore increasing the workability and compactability of the mix at lower temperatures (Eurovia Services). Sasobit® is a long chain aliphatic hydrocarbon obtained from coal gasification using the Fischer-Tropsch process. After crystallization, it forms a lattice structure in the binder which is the basis of the structural stability of the binder containing Sasobit® (Sasol Wax). More detail information regarding the two additives can be found in other studies (Gandhi, 2008).

Table 1. *Crumb rubber gradation*

Sieve No. (mm)	Ambient rubber (-40 mesh)	
	% Retained	% Cumulative Retained
30 (600)	0	0
40 (425)	9	9
50 (300)	31.9	40.9
80 (180)	32.9	73.8
100 (150)	7.6	81.4
200 (75)	18.6	100

Two aggregate sources (A and B) were used for preparing the samples (Table 2). Aggregate A, granite source, is composed predominantly of quartz and potassium feldspar while aggregate B (schist) is a metamorphic rock. Hydrated lime, used as an anti-strip additive, was added at a rate of 1% by dry mass of aggregate. A total of 12 mixtures were evaluated in this research.

Table 2. *Aggregate Properties*

Aggregate Source	LA Abrasion Loss (%)	Absorption (%)	Specific Gravity			Soundness % Loss at 5 Cycles			Sand Equivalent	Hardness
			Dry (BLK)	SSD (BLK)	Apparent	11/2 to 3/4	3/4 to 3/8	3/8 to #4		
A	51	0.80	2.740	2.770	2.800	0.2	0.1	0.1	-	5
B	34	0.60	2.780	2.800	2.830	0.4	0.6	0.9	35	5

2.2 Experimental Procedure

The combined aggregate gradations for a 12.5 mm mixture were selected in accordance with the specification set by the South Carolina Department of Transportation (SCDOT). The combined gradations for each aggregate source (A and B) are shown in Figure 1, which shows that the design aggregate gradations for each aggregate source are the same when using different WMA additives (Asphmin and Sasobit) at the same percentages of rubber (0% or 10% rubber). The mixing and compaction temperatures of CRM mixture were 15-20 ℃ higher than warm mix asphalt mixtures that used 145-155 ℃ and 125-135 ℃ as mixing and compaction temperatures, respectively.

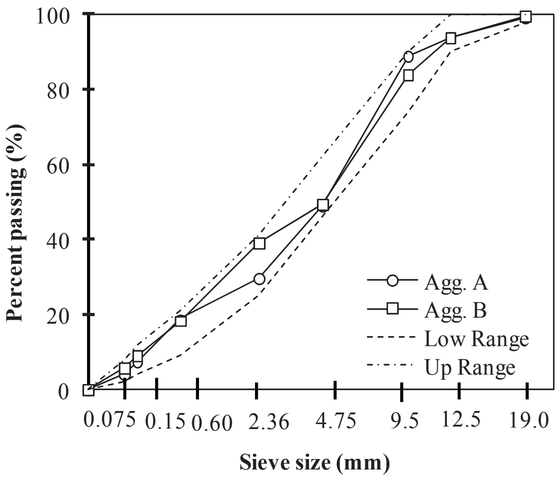

Figure 1. *Gradation of aggregate sources*

For this study, the optimum binder content, during the mix design process, was defined as the amount required to achieve 4.0% air voids at a given number of design gyrations (Ndesign= 75). Six indirect tensile strength (ITS) specimens, compacted to 7 ± 1% air voids, were used to evaluate the moisture susceptibility of various mixtures as modified AASHTO T283 procedures (no freeze/thaw cycle) were followed.

Six cylindrical Asphalt Pavement Analyzer (APA) specimens, for each mix type, were compacted to 4.0 ± 0.5% air voids using a Superpave gyratory compactor. All testing with the APA samples were carried out to 8,050 cycles to measure the rut depth of the HMA at 64℃. The testing temperature was based on the virgin binder's "performance grade" used in this study.

Fatigue beams were made in the laboratory and four beams of each mixture were tested for this study. All tests were performed in a temperature-controlled chamber at 20 ± 0.5℃. In this study, a repeated sinusoidal loading at a frequency of 10 Hz was used; in addition, the controlled strain mode was employed. The control and data acquisition software measured the deflection of the beam specimen, computed the strain in the specimen and adjusted the load applied by the loading device (AASHTO T321).

The test apparatus also recorded load cycles, applied load, and beam deflections. Failure is assumed to occur when the stiffness reaches half of its initial value, which is determined from the load at approximately 50 repetitions; the test is terminated automatically when this load has diminished by 50 percent. The initial flexural stiffness of fatigue beam is determined as follow (AASHTO T321):

$$S = \sigma / \varepsilon = \frac{aP(3l^2 - 4a^2)}{4b\delta h^3} \tag{1}$$

Where, S = initial stiffness; σ = tensile stress, in Pa; ε = maximum tensile stain, in m/m; P = applied peak-to-peak load, in Newton; a = space between inside clamps, in meters; b = average beam width, in meters; h = average beam height, in meters; δ = beam deflection at neutral axis, in meters; and l = length of beam between outside clamps, in meters.

Statistically analyzed with 5% level of significance (0.05 probability of a Type I error) with respect to the effects of aggregate sources and WMA additive types. For these comparisons, it should be noted that all specimens were produced at optimum binder content.

3. Test results and discussions

3.1 Binder analysis

Table 3 indicates that the viscosity of rubberized asphalt binder decreases while the high temperature performance (G*/sinδ) of overall binders increases with the addition of WMA additive. The unaged binder test result shows that the Asphamin and Sasobit can improve the workability (viscosity) and rutting resistance (G*/sinδ) of mixtures. The aged rubberized binders show that the G*sinδ values decrease with the addition of rubber but these values increase slightly as the WMA additives are added. It can be noted that the stiffness values of binders have similar trends with G*sinδ values due to the addition of these materials. Aged binder properties show that the WMA additives do not noticeably affect the long-term performance of asphalt binder. For two WMA types, Sasobit shows a slightly greater influence on the rheological properties regardless of the test conditions.

Table 3. *Rheological properties of binders*

Aging states	No aging		RTFO	PAV		
Test properties	Viscosity @ 135°C (Pa-s)	G*/sin(δ) @ 64 °C (kPa)	G*/sin(δ) @ 64 °C (kPa)	G*sin(δ) @ 25 °C (kPa)	Stiffness @-12 °C (MPa)	m-value @-12 °C
Control (PG 64-22)	0.405	1.243	3.295	2970	183	0.311
10% CRM	1.600	0.934#	2.450#	1705	129	0.320
10% CRM+Asphamin	1.477	1.196#	3.289#	2042	148	0.330
10% CRM+Sasobit	1.438	1.402#	3.325#	2160	151	0.290

Note: #~ testing at 76℃

3.2 ITS analysis

ITS values of mixtures made from aggregate sources A and B are shown in Figures 2 and 3, respectively, it can be noted that the dry ITS values generally are greater than the wet ones, as expected, but the mixtures containing WMA additive without rubber exhibit greater wet ITS values regardless of the aggregate source. For the mixtures from aggregate A, it can be found that the rubberized mixtures have slightly lower ITS values, conversely, the ITS values of rubberized mixture are slightly greater as using the aggregate B.

In addition, the ITS values of all mixtures presented in Figures 2 and 3 are greater than the value of 448 kPa (65 psi), a minimum wet ITS value set forth by SCDOT. Statistical analysis (t-statistics) shows that there are no significant differences in the ITS values of the rubberized mixtures containing two WMA additives. Similar results can be obtained from the control mixtures. In addition, in comparison with two aggregate sources, it can be noted that the mixtures from aggregate source A show lower ITS values regardless of the WMA additive or rubber content.

3.3 TSR analysis

The TSR results are presented in Figure 4. It can be found that the ITS values of the rubberized mixtures are lower regardless of aggregate sources and WMA types. As expected, the rubber exhibits a negative effect on moisture resistance of asphalt mixtures. However, these ITS values are greater than 85%, a minimum value of SCDOT's specification. In addition, for mixtures without rubber, it can be noted that the TSR values of those mixtures containing WMA additive are greater than 100%, it seems that WMA additive benefits the moisture resistance of these mixture regardless of the aggregate sources in this study. Moreover, statistical analysis shows that there are no significant differences in TSR values of the mixtures containing two WMA additives.

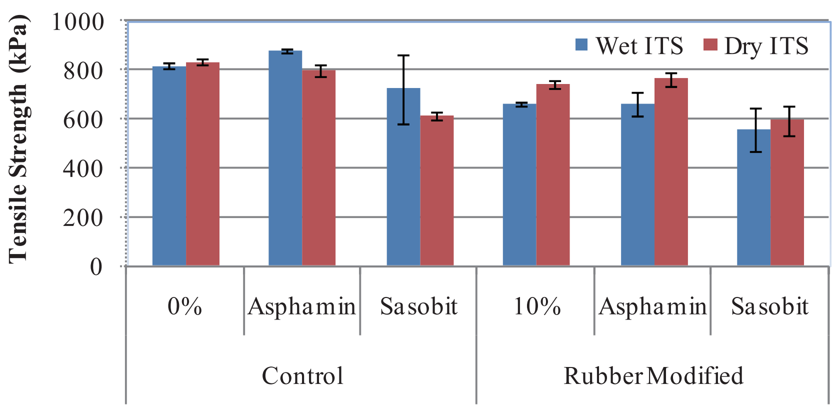

Figure 2. ITS values of mixtures made with aggregate A

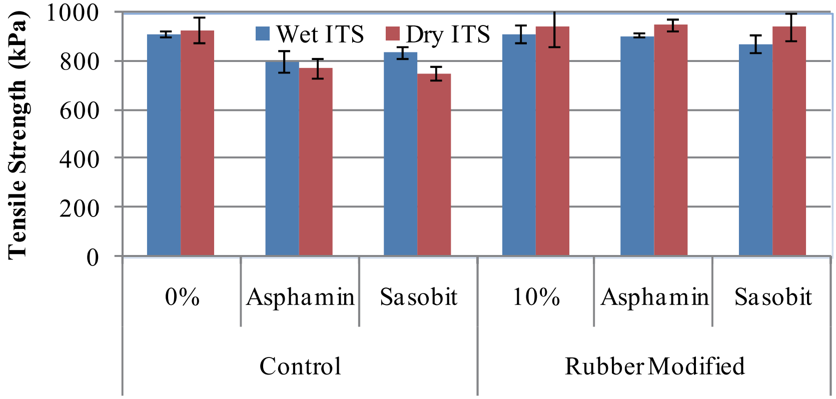

Figure 3. *ITS values of mixtures made with aggregate B*

3.4 Rut depth analysis

The rut depth results shown in Figure 5 indicate that, as expected, the rut depths of the rubberized mixtures have lower values than other mixtures. Previous research by Xiao *et al*. 2007 presented a similar result. In general, the mixtures from aggregate source A show greater rut depth, this indicates that aggregate properties play a key role in determining the rut resistance of the mixtures. The mixtures containing two WMA additives show no significantly different rut depth values at $\alpha = 0.05$ level. As a result, the effect of WMA additives on the rut depth of the mixture can be ignored in this study although the addition of these materials increases the $G^*/\sin\delta$ value of the binder.

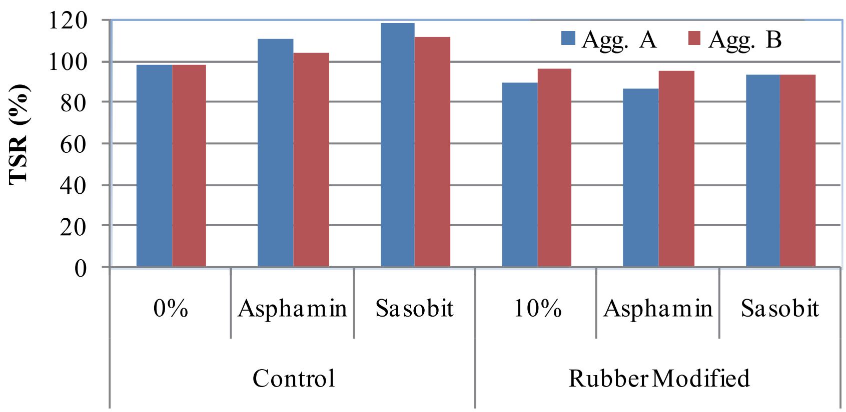

Figure 4. *TSR values of mixtures*

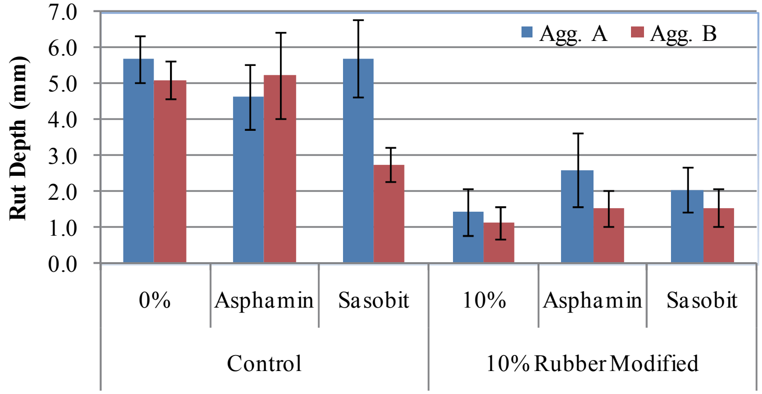

Figure 5. *Rut depth values of mixtures*

3.5 Stiffness analysis

As shown in Figure 6, it can be noted that the initial stiffness values (Equation 1) of the mixtures from aggregate source A are lower. The additions of rubber and WMA additive do not significantly affect the initial stiffness values of the mixtures. There is also no trend for these stiffness values based on the test results in this study.

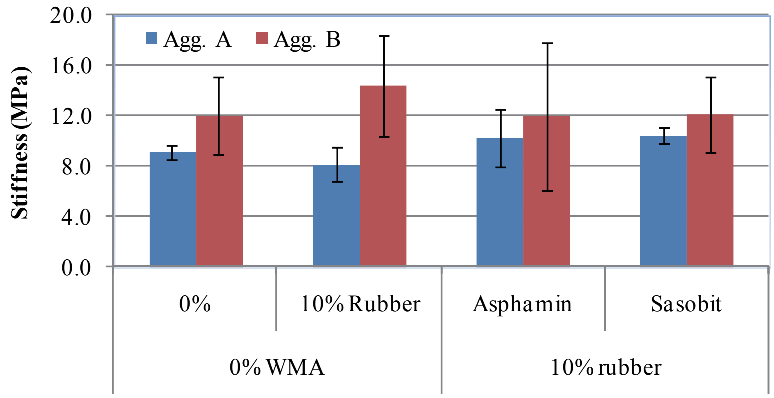

Figure 6. *Initial stiffness values of mixtures*

3.6 Fatigue life analysis

Fatigue life values of the mixtures are shown in Figure 7. Obviously, the standard deviations of the fatigue test results for each mixture are large since the variability of fatigue life is generally based upon the micro-structure of beams (e.g. the aggregate-binder interface, the void size distribution, the interconnectivity of voids, distribution of aggregate particles, film thickness and the aged status of binder). Previous research has indicated that increasing the number of the repeated specimens reduced the variability (Xiao *et al* 2007). Moreover, in comparison with the control fatigue beam, fatigue life of mixture shows no obvious trend when the rubber or WMA additive were added.

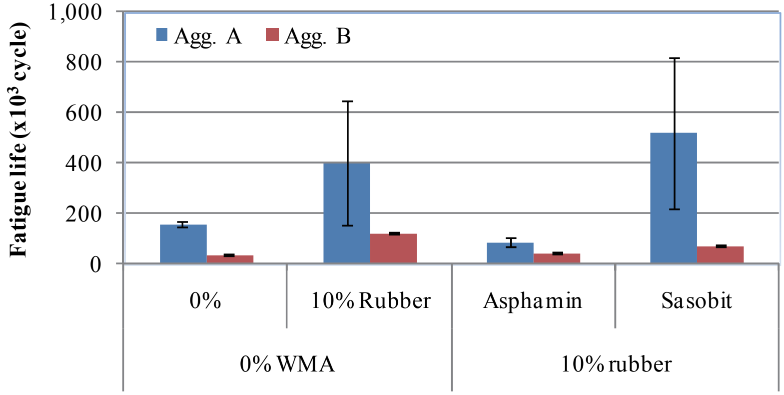

Figure 7. *Fatigue life values of mixtures*

4. Conclusions

Based on the experimental data shown in this limited study, the following conclusions are reached:

- The viscosity of rubberized asphalt binder slightly decreases while the high temperature performance (G*/sinδ) of overall binders slightly increases with the addition of WMA additive. The unaged binder test result shows that the Asphamin and Sasobit can improve the workability (viscosity) and rutting resistance (G*/sinδ) of mixtures.

- Statistical analysis shows that there are no significant differences in the ITS values of rubberized mixtures containing two WMA additives. Similar results can be obtained from the control mixtures. In addition, in comparison with two aggregate sources, the mixtures from aggregate source A show lower ITS values regardless of the WMA additive or rubber content. Moreover, statistical analysis shows that there are no significant differences in TSR values of the mixtures containing two WMA additives

- Rut depths of the rubberized mixtures have lower values than other mixtures. In general, the mixtures made with aggregate source A show greater rut depth. The mixtures containing two WMA additives show no significantly different rut depth values.

- The additions of rubber and WMA additive do not significantly affect the initial stiffness values of the mixtures. There is also no trend for these stiffness values based on the test results obtained in this study. The standard deviations of the fatigue test results for each mixture are high since the variability of fatigue life is generally based upon the micro-structure of beams. Fatigue life of mixture shows no obvious trend when the rubber or WMA additive were added.

5. Acknowledgement

The authors wish to thank the financial support of South Carolina Department of Health and Environmental Control (SC DHEC) to conduct this research work.

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Stability Assesment Through Solubility and Rheological Measurements of Gtr-modified Bitumen

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ABSTRACT. GTR(ground-tyre rubber)-modified binders or CRMB (crumb rubber-modified binders), by the so-called wet process, represent a particular case of PMB (polymer-modified binder), due to the specific features of this recycled-to-be rubber: a mixture of natural and synthetic vulcanized rubbers, carbon black and plasticizers among others. When the GTR is incorporated to the bitumen, the strong structure created through vulcanization and the strong bond between carbon black and rubber (carbon gel or bound rubber) hinder the swelling of the rubber and the extensive dispersion within the bitumen. Crumb rubber particles tend to settle towards the bottom of the storage tanks.

The use in many formulations of co-additives may hide or cover up to some extent the results of PMB-criterion stability, showing as stable binders with high concentration of decanted fine solid particles. Therefore, a more severe assessment is proposed, based on solubilisation, in order to quantify the concentration of "non-assimilable" rubber particles, allowing thus for a sedimentation extent and profile kinetics evaluation, as well as an assessment of the extent of digestion of the rubber. This method was applied to several binders manufactured with different concentration of GTR, different additives and different processes, to support the solubility approach. When these portions of the stored samples were analyzed through rheological measurements, a good agreement was in most cases found with the solubility tests. The prediction of stability was also evaluated at the industrial production site as well as the asphalt plants storage tanks.

KEYWORDS: GTR, wet process, stability, solubility, rheology.

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1. INTRODUCTION

The modification of bitumen with a range of polymers yields polymer-modified binders (PMB) showing enhanced service properties. Depending on both polymer and bitumen, as well as on the mixing process, the polymer is partially swollen by the light fractions of the bitumen by the so-called physical distillation process (Lesueur, 1998). In most cases, both systems, polymer and bitumen, are incompatible, and a compatibilization method must be developed in order to avoid a possible destabilization and gross-phase separation phenomenon. For many polymers (plastomers and elastomers), the mechanism by which the polymer-rich phase ends up separated from the bitumen-rich medium, begins with a coalescence stage (Hesp *et al*., 2002; Pérez-Lepe *et al*., 2006).

Ground-tyre-rubber (GTR) modified binders obtained by the so-called *wet process* represent, due to the specific features of ground-tyre rubber, a different type of binder than binders modified with elastomers or plastomers. GTR is generally composed by a mixture of natural and synthetic vulcanized rubbers, carbon black and plasticizers, among others. When the GTR is incorporated to the bitumen, the strong structure in the tyre created through vulcanization and the strong bond between carbon black and rubber (carbon gel or bound rubber) hinder the swelling of the rubber and the extensive dispersion within the bitumen (Billiter *et al*., 1997; Panagiotis *et al*., 2004). Due to the greater density of the non-digested GTR particles at common storage temperatures, a clear tendency to particle settling is observed in most cases.

The wet process involves a mixing stage of rubber and bitumen to form a modified binder, previous to the mixing with the aggregates. The dry process implies that the rubber is mixed in-situ with the aggregates, with or without a short pre-mixing stage. The dry-process solution may eventually cause some cohesion shortcomings (Páez *et al*., 2004) as well as quality control and heterogeneity problems. Many efforts have been carried out in order to obtain a GTR-modified binder such that the maximum beneficial properties of the rubber at the ground tyre particles, is transferred to the final binder and consequently, the recycling of tyres is of greater value. Additionally, the manufacture of modified bitumen at the binder production site is a logistics/cost-beneficial issue, provided the binder proves to be hot-storage stable during the transportation and storage previous-to-mix stages. Although a continuous stirring or recirculation in storage tanks reduces to a great extent the subsequent problems of settling, these best-practice facilities are not always readily available. The present work is focused on the production of stable GTR-binders with enhanced properties by the wet process.

Based on the experience of GTR on the Spanish roads, the use of crumb rubber is regulated by the standard OC 21-2007 for road construction (published by the Roads and Transportation Spanish Government Section). Binders produced by the *wet process* are permitted in the top traffic level (T00), while the dry process is restricted to a medium-level traffic demand (T2 level, in a range from T4 to T0, plus T00 as top extra level). Two groups of GTR binders are standardized for binder-production-site manufacture: those with low content of crumb rubber or low level of modification and those with high viscosity and high rubber content.

Various methods of stabilization, as the use of additives (synthetic polymers, reactive polymers, waxes, acids, aromatic oils,...) and/or high-digestion processes, as well as activation of the surface through additives or through grinding, have been carried out by different researchers in the field in order to obtain stable binders (Soto *et al*., 2005; Billiter *et al*., 1997; Peña *et al*., 2005; Bahia *et al*., 1994). The stability of the binder is an issue of importance, even when the plant storage tank is sufficiently agitated and the material is periodically recirculated (not always available).

The commonly accepted or practiced high-temperature storage stability criterion consists of assessing the difference of penetration and softening point between the upper and bottom binder portions of a binder tank, stored at high temperature, either a toothpaste tube in a laboratory oven or a large scale tank . This is the same criterion used for all PMB's (72h of still storage at 180℃, as pointed out by EN13399 standard). In the present work, a need for other types of tests for stability assessment is remarked.

The needle penetration test is not a reliable technique for asphalt-rubber. The cone-penetration test, commonly used for waxes or particulate-filled systems, is envisaged as an alternative. Even so, the use of additives in formulations may "hide" to some extent the results of PMB-criterion stability, showing as stable binders with high concentration of decanted fine solid particles. Therefore, a more severe assessment is proposed. The dissolution of the upper and lower portions, as well as the original blend, with a specific solvent (THF or toluene) renders the Concentration of "non-assimilable" rubber particles. By a simple solubility test, an assessment of the settling extent and a kinetics profile evaluation may be inferred, as well as an assessment of the extent of digestion of the rubber (Billiter *et al*., 1997; Youtcheff *et al*., 2004). The digestion of rubber, and more specifically the quantity of rubber (synthetic and natural) effectively transferred to the bitumen, can be further extracted from thermo-gravimetric analysis.

The thermal degradation of GTR exhibits four main stages as registered from weight loss under increasing temperature ramps. At the beginning, the plasticizers are lost within a range of 100℃ to 300℃, approximately. From 270℃ to 440℃, the natural rubber in the particles is decomposed. Synthetic binders appear on the thermogram between 410℃ and 520℃. A weight loss peak related to the degradation with oxygen of carbon black is found at higher temperatures (600℃-700℃).

By evaluating the rubber particles before and after the mixing with bitumen and right before the application on the field, the degree of incorporation or digestion of rubber could be attained.

Natural rubber vulcanised with common crosslinking promoters exhibits a transition due to the break and looseness of bonds from just 170℃ upwards (as registered by DSC). This phenomenon would eventually lead to a more viscous and sticky material that could promote particle size growth and/or irreversibly adhere to hot surfaces. This issue could induce a heat transfer efficiency loss within the storage tanks, with the consequent storage problems, even in the case of agitated containers. When the majority of the loose natural rubber (not as carbon gel) has already been transferred to the bitumen through a high-digestion process or anti-stick additives are used, the problem is minimized.

2. RESULTS

Ground tyre rubber analysis

Crumb rubber composition was studied by a thermo-gravimetric analysis in order to decompose at a glance the GTR particles into the different components at the reception of the tyre powder from different sources, type of grinding and size distribution.

In all cases, the same pattern was observed, with 4 main stages of material decomposition and loss under increasing temperature: plasticizers, followed by natural rubber, synthetic rubbers and carbon black (CB).

Depending on the origin of tyres (mainly, the relation of heavy-load-vehicle tyres to automobile tyres), the grinding process (cryogenic or ambient temperature) and the cleaning process (removal of steel and textile fibres), two important features may be extracted from the thermograms: the total amount of rubber and the final residue over the CB. As a reference, minimum 60% weight loss before carbon black (rubber + plasticizers) and maximum 10% of final residue is an acceptable raw material, for the storage stable binder + high digestion approach. Volatiles content are expected to be very low at common HMA temperatures. For example, at a maintained temperature of 185℃, a ground tyre rubber mass would lose a 0,7% weight of volatiles (from a TGA isothermal analysis). An identification of compounds is being carried out by TGA-MS.

In figure 1, a typical thermogram of GTR particles is presented. With the derivative of weight (%/℃), the extension and form of the loss peaks can be evaluated.

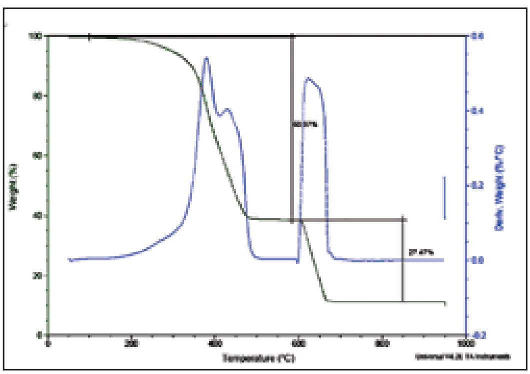

Figure 1. *Thermogram of a typical crumb tyre rubber. Weight percentage and derivative of weight percentage with temperature are represented.*

Shape and surface area of the particles are important for the digestion of the crumb rubber. Depending basically on the grinding process, the surface area obtained can be completely different. Cryogenic grinding is expected to yield more spherical particles with less surface area than the ambient temperature grinding. A tearing of rubber while grinding is an option yielding particles with high accessible area. Particle analysis with digital image processing was found a useful technique to measure size distribution and shape of the particles, as well as an estimate for the surface area. In figure 2 and 3, the results of the image analysis for two different kinds of particles (ground at ambient temperature with two different processes) are represented. In the first case, figure 2, a bimodal distribution is obtained as representing the amount of particles (Q3 in y-axis) sorted according to size (x-axis). The surface area in the first case is 180 /g, while the particles represented in figure 3 have an average of 430 cm^2/g.

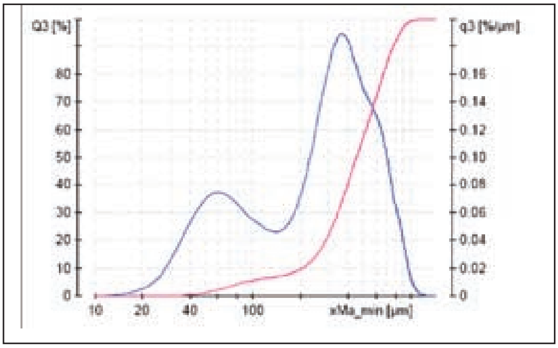

Figure 2. *Image analysis size distribution of crumb rubber with low surface area. Q3(number of particles(%)) vs size of particles (xMa_min) and q3 (number of particles (%)/size).*

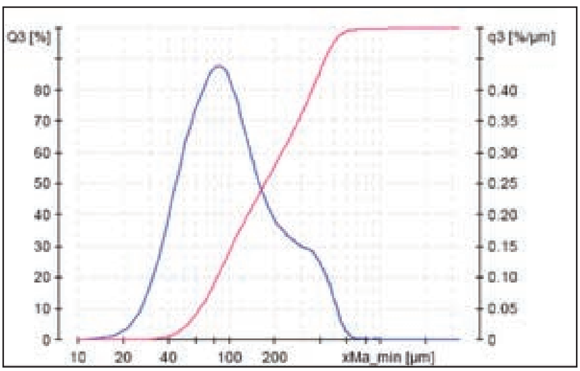

Figure 3. *Image analysis size distribution of crumb rubber with high surface area. Q3(number of particles(%)) vs size of particles (xMa_min) and q3 (number of particles (%)/size).*

Instability and settling of GTR particles

Due to the higher density of a single crumb particle, a settling is expected to occur when GTR is mixed (even extensively) with bitumen, during a high-temperature still storage. Crumb rubber is therefore gradually accumulated on the bottom part of tanks with no or non-effective agitation. A higher viscosity and tackiness is formed on the bottom part of the containers, causing tank problems of pumping, sticking to the heating coils and heterogeneity. This settling phenomenon can be reproduced with toothpaste tubes stored in an oven. The analysis by needle penetration is not able to reveal the cumulative quantity of digested particles on the bottom part of the stored binder. In order to evaluate the extent and profile of sedimentation, a basic dissolution method was applied. After the dissolution in THF (although toluene is also a preferred solvent) of a selected portion from the stability tube or height of the tank, the remaining digested material is vacuum recovered through a 2,7 μm fibre glass filter. A quantification of undigested rubber is done and the percentage weight of rubber on the bottom part and the upper part represents a valuable stability criterion, under the more severe storage conditions in cases where no agitation is available or required. Depending on the amount and features of the rubber added, viscosity and chemical composition of bitumen and the processing conditions, the settling would be ruled by a different profile. A further analysis by TGA of the recovered rubber particles can reveal the quantity of rubber dissolved or transferred to the bitumen phase. Following this set of experiments, an average of 23% of the total rubber content is found to be lost through the mixing and temperature digestion process. The rest of the rubber remains strongly bound to carbon black. The particles after the digestion show a higher carbon black concentration and higher residue after 1000℃ (Figure 4).

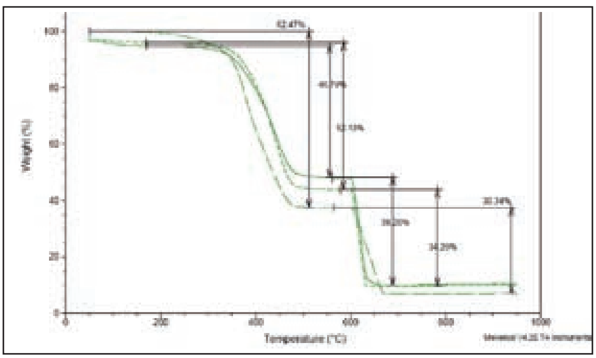

Figure 4. *Thermogram of crumb tyre rubber(— · —)and the recovered particles after a stability test form the upper section(—) and bottom section ().*

In a paper by the authors of the present work (2007), several binders produced without or with the addition of anchoring, bitumen structure modification or surface treatment compounds, revealed that the stability of a GTR-binder could not be assessed by the penetration and softening points of the binder. In cases where the differences of these two parameters were under 5-10 units, a high concentration of crumb particles was found on the bottom parts of the stability tubes. On the contrary, compositions showing values for penetration and softening point in the range 5 to 10 units, showed no accumulation of particles on the bottom.

From a rheological point of view, the behaviour of a GTR-modified binder material and the presence of rubber particles should be noticed through dynamic shear tests. In some cases, the system proposed for stabilisation or co-modification is a polymer (synthetic rubber in many cases). Some instability indices based on a comparison of modulus or dynamic viscosity at a specific frequency, after a hot storage test, have appeared on the literature. For this purpose, the storage modulus, G', is a preferred selective parameter, registered in the low frequency region and preferably at temperatures higher than 50℃, where the elastic rubber particles are best differentiated from the bitumen matrix. If the conditions of the rheological tests are not carefully selected, the presence of particles could not be reliably noted.

Even so, with the presence of polymers or waxes in the composition a basic rheological study may hide the settling of particles. In Figure 5, the complex modulus, G*, and the dynamic viscosity, η*, were registered within a range of frequencies of oscillation. The upper and bottom portions of a stored binder were tested at 60℃. The behaviour of both samples was very similar in the low frequency region, with only a small increase in G* for the bottom portion. The differences of penetration and R&B softening point were 9.6 1/10mm and 4.8℃. The bottom sample had a content of filtration recovered material of 15.3% (percent weight over the total amount), whereas the upper portion only 0.8% (the original binder had a 10% content of crumb rubber). Even with severe settling, the presence of particles was not quantitatively noticed.

When an effective stabilisation process is applied, the rheological curves before and after the stabilisation period show identical behaviour (figure 6). After dissolution and filtration of the samples, the content of undissolved matter is 7.4% , 7.7% and 7.6%, for the upper, medium and bottom parts of the stability tube.

Industrial production of stable binders

The manufacture of stable tyre rubber binders is carried out at a central site mixing facility. From the binder production site, the product is delivered in a tanker to the aggregate plant, where it is stored. From the manufacture point till the moment of mixing with the aggregates, the binder is preferably kept at a temperature close to the temperature of mixing and agitated. The product is not rarely subjected to gradients of temperature, periods with no agitation, or extended storage due to works delay. The product must then be completely storage stable in order to count on a robust binder for this cost-effective production-site application.

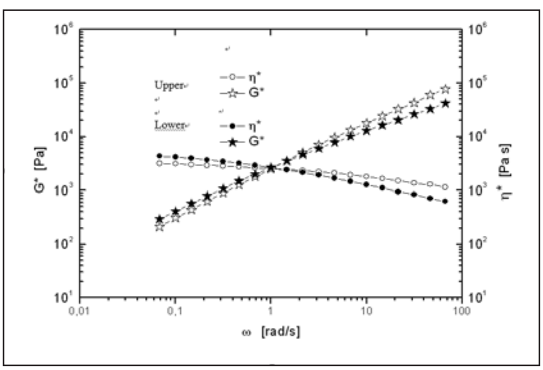

Figure 5. *Evolution of complex modulus and dynamic viscosity with angular velocity at 60℃ of the upper and lower portions after stability test (120h-163℃) of a 10% GTR binder with polymeric additive.*

The blend and reaction stage of rubber and bitumen can be carried out with a traditional batch mixing facility. An in-line mixing of the components is preferred for production time saving. As higher shear is achieved by adjusting mainly temperature and flow, a higher homogeneity and digestion extent may be obtained, in the following swelling stage.

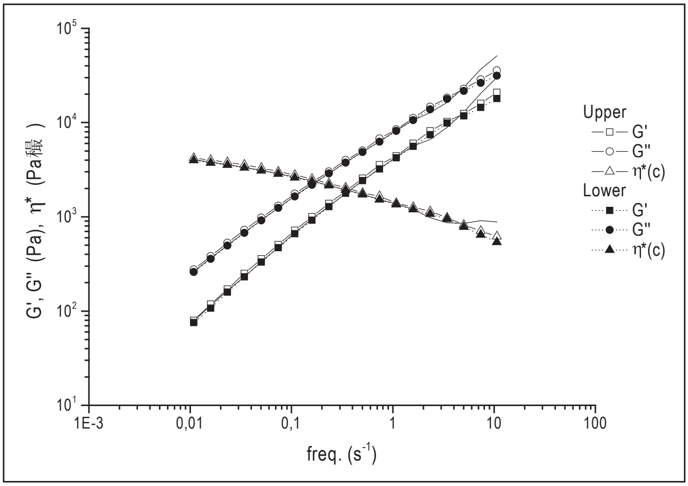

Figure 6. *Evolution of storage (●) and loss (▪) moduli and dynamic viscosity (▲) with frequency at 60℃ of the upper (empty) and lower(full) portions after stability test (120h-163 ℃) of a 10% GTR binder with polymeric additive.*

Evaluation of ground tyre rubber in certain hot-mix asphalt applications. Product range and applications

Repsol-YPF is an integrated international oil and gas company, operating in more than 30 countries and is the leader in Spain and Argentina. It is one of the ten major private oil companies in the world and the largest private energy company in Latin America in terms of assets. Repsol has a vast experience with the recycling of tyre rubber in roads through modified binders produced by the *wet process* at the binder production site. Since year 2004, binders for more than 400000Tm of HMA have been produced with an own developed technology to ensure binder homogeneity and stability.

Three levels of modification have been standardized, but tailor-made products are manufactured based on a precise application.

a) Low modification level

b) PMB equivalence

c) High-viscosity binders

a) Low modification level

These low modified rubber binders (production-site manufactured) present a response which falls between those of base bitumen and polymer-modified binders, with no special elastic recovery requirements. A specification for these binders was introduced in Spain for two penetration grades: 35-50 and 50-70 1/10mm. Typical binders properties produced at Repsol are showed in table 1.

Table 1. *Low modification binder standard properties.*

BINDER	**35-50 grade**	**50-70 grade**
Penetration 25 ℃, 1/10 mm	48	61
R&B SP, ℃	59.4	53.9
PI	+0,8	+0.2
Brookfield viscosity, cP		
At 135º C	1170	830
At 150º C	595	425
At 180º C	216	145
Storage Stability		
Penetration difference, 1/10mm	5	4
R&B difference, ℃	2.8	3.7
Difference THF-residue (lower-upper), %.	0.9	1.5
RTFOT		
Penetration retained (%)	67	66
Variation R&B., ℃	7.6	6.5
Variation mass (%)	-0.100	0.045

As an example, the behaviour of this binder (50/70) is studied for a dense-graded asphalt mix (20 mm nominal maximum size) and compared with base bitumen of the same penetration grade. Specimens were analyzed following the Marshall method as a reliable HMA design method allowing for optimum asphalt binder content determination. In table 2 and 3 Marshall results are presented for unmodified and modified binders, respectively. The aggregates size distribution is shown on figure 7.

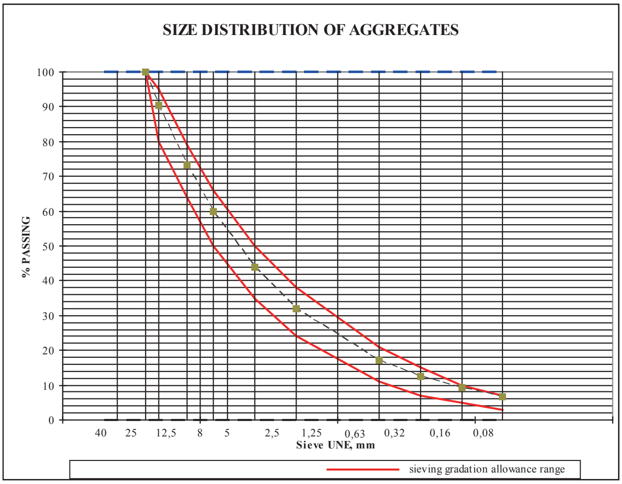

Figure 7. *Size distribution of aggregates according to a UNE sieve analysis.*

As can be seen from the Marshall results, a higher density is obtained as well as higher stability for the modified binder called BC50/70 (low-modified rubber binder). In the case of modified binder, the optimum content was selected on order to have a recommended 4,5% air voids. The optimum content with this criterion is higher for the modified binder than for the unmodified bitumen.

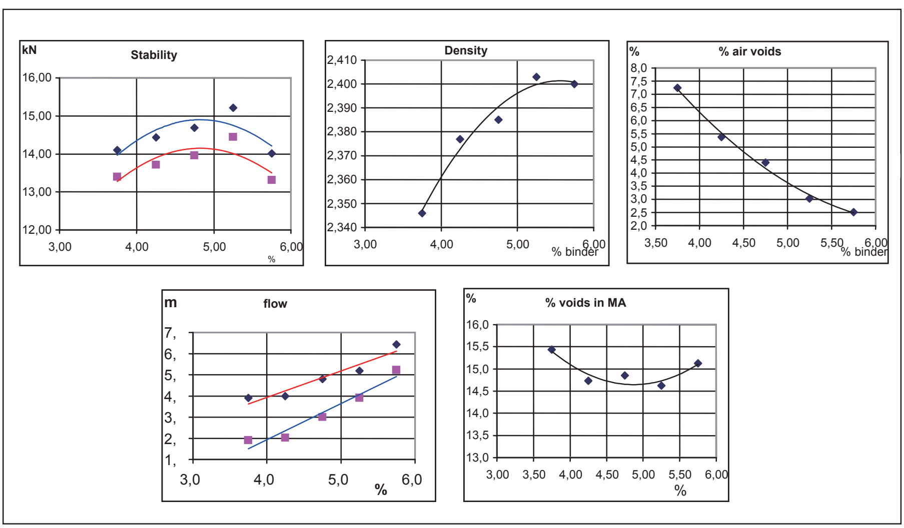

Figure 8. *Marshall method results on a 60/70 penetration grade bitumen: stability, density, % air voids, flow and % voids in mineral aggregates.*

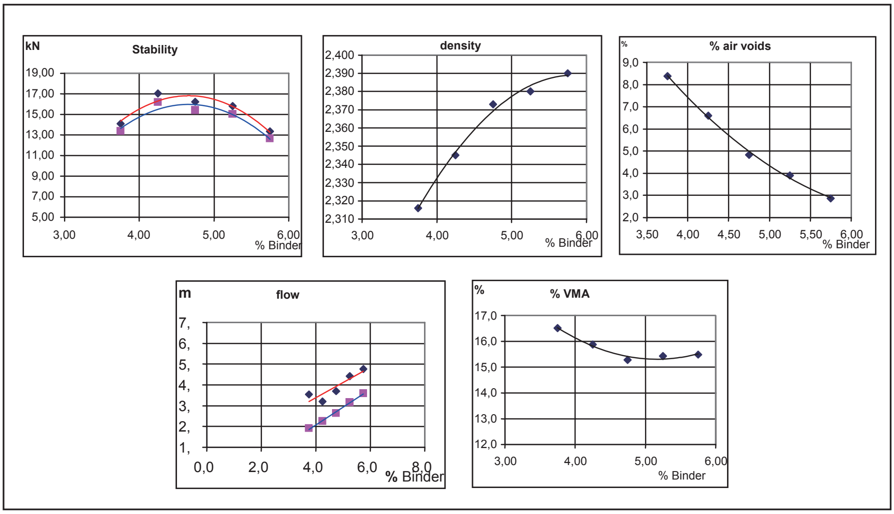

Figure 9. *Marshall method results on a low-modified rubber binder BC50/70: stability, density, % air voids, flow and % voids in mineral aggregates.*

Dynamic modulus experiments (NLT standard) and fatigue analysis (NLT standard) were carried out at a content of 4,5% b/m, for unmodified and CRMB (table 2). Dynamic modulus testing was carried out at 20℃ over tall cylindrical specimens compacted by vibrocompression and submitted to repetitive axial loading at 10 Hz.

Table 2. *Dynamic modulus and Fatigue results for bitumen and tyre-rubber binder BC50/70.*

	Bitumen 60/70	BC 50/70
Modulus @ 20℃ - 10 Hz (MPa)	6116	4781
Phase angle (°)	20.01	21.92
Fatigue Law $\log \varepsilon = a + b * \log N$	a: 3.3672 b: -0.1966 R^2: 0.79	a: 3.5869 b: -0.2412 R^2: 0.76
ε_6	149.83	137.94

Mixes prepared with CRMB and base bitumen show a similar response towards Fatigue. A higher resistance to permanent deformations is obtained by the rubber binder at 60℃. In table 3, the wheel-tracking test results on both samples are presented.

Table 3. *Wheel-tracking test results for bitumen and tyre-rubber binder BC50/70.*

BINDER	**60/70**	**BC 50/70**
Total Deformation	2.043 mm	1.103 mm
Rate of Deformation interval 30-45 min.	14.10-3mm/min	6.10-3mm/min
Rate of Deformation interval 75-90 min.	10. 10-3mm/min	3. 10-3mm/min
Rate of Deformation interval 105-120 min.	9.10-3mm/min	2. 10-3mm/min

Laying and compaction can be done with conventional machinery, but a special attention may be paid due to a slight more difficulty to compact the CRMB mixes due to a certain elastic response of the rubber, even at compaction temperatures . A mixing temperature of 185℃ and compaction at 175℃ was used for this particular work.

b) PMB

Traditional PMB's, usually made with elastomers SBR, SBS or plastomers EVA, can find a substitute in rubber-containing binders. A crumb rubber particle has about 50%-60% rubber in its composition. Not all the rubber can be efficiently swollen by the light fractions in the bitumen. In order to get similar physical properties than a PMB, more crumb rubber weight content is therefore needed. As a reference, around 10% rubber content is commonly used for this grade. AR binders have a good performance in gap-graded mixes as well as in open-graded mixes. The use of GTR binders in open-graded layers enables a considerable noise reduction due to tyre/road friction interaction. A greater binder film thickness is acquired due to the presence of CR. This is reported to help the noise reduction offered by a high number of voids.

As an example, a HMAR with 25% VMA was extended using a medium-modification level crumb rubber binder. The Spanish binder grade used corresponds to BM-2. Binder properties are shown on table 3, compared to those of the corresponding PMB (with synthetic elastomers). In figure 10, the sieving size distribution of the aggregates is presented.

The storage stability of the binder was evaluated by applying the dissolution approach. The content of non-digested crumb was practically identical in the upper and lower portions of the binder after hot temperature storage.

The problem of settling constitutes a heterogeneity problem and bulk viscosity evolution during storage. It can be treated as settling of concentrated suspensions of elastic solids in a viscoelastic matrix. A dynamic shear rheometer may be used to evaluate the evolution and profile of settling during hot temperature storage. A tall cup filled with binder was kept heated a several temperatures (163℃ and 180℃) and the binder was then studied with a vane rotor coupled to the rheometer head. Although dynamic experiments can be carried out in order to register the changes in elasticity due to rubber migration, continuous flow at low shear rate with the upper sensor on the bottom part of the container (to register clarification) or on the lower part (to register sedimentation), is also a good method to track the possible settling phenomenon. In figure 11, the viscosity evolution was tracked on a DSR at 163℃ during 48 hours of storage. No increase in viscosity was reported.

Table 4. *Properties of rubber-modified binder and polymer-modified binder.*

BINDER	BM-2(SBR)	BMC-2(GTR)
PENETRATION P25℃ (1/10 mm)	43	46
R&B (℃) NLT-125	72.4	69
PI	+2.85	+2.50
FRAASS T (℃)	-16	-17
DUCTILITY 5℃(cm)	12	10
STORAGE STABILITY R&B difference Penetration difference (25℃)	 1 3	 3 4
ELASTIC RECOVERY Torsion 25℃	70%	61%
VISCOSITY 135℃ (cP)	2724	3500
VISCOSITY 150℃ (cP)	1300	1900
VISCOSITY 175℃ (cP)	375	401
RHEOMETRY AASHTO TP5 Complex Modulus, G* (Pa) 60℃ Phase angle, δ, (º) 60℃	 8900 55	 9550 57

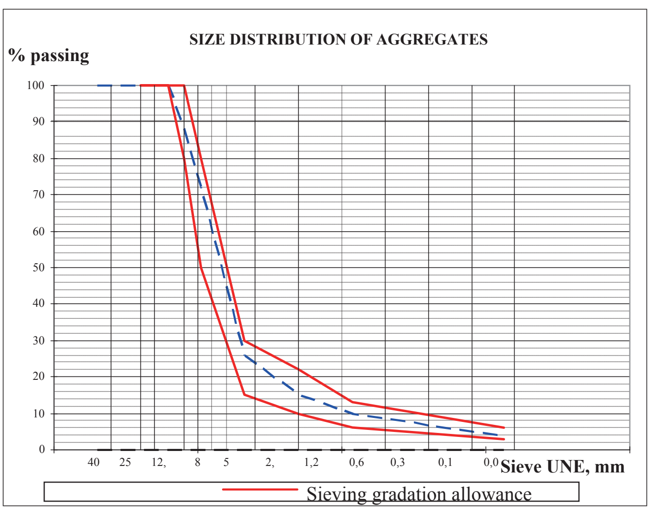

Figure 10. *Size distribution of aggregates according to a UNE sieve analysis.*

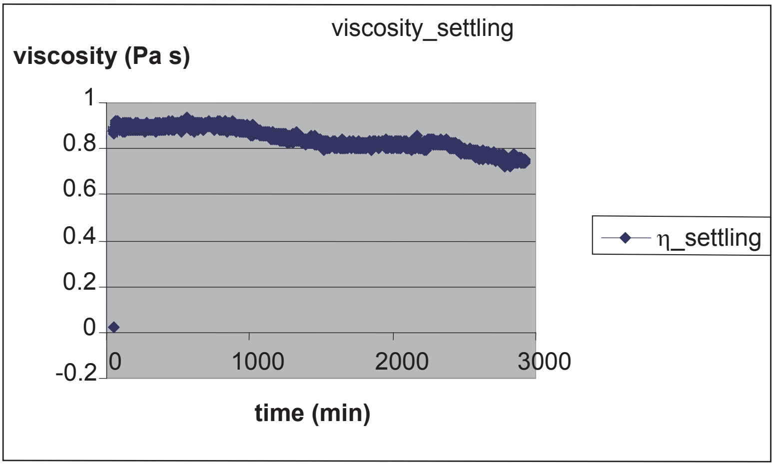

Figure 11. *Viscosity evolution during 48 hours of BMC-2 crumb rubber binder.*

At the production site, the product was previous to serve evaluated in the storage tanks to ensure stability. Samples from the upper part of the tank and through the bottom discharge sampler were tested to check the complete homogeneity of the product. No problem associated with the instability of the product was found during the aggregate mix preparation.

Due to a bad quality of the aggregates, a lack of adhesion was noted, even for a PMB of the same grade used on purpose. An adhesion promoter additive (surfactant type) was added during the formulation of the binder with satisfactory results. Mix design was established with the Cantabrian (resistance to disintegration) and drainage tests for open-graded mixes. An optimum of 4,75% b/m was selected.

Table 5. *Mix-design for an open-graded asphalt-mix with PMB and GTR-modified binder. Data for abrasion resistance tests.*

Mixing Temperature	160℃							
Compaction Temperature	150℃							
Blows per side	50							
Test Temperature	25℃							
binder BMC-2 (GTR)	% b/m		4.0	4.5	5.0			
Binder BM-2 (PMB)	% b/m	*4.5*						
binder BMC-2 + adhesión promoter						*4.0*	*4.5*	*5.0*
Density	g/cm^3	*1.936*	*1.950*	*1.963*	*1.988*	*1.939*	*1.946*	*1.958*
Air voids	%	*21.6*	*21.6*	*20.3*	*19.0*	*22.0*	*21.2*	*20.2*
Dry Abrasion Loss	%	*38*	*30*	*29*	*14*	*23*	*22*	*17*
Wet Abrasion Loss (24h / 60º C)	%	*68*	*37*	*33*	*25*	*36*	*32*	*18*

c) High-viscosity binders

For non-continuous mix gradation, high contents of binders with a high viscosity and high stiffness modulus are a preferred solution to obtain mixes with improved durability, rutting resistance, fatigue resistance and anti-cracking properties. High-content CR binders present higher viscosities than a PMB, enabling for higher binder contents in the mix.

As an example, a discontinuous-graded asphalt-mix was produced with a high viscosity binder. The binder was manufactured with a high content of crumb rubber, which was high-shear mixed with a low penetration bitumen. Data for aggregate size distribution and binder properties are provided below.

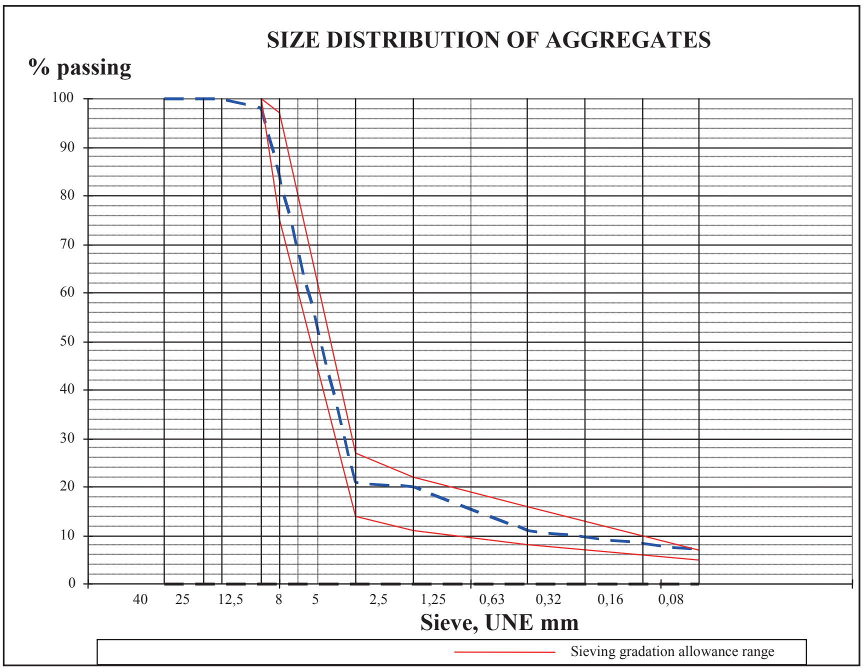

Figure 12. *Size distribution of aggregates according to a UNE sieve analysis.*

The mixing design was selected according to the Marshall method with a compaction of 50bps. 7% was selected as the optimum binder content (weight percent of binder by aggregate).

Table 6. *Properties of high-viscosity asphalt rubber binder.*

BINDER	**Standard**	**BMAVC-1**
PENETRATION P25℃ (1/10 mm)	15-30	28
R&B (℃) NLT-125	75	81
PI		+3.0
FRAASS T (℃)	-4	-16
STORAGE STABILITY R&B difference Penetration difference (25℃)	 5 20	 3 5
ELASTIC RECOVERY TORSION 25℃	10%	43%
VISCOSITY 135℃ (cP)		10500
VISCOSITY 150℃ (cP)		4800
VISCOSITY 175℃ (cP)	2000	2100
RHEOMETRY AASHTO TP5 Complex Modulus, G* (Pa) 60℃ Phase angle, δ, (º) 60℃		 23000 69

Table 7. *High-viscosity asphalt rubber mix Marshall results.*

Mixing Temperature	**185ºC**		
Compaction temperature	**175ºC**		
Binder content BMAVC-1 b/a	%	7.0	8.0
Density	g/cm^3	***2.264***	***2.261***
%VMA	%	***21.5***	***22.3***
% Air voids	%	***7.1***	***6.1***
Stability (E_p)	kN	***10.12***	***8.80***
Flow (D_p)	mm.	***2.7***	***1.9***
Stability/Flow	kN/mm	***3.8***	***4.5***

4. CONCLUDING REMARKS

- The production of ground tyre rubber binders through a *wet process* by using a high digestion process and co-additives is a preferred alternative of recycling tyres into the pavements. The benefits of manufacturing at the production site a stable product along the factory storage, transportation and plant storage before mixing represent a cost-effective alternative.

- By incorporating the crumb rubber with a high digestion process, part of the rubber can be effectively transferred to the continuous phase and can be partially swollen by the lights fractions of bitumen. The cohesion attained in the binder by the *wet process* is much higher than through dry process or low energy blending.

- The stability of the binder against settling and heterogeneity must be evaluated through a different test than that prescribed for PMB's. A dissolution and filtration of portions at different heights of a binder stored at high temperatures is proposed as a best practice for detection and quantification of possible settling phenomenon. Rheological measurements can in most cases be a reliable technique for this purpose, as far as the test conditions are thoughtfully selected. With the presence of certain polymeric additives this technique may lead to misinterpretation.

- Whether the crumb rubber is incorporated to the bitumen through a high digestion process or a semi-dry (right before the aggregate mixing) process, the material must be carefully treated at high temperatures, due to the tendency of natural rubber to stick together and to hot surfaces when a limit temperature is surpassed.

- A wide range of binders, from low to high content, can be manufactured from GTR, provided the stability of the binder is assured, for a particular application: from dense-graded to porous asphalt, as well as special applications as high-content binder anti-cracking layers.

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Comparison Between Various Bituminous Binders Modified With Crumb Tyre Rubber

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ABSTRACT. The aim of this paper is to show how the rheology, the performance and the storage stability of different bituminous binders, obtained from two bitumens with different asphaltenes content, change their properties when they are modified with recycled tyre crumb rubber. Physical, chemical and performance characterisation followed by rheological and storage stability analyses have been undertaken as the basis for the comparison. The results of the investigation indicate that bitumen with lower asphaltenes content is more influenced by the modification with the rubber. However, the modification of the bitumen with the higher asphaltenes content gives a binder with better performance and storage stability. Moreover, results shows that the addition of a certain amount of oil extender to the bitumens reduces the effect of the modification and has no positive effect on storage stability.

KEYWORDS: crumb rubber, modified bitumen, performance grade, DMA, PDA

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1. Introduction

Historically, recycled rubber from tyre waste has been used in asphalt mixtures using two approaches, namely the wet process and the dry process. In the dry process, the recycled rubber is mixed with the aggregates before introducing the binder to the mixture, the wet process, instead, involves blending the bitumen with rubber particles (known as crumb rubber modifier) at an elevated temperature (170 to 200°C) eventually in the presence of an oil extender. The obtained product is a rubberised bitumen, which is then mixed with aggregate to form a mixture. The interaction of bitumen with rubber in the wet process is known to be affected by the blending temperature, the duration of blending, the type and amount of mechanical blending energy, the size and texture of the rubber particles, and the aromatic component of the bitumen. The absorption of aromatic oils from the bitumen into the rubber's polymer chains causes the rubber to swell and soften. The type and amount of aromatic oil in the bitumen also plays a major role in determining the compatibility of bitumen-rubber blends (Chehovits *et al*, 1982; Oliver, 1982).

This paper compares the physical, rheological and performance properties of a series of tyre rubber-modified asphalt binders produced using a combination of different base bitumens, an oil extender and crumb rubber. The bitumen testing varies from standard empirical tests, such as the penetration and softening point tests, to more detailed rheological tests using a dynamic shear rheometer. In addition to the rheological properties, the storage stability of the tyre rubber-modified asphalt binders has been assessed.

2. Background

2.1 Bitumen composition and structure

Bitumen has a complex chemical composition, consisting mainly of hydrocarbons, with a high carbon content of between 80% and 90% along with small percentages of analogous hetrocyclic species and functional groups of nitrogen, sulphur and oxygen atoms. Traditionally, bitumen is regarded as a colloidal system consisting of a suspension of high molecular weight asphaltene micelles dispersed in a lower molecular weight oily medium identified as maltenes (Lu *et al*, 1999). By means of absorption chromatography, bitumen can be separated into four functional groups with related properties with regard to chemical reactivity and rheological properties despite its complexity. These generic components are generally referred to as: Asphaltenes, Resinous components (polar aromatics), Aromatics (non-polar naphtene aromatics), and Saturates (Read and Whiteoak, 2003). In the presence of sufficient quantities of resins and aromatics of adequate solvating power, the asphaltenes are fully peptized, at every temperature, and the resulting micelles have good mobility within bitumen. Peptization is provided by a shell of resins that surrounds the asphaltene core with the thickness of this resin layer being temperature dependent (Figure 1a). At low temperature and in the case of high asphaltene content the bitumen can show a compact structure as shown in Figures 1b and 1c. On the other hand, insufficient quantities of resins and aromatics (insufficient solvating power) may lead to an association of several micelles, which may be simply expressed using the following thermal dependent equilibrium:

molecules <=> micelles <=> clusters

The system is therefore formed by a liquid part (resins dissolved in a maltene matrix), in equilibrium with a solid part (resins peptizing the asphaltenes) and it is, of course, temperature dependent (Yen, 1990).

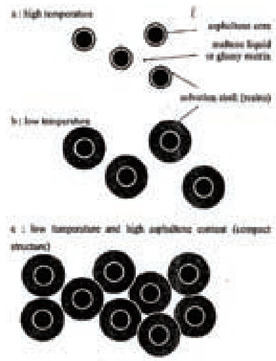


Figure 1.*Colloidal structures of asphalt*

2.2 Rheology and Rheometry

Rheology is literally "flow science". Rheological experiments do not merely reveal information about the flow behaviour of liquids, but also the deformation behaviour of solids. The connection here is that large deformations produced by shear forces cause many materials to flow. All forms of shear behaviour, which can be described rheologically in a scientific way, can be viewed as lying in between two extremes: the flow of idealviscous liquids on one hand and the deformation of idealelastic solids on the other (Mezger, 2002). Bitumen is a thermoplastic material that, under most pavement operating conditions, shows a behaviour which is between these extremes and it is based on the combination of both the viscous and the elastic portion and therefore is called viscoelastic

Rheometry is the measuring technology used to determine rheological properties of a material. It includes measuring systems, instruments, tests and analysis methods. At present the most common rheometry of bitumens includes Dynamic Shear Rheometer (DSR), which performs a means of dynamic/cyclic analysis applying pure shear stresses in continuous/oscillatory regimes, and Bending Beam Rheometer (BBR) which operates only at really low temperatures at a constant rate, inducing normal stresses in a bitumen beam. The most recent DSRs can even work at below zero degrees; this makes it probably the most suitable equipment for a complete rheological, viscoelastic characterisation of bituminous binders.

2.3 Definition of terms and analysis methods

The principal viscoelastic parameters that are obtained from rheometrical measurements, mainly with the DSR, are the complex shear modulus, G*, and the phase angle, δ. G* is defined as the ratio of maximum (shear) stress to maximum strain and provides a measure of the total resistance to deformation when the bitumen is subjected to shear loading. It contains elastic and viscous components which are designated as the storage modulus, G', and loss modulus, G'', respectively. These two components are related to the complex (shear) modulus and to each other through the phase (or loss) angle δ which is the phase, or time, lag between the applied shear stress and shear strain responses during a test. In order to perform a complete rheological characterisation it is therefore fundamental to firstly collect the rheological parameters with oscillatory tests with variable frequency and constant small strain level within the Linear Visco-Elastic response (LVE) over a range of temperatures and then elaborate the results through analytical techniques. One of the primary analytical techniques used in analysing dynamic/cyclic mechanical data involves the construction of master curves, which represents the viscoelastic behaviour of a bituminous binder at a given temperature for a large range of frequencies. The principle that is used to relate the equivalency between time and temperature and thereby produce the master curve is the Time Temperature Superposition Principle (TTSP). Once TTSP is applied another interesting analysis consists in correlating the viscoelastic parameters with temperatures at constant frequency. This kind of elaboration provides the Isochronal plots. Another fundamental curve obtainable from the frequency sweep data is the Black diagram. A Black diagram is a graph of the magnitude (norm) of the complex modulus, G*, versus the phase angle, δ, obtained from a dynamic/cyclic test. The frequency and the temperature are therefore eliminated from the plot, which allows all the dynamic data to be presented in one plot without the need to perform TTSP manipulations of the raw data. For this reason the Black diagram is well known even as the finger print of the binder (Airey 2002).

3. Experimental Programme

3.1 Materials

Two different base bitumens, a PG 64-16 (bitumen A) and a PG 64-22 (bitumen B), were used with and without an oil extender (flux) to produce four different tyre rubber-modified asphalt binders (TR-MAB). The TR-MABs were blended using a Silverson high shear laboratory mill to mix the base binders with fine crumb rubber obtained from used tyres. Physical, chemical and performance properties of the base bitumens (with and without flux) are reported in Section 4. Table 1 reports the characteristics of the crumb rubber as received by the provider. Table 2 and Figure 2 show the results of the sieving procedure and the portion of rubber used in this study denominated as "fine".

Table 1. *Properties of the crumb rubber*

Origin :	**car tyre rubber reduced to crumbs by mechanical treatment**
Particle shape :	irregular
Fibre content :	0.5 %
Steel content :	0.1 %

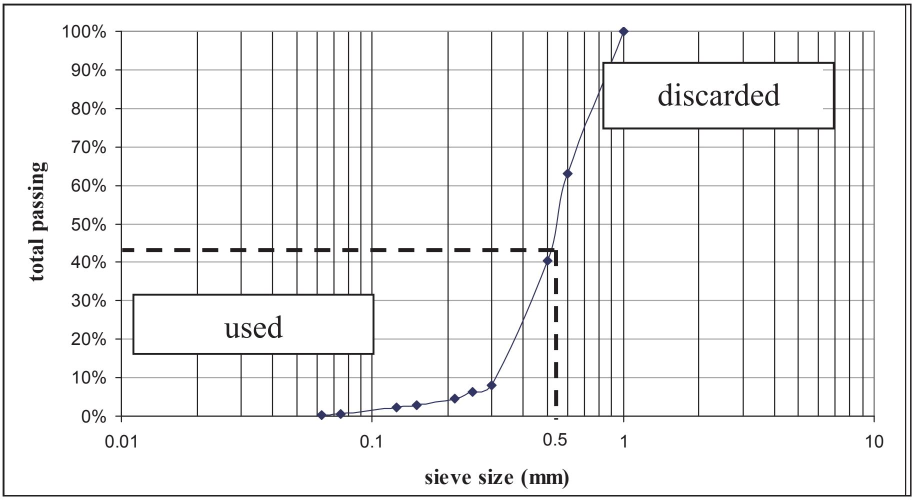


Figure 2. *Gradation curve of the crumb rubber*

Table 2. *Sieving of the crumb rubber*

sieve size	Mass Retained		retained on sieve	Total passing	
	cumulative	individual			
mm	g	g	%	%	
1.0	0	0	0	100	discarded
0.6	44.1	44.1	37.1	62.9	
0.5	70.9	26.8	22.5	40.4	used
0.3	109.5	38.6	32.4	8.0	
0.25	111.6	2.1	1.8	6.2	
0.212	113.5	1.9	1.6	4.6	
0.15	115.4	1.9	1.6	3.0	
0.125	116.1	0.7	0.6	2.4	
0.075	118.3	2.2	1.8	0.6	
0.063	118.8	0.5	0.4	0.2	
	119	0.2	0.2		
Total mass	119	119			

The following binders have therefore been incorporated into this study:

- **bitumen A:** PG 64-16 straight run bitumen
- **bitumen B:** PG 64-22 straight run bitumen
- **binder Af:** blend of 92.5% of PG 64-16 straight run bitumen and 7.5% of flux
- **binder Bf:** blend of 92.5% of PG 64-22 straight run bitumen and 7.5% of flux
- **TR-MAB A:** tyre rubber-modified asphalt binder produced as 85% of bitumen A and 15% of crumb rubber
- **TR-MAB B:** tyre rubber-modified asphalt binder produced as 85% of bitumen B and 15% of crumb rubber
- **TR-MAB Af:** tyre rubber-modified asphalt binder produced as 85% of base binder Af and 15% of crumb rubber
- **TR-MAB Bf:** tyre rubber-modified asphalt binder produced as 85% of base binder Bf and 15% of crumb rubber

All the binders were tested in their unaged (virgin) condition and after being subjected to two laboratories ageing procedures:

- Short-term laboratory ageing using the Rolling Thin Film Oven test (RTFOT) according to AASHTO T 240.

- Long-term laboratory ageing using a Pressurised Ageing Vessel (PAV) according to AASHTO R 28.

Due to practical concerns (more details in Section 4), the short term ageing of the modified bitumens was performed by using the Thin Film Oven test(TFOT)according to AASHTO T 240.

3.2 TR-MAB blending protocol

The blending of rubber and bituminous binders was carried out using the following protocol:

1. The required amount of bitumen was heated at 180°C in the oven and then transferred to a hot plate at the same temperature.
2. High shear mixing up to 2000 rpm was applied for the first 10 minutes while the flux (only for binders Af and Bf and TR-MAB Af and Bf) and then only the fine portion of the rubber was feed into the bitumen.
3. Time was allowed for the temperature to stabilise at 180°C.
4. Once the temperature reached 180°C, blending time was noted and mixing undertaken at 1000 rpm for one hour.

Tables 3 and 4 summarise the mixing parameters used in the blending procedure.

Table 3. *Blending protocol for TR-MAB A and TR-MAB B*

mass of bitumen (85%)	mass of flux (0%)	mass of rubber (15%)	rubber size	total weight	mixing time	mixing speed	mixing temp.
g	g	g	mm	g	min	rpm	°C
1700	0	300	0-0.5	2000	60	1000	180

Table 4. *Blending protocol for TR-MAB Af and TR-MAB Bf*

mass of bitumen (78.65%)	mass of flux (6.35%)	mass of rubber (15%)	rubber size	total weight	mixing time	mixing speed	mixing temp.
g	g	g	mm	g	min	rpm	°C
1572.5	127.5	300	0-0.5	2000	60	1000	180

3.3 Physical and chemical characterization

The first part of the programme consisted of characterising the bitumens and base binders in terms of their physical and chemical properties and to quantify the effect of the oil extender. The physical classification consisted of the following tests: penetration (ASTM D5), softening point (ASTM D36-95), Fraass breaking point (IP 80), ductility (10ºC, ASTM D113) and rotational viscosity at 100 and 160ºC (ASTM T316-04). In order to have an idea of the chemical composition of the binders, the asphaltenes content according to ASTM D6560 was measured.

The second part of the programme consisted of comparing the different binders in terms of their rheological characteristics using a dynamic shear rheometer (DSR). Finally, in the third part of the programme, the storage stability of the binders was evaluated using softening point (ASTM D36-95) and polymer dispersion of the elastomer in the binders.

3.4 Performance grading procedure

In order to classify the binders by their performance properties, all of them were subjected to the Superpave grading procedure according to AASHTO R 29-02. The performance grading is undertaken by testing the binder at three different ageing conditions: unaged, short-term aged (RTFO aged) and long-term aged (PAV aged). As shown in Tables 4 and 5, the high performance grade (high PG) is determined based on tests performed on the unaged and RTFO aged binder.

Table 4. *Overview of the procedure to find the high PG*

High PG Value
Unaged binder: - the binder is tested to determine the flash point in °C (AASHTO T 48), - viscosity at 135°C (AASHTO T316), - the shear modulus (G*) and phase angle (AASHTO T 315)
RTFO aged binder: - the binder is aged in the Rolling Thin-Film Oven (AASHTO T 240) - and the residue is tested to determine the mass loss (AASHTO T 240), - the shear modulus (G*) and phase angle (AASHTO T 315)

Completing the sequence, the low performance grade (low PG) is determined by tests on the PAV aged binder. The low PG of all the binders has been established using Table 1 of AASHTO M 320-05.

Table 5. *Overview of the procedure to find the low PG*

Low PG Value
PAV aged binder: - The residue from the rolling thin-film oven is aged in the Pressurised Aging Vessel (AASHTO R 28) and this residue is tested to determine: - the shear modulus (G*) and phase angle (AASHTO T 315), - the creep stiffness (S) and slope, m of the log creep stiffness versus log time relationship at different instants of load (AASHTO T 313)*, - eventually the failure stress in Direct Tension (AASHTO T 314), - and in post-processing the Critical Cracking Temperature (AASHTO PP 42)**
* last step for assigning the low grade using Table 1 of the specification ** last step for assigning the low grade using Table 2 of the specification

3.5 Rheological analysis

In order to obtain a rheological characterisation of all the binders, a dynamic mechanical analysis (DMA) over a wide range of temperatures and frequency sweeps was performed using a Bohlin GEMINI DSR. The tests were performed under the following conditions:

- **Mode of loading:**	controlled-strain
- **Temperatures:**	0 to 80ºC at 5ºC intervals
- **Frequencies:**	0.10, 0.16, 0.25, 0.40, 0.63, 1, 1.6, 2.5, 4, 6.3 and 10 Hz
- **Spindle geometries:**	8 mm φ and 2 mm gap (0-45ºC), 25 mm φ and 1 mm gap (35-80ºC)
- **Strain amplitude:**	0.5% (within LVE response dependent on G*)

For each test, samples were prepared by means of a hot pour method, based on Alternative 1 of the AASHTO TP5 Standard. The gap between the upper and lower plates of the DSR was set to a height of 50 μm plus the required testing gap at the mid-point of the testing temperature range. Once the gap has been set, a sufficient amount of hot bitumen (160ºC) was poured on to the lower plate of the DSR to ensure a slight excess of material appropriate to the chosen testing geometry. The upper plate of the DSR was then gradually lowered to the required nominal testing gap. The bitumen that was squeezed out between the plates was then trimmed flush to the edge using a hot blade. Finally, the gap was closed until there was a slight bulge around the circumference of the testing geometry.

The rheological properties of the binders were measured in terms of their complex (shear) modulus, G*; and phase angle (viscoelastic balance of rheological behavior), δ. Once measured, the data was used together with the Time Temperature Superposition Principle and shift factors to produce master curves at 25°C and isochronal plots. Master curves, isochronal plots and black diagrams have been used as the basis of all the rheological analysis in this paper.

4. Results and discussions

4.1 Base bitumens

Two base bitumens were available and both have been characterized in terms of their physical, chemical and performance properties. The two bitumens have similar physical properties, but differ chemically in terms of their asphaltenes content (Table 6). This difference in chemical composition potentially has an effect on the modification process when they are mixed with polymers. In general a smaller percentage of asphaltenes means a lower Colloidal Index value that leads to a system with a higher degree of solvency for the polymers (Airey, 2003).

Table 6. *Properties of base bitumen A and base bitumen B*

	bitumen A	bitumen B
Penetration (ASTM D5)	42 dmm	54 dmm
Softening Point (ASTM D36-95)	51ºC	52.2 ºC
Fraass breaking point (IP 80)	0 ºC	-2 ºC
Ductility (ASTM D113)	1000 mm	1000 mm
Rotational Viscosity @ 100ºC (ASTM T316-04)	3.86 Pa.s	5.13 Pa.s
Rotational Viscosity @ 160ºC (ASTM T316-04)	0.12 Pa.s	0.19 Pa.s
Asphaltenes content	3.4%	16.7%

The rheological properties of the two base bitumens in terms of master curves, black diagrams and isochronal plots are shown in Figure 3.

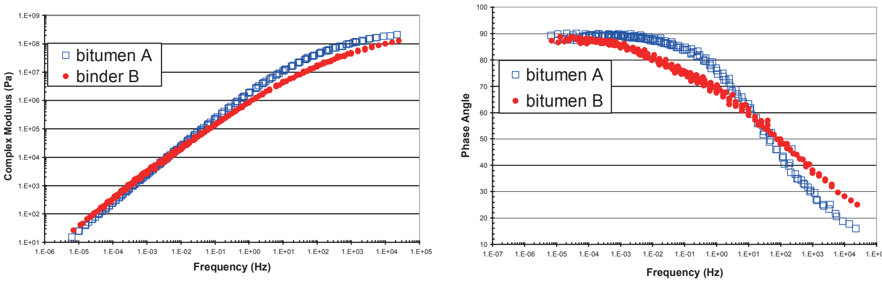


Figure 3a. *Master curves of complex modulus and phase angle at 25 ºC for the base bitumens*

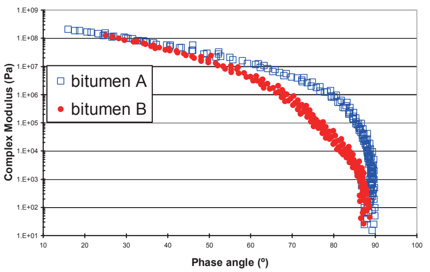


Figure 3b. *Black diagram for the base bitumens*

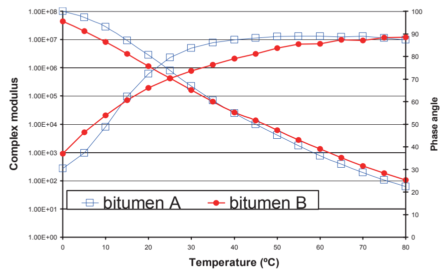


Figure 3c. *Isochronal plots of complex modulus and phase angle at 0.4 Hz for the base bitumens*

Although the empirical rheological properties of penetration and softening point for the two base bitumens are similar, their detailed rheological properties shown in Figures 3a-c indicate differences in their more detailed rheological properties. Bitumen “A” has a higher complex modulus at high frequencies although bitumen “B” tends to be less temperature and frequency susceptible. Both bitumens are thermorheologically simple as shown by their smooth curves in both the master curves and Black diagram (Mezger, 2002). The higher asphaltenes content of bitumen “B” results in a shifting of the viscoelastic balance with a reduction in phase angles (decreasing viscous response/increasing elastic response) compared to the low asphaltenes content bitumen “A”.

4.2 Base binders

As reported by other researchers, it is common practice to add an oil extender into the blend to improve the solvating power of the base bitumen and increase the digestion of the crumb rubber within the bitumen (Potgieter & Coetsee, 2003). A blend of 92.5% base bitumen and 7.5% of oil extender (flux) was made for each bitumen obtaining “base binder Af” and “base binder Bf”.

Table 7. *Physical properties of base binder Af and base binder Bf*

	binder Af	**binder Bf**
Penetration (ASTM D5)	136 dmm	157 dmm
Softening Point (ASTM D36-95)	39.8ºC	40.3ºC
Fraass breaking point (IP 80)	-14ºC	-17ºC
Ductility (ASTM D113)	1000 mm	1000 mm
Rotational Viscosity @ 100ºC (ASTM T316-04)	1.57 Pa.s	2.14 Pa.s
Rotational Viscosity @ 160ºC (ASTM T316-04)	0.07 Pa.s	0.13 Pa.s
Asphaltenes content	3.6%	16.7%

As it is noticeable from Tables 6 and 7 that the addition of the flux does not affect the asphaltenes content. Analysing the results, it is evident that the oil extender affects the base binder by softening it and thereby enhancing its low temperatures properties. Tables 6 and 7 show how physical properties are influenced by the flux with an increase in the cracking resistance of both binders at low temperatures, as shown by the lower Fraass breaking point temperature, and a softening of the binders at high temperatures, as shown by a decrease in softening point and reduced viscosity. The modification in the performance properties have been investigated and the results in Tables 8 and 9 show that in both cases the base bitumen changes its performance grade moving towards lower values but maintains the same performance temperature range. It should be noted that this shifting towards lower performance grade values, due to the oil extender, has a greater effect on the bitumen with the lower asphaltenes content (bitumen A).

The softening of the base bitumens throught the addition of the flux is also seen in the detailed rheological analysis for both base bitumens and binders in Figures 4a-c and 5a-c. The results show a uniform shifting of the complex modulus towards lower values and an increase in the viscous behaviour at high frequencies and low temperatures (higher values of phase angle). The Black diagrams (Figures 4c and 5c) show that even if the base binder contains flux, it still has the same black curve as that of the base bitumen, with both binders being considered to be thermo-rheologically simple (Mezger, 2002). Considering that elastomers mainly modify the bitumen by increasing its performance at low frequencies and high temperatures (Airey, 2002), the results demonstrate how the addition of the flux is important to improve the performance of the base binder at high frequencies and low temperatures.

Table 8. *Performance grading of bitumen A and binder Af*

Aging states	AASHTO specifications	bitumen A	binder Af
Unaged binder	Rotational Viscosity (Pa.s)	@135°C/ 0.40	@135°C/ 0.20
	G*/sin(delta) (kPa) > 1.00	@64°C/ 1.41	@52°C / 1.28
RTFO aged residue	G*/sin(delta) (kPa) > 2.20	@64°C/ 2.90	@52°C / 3.51
RTFO+PAV aged residue	G* x sin(delta) (kPa) < 5000	@28°C/ 4790	@16°C / 4970
	Stiffness@ (MPa) < 300	@-6°C / 218	@-18°C/ 275
	m-value has to be > 0.3	@-6°C/ 0.36	@-18°C/ 0.33
Flash point (°C) has to be > 230		> 300	> 300
Mass loss after RTFO ageing has to be< 1.0%		0.15%	0.09%
Performance grade		PG 64 - 16	PG 52 - 28
Performance temperature range		80°C	80°C

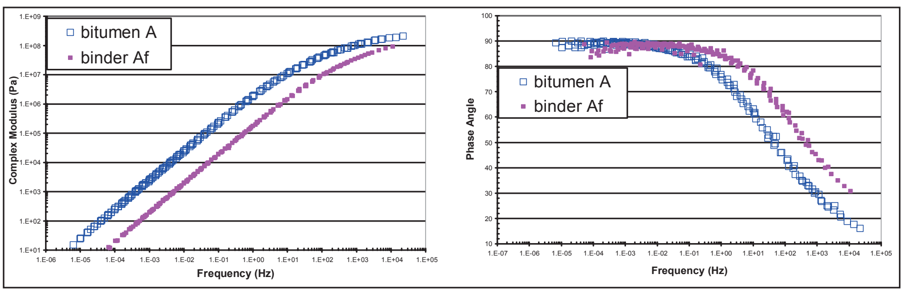


Figure 4a. *Master curves of complex modulus and phase angle at 25 ºC for bitumen A and binder Af*

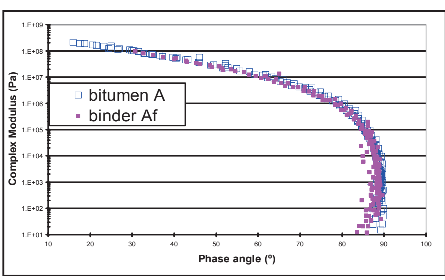


Figure 4b. *Black diagram for bitumen A and binder Af*

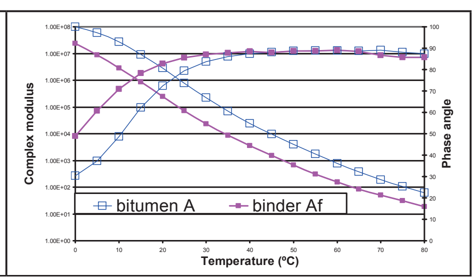


Figure 4c. *Isochronal plots of complex modulus and phase angle at 0.4 Hz for bitumen A and binder Af*

Table 9. *Performance grading of "bitumen B" and "binder Bf"*

Aging states	AASHTO specifications	bitumen B	binder Bf
Unaged binder	Rotational Viscosity (Pa.s)	@135°C/ 0.45	@135°C/ 0.23
	G*/sin(delta) (kPa) > 1.00	@70°C/ 1.05	@58°C / 1.47
RTFO aged residue	G*/sin(delta) (kPa) > 2.20	@64°C/ 4.56	@58°C / 3.05
RTFO+PAV aged residue	G* x sin(delta) (kPa) < 5000	@22°C/ 5317	@16°C / 4180
	Stiffness@ (MPa) < 300	@-12°C / 225	@-18°C/ 130
	m-value has to be > 0.3	@-12°C/ 0.34	@-18°C/ 0.38
Flash point (°C) has to be > 230		> 300	> 300
Mass loss after RTFO ageing has to be< 1.0%		0.07%	0.01%
Performance grade		PG 64 - 22	PG 58 - 28
Performance temperature range		86°C	86°C

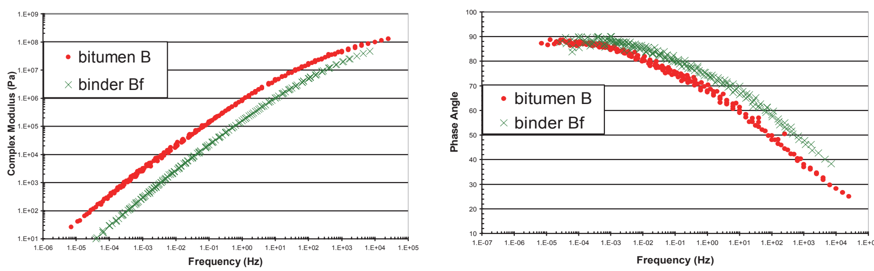


Figure 5a. *Master curves of complex modulus and phase angle at 25 °C for bitumen B and binder Bf*

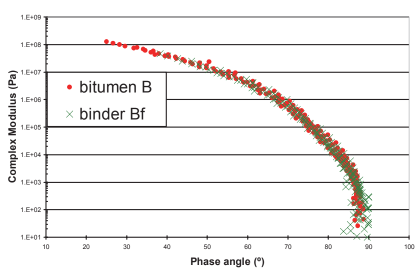


Figure 5b. *Black diagram for bitumen B and binder Bf*

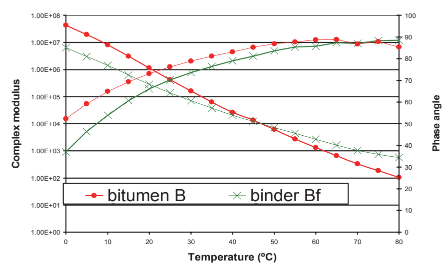


Figure 5c. *Isochronal plots of complex modulus and phase angle at 0.4 Hz for bitumen B and binder Bf*

4.3 Modified bitumens

Four laboratory blended tyre rubber-modified asphalt binders (TR-MAB), produced using the same kind and amount of rubber but different base binders, were tested and compared. The comparison was made in terms of rotational viscosity (Table 10), performance properties and rheological characteristics. Furthermore, to better understand the test results and properties, all the modified binders were subjected to a storage stability test and then compared in terms of softening point, polymer dispersion and rheological properties of their top and bottom sections after a period of hot storage.

Table 10. *Rotational viscosity @ 100 and 160ºC*

Rotational Viscosity (ASTM T316-04)	@ 100 ºC	@ 160 ºC
	Pa.s	
TR-MAB A	45.87	2.02
TR-MAB Af	19.85	1.46
TR-MAB B	49.44	2.40
TR-MAB Bf	24.18	2.01

4.3.1 Superpave Performance grade

The grading procedure for bitumens (AASHTO R 29-02) has been followed to find the performance grade (PG) of all the modified bitumens. Short-term ageing using the RTFO equipment was impossible to perform, firstly, due to the rubber particles not allowing the binder to form a uniform film (coat) on the internal surface of the glass containers and, secondly, due to "roll out" problems. The Thin Film Oven test (AASHTO T 240) was therefore used to simulate the short-term ageing of the TR-MAB. The performance grade test results in Tables 11 and 12 show three significant results. Firstly, all the TR-MABs have got a better performance temperature range in comparison with the base bitumen. Secondly, TR-MABs obtained without adding flux perform better at high temperature and show a wider performance temperature range, but have a higher viscosity at operating temperatures (135°C). Finally, comparing the results between the TR-MABs originated from the two different base bitumens, it is possible to notice that after modification with crumb rubber, bitumen B is the one which demonstrates the biggest improvements (Figure 6).

Table 11. *Performance grading of "TR-MAB A" and "TR-MAB Af"*

Aging states	**AASHTO specifications**	**TR-MAB A**	**TR-MAB Af**
Unaged binder	Rotational Viscosity (Pa.s)	@135°C/ 4.41	@135°C/ 2.75
	G*/sin(delta) (kPa) > 1.00	@76°C / 1.12	@76°C / 0.99
RTFO/TFO[1] aged residue	G*/sin(delta) (kPa) > 2.20	@76°C / 2.38	@64°C / 2.60
RTFO/TFO[1] +PAV aged residue	G* x sin(delta) (kPa) < 5000	@22°C / 4670	@13°C / 4000
	Stiffness@ (MPa) < 300	@-12°C / 181	@-18°C / 143
	m-value has to be > 0.3	@-12°C / 0.35	@-18°C / 0.38
1: Thin Film Oven test (AASHTO T 240) was used to simulate the short-term ageing of the TR-MAB			
Flash point (°C) has to be > 230		> 230	> 230
Mass loss after RTFO ageing has to be < 1.0%		*not possible*	*not possible*
Performance grade		PG 76 - 22	PG 64 - 28
Performance temperature range		98°C	92°C

Table 12. *Performance grading of "TR-MAB B" and "TR-MAB Bf"*

Aging states	**AASHTO specifications**	**TR-MAB B**	**TR-MAB Bf**
Unaged binder	Rotational Viscosity (Pa.s)	@135°C/ 6.45	@135°C/ 3.83
	G*/sin(delta) (kPa) > 1.00	@88°C / 1.24	@76°C / 1.83
RTFO/TFO[1] aged residue	G*/sin(delta) (kPa) > 2.20	@88°C / 2.60	@76°C / 2.96
RTFO/TFO[1] +PAV aged residue	G* x sin(delta) (kPa) < 5000	@19°C / 4610	@10°C / 3720
	Stiffness@ (MPa) < 300	@-18°C / 183	@-24°C/ 150
	m-value has to be > 0.3	@-18°C / 0.33	@-24°C/ 0.34
1: Thin Film Oven test (AASHTO T 240) was used to simulate the short-term ageing of the TR-MAB			
Flash point (°C) has to be > 230		> 300	> 230
Mass loss after RTFO ageing has to be < 1.0%		*not possible*	*not possible*
Performance grade		PG 88 - 28	PG 76 - 34
Performance temperature range		116°C	110°C

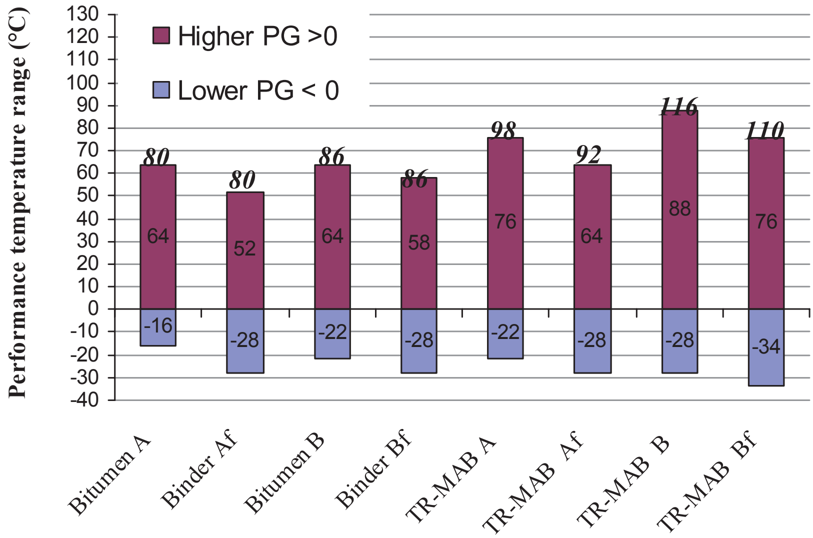
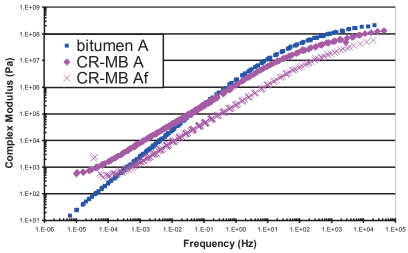


Figure 6. *Overview of the performance properties of all the binders*

4.3.2 Rheological analysis

The frequency dependence of complex modulus and phase angle for the modified bitumens has been assessed in Figures 7a-b and 8a-b by producing rheological master curves at a reference temperature of 25°C and black diagrams. Isochronal plots at low frequency (0.4Hz) were obtained to show the temperature dependency (Figures 7c and 8c). A comparison regarding the TR-MABs and the respective original base bitumens, shows how the modified binders have a lower frequency and temperature susceptibility. The results indicate that this is true for both kinds of modification, with or without oil extender, but with the softer base binder (with flux) showing, on average, lower stiffness values for the TR-MAB binders. The polymer network effect is clearly noticeable in Figures 7a,c and 8a,c which show unique phase angle curves due to the typical effect of the elastomers that significantly increase the elastic response of both modified binders especially at low frequencies and high temperatures (Airey, 2003). Moreover, the presence of a slight plateau on the complex modulus curves (Figures 7a,c and 8a,c) shows how the dominance of the polymer networks improves the stiffness of the modified binders. An overall analysis of the graphs shown in Figures 7 and 8 demonstrates that laboratory blended TR-MABs perform better rheologically, than their original base bitumens, showing a lower temperature and frequency susceptibility. The graphs show the effect of the oil extender on the TR-MABs as it does with the base binder as explained in Section 4.2. Similar to the effect seen between the base bitumens and base binders, the addition of the oil extender for the TR-MABs does not significantly change the shape of the black diagrams as shown in Figure 7b and 8b.



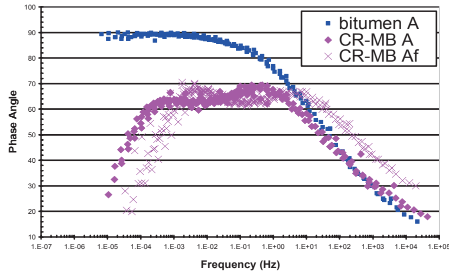


Figure 7a. *Master curves of complex modulus and phase angle at 25 ºC for "TR-MAB A", "TR-MAB Af" and their base "bitumen A"*

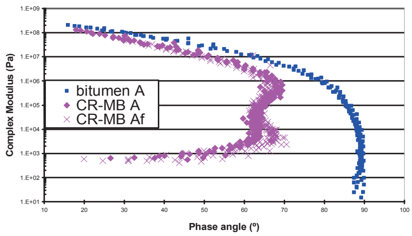


Figure 7b. *Black diagram for "TR-MAB A", "TR-MAB Af" and their base "bitumen A"*

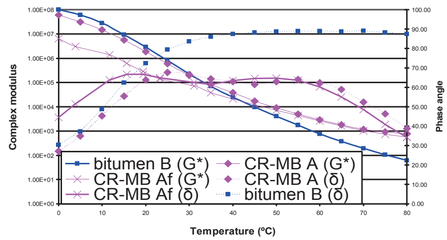
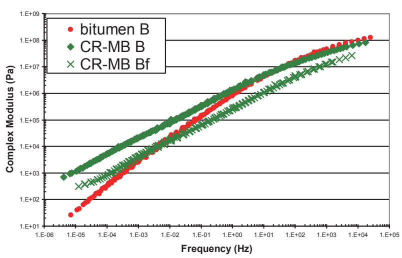


Figure 7c. *Isochronal plots of complex modulus and phase angle at 0.4 Hz for "TR-MAB A" and "TR-MAB Af"*



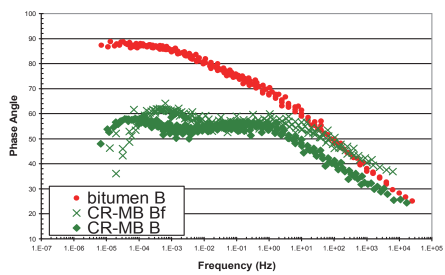


Figure 8a. *Master curves of complex modulus and phase angle at 25 ºC for "TR-MAB B", "TR-MAB Bf" and their base "bitumen B"*

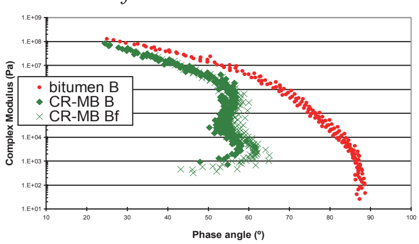


Figure 8b. *Black diagram for "TR-MAB B", "TR-MAB Bf" and "bitumen B"*

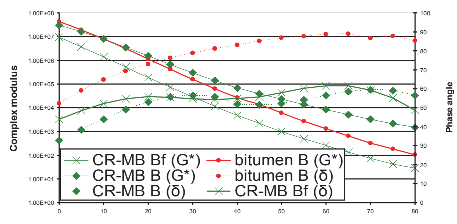


Figure 8c. *Isochronal plots of G* and δ at 0.4 Hz: "TR-MAB B", "TR-MAB Bf"*

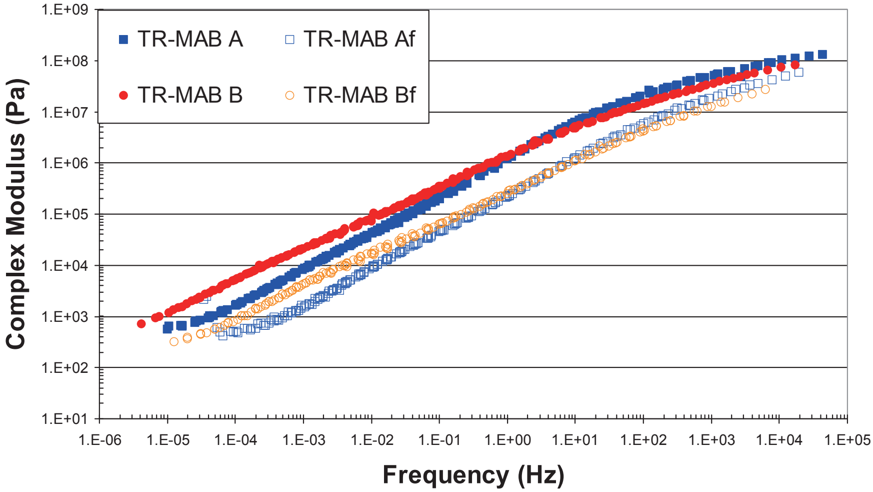


Figure 9. *Master curves of complex modulus at 25 ºC for all the TR-MABs*

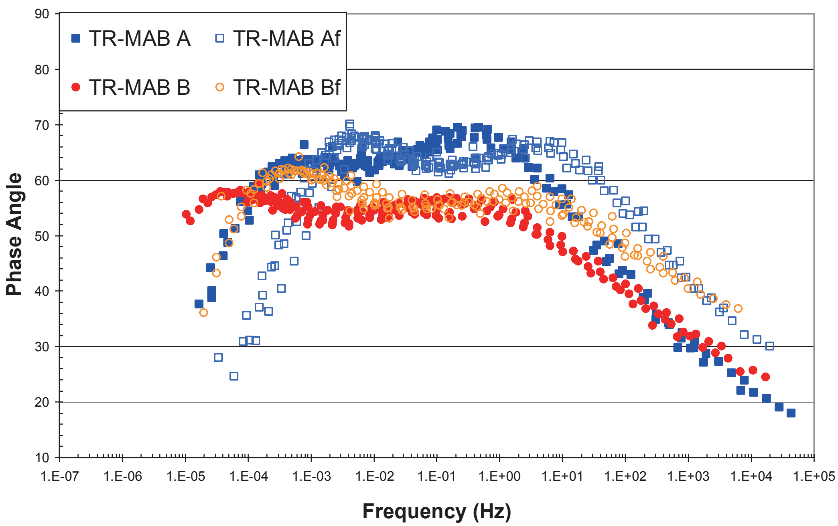


Figure 10. *Master curves of phase angle at 25 ºC for all the TR-MABs*

Further information comes from the comparison between the rheological properties of all the TR-MABs (Figures 9 to 12). Comparing the TR-MABs originated from bitumen B with the correspondent TR-MABs created from bitumen A. It is possible to notice that the modified binders obtained using the higher asphaltenes content base bitumen (bitumen B) are on average stiffer (higher G* values) and more elastic (lower phase angles) over the range of considered frequencies and temperatures (Figures 9 and 10). However, the TR-MABs originated from bitumen A demonstrate a greater degree of modification with the rubber. At low frequencies and high temperatures, they show slight plateaus on the complex modulus curves and isochronal plots (Figures 9 and 12) and have much lower values of phase angle (Figure 10). These are typical effects seen for elastomers which tend to increase the elastic response of the modified binders (Airey, 2003).

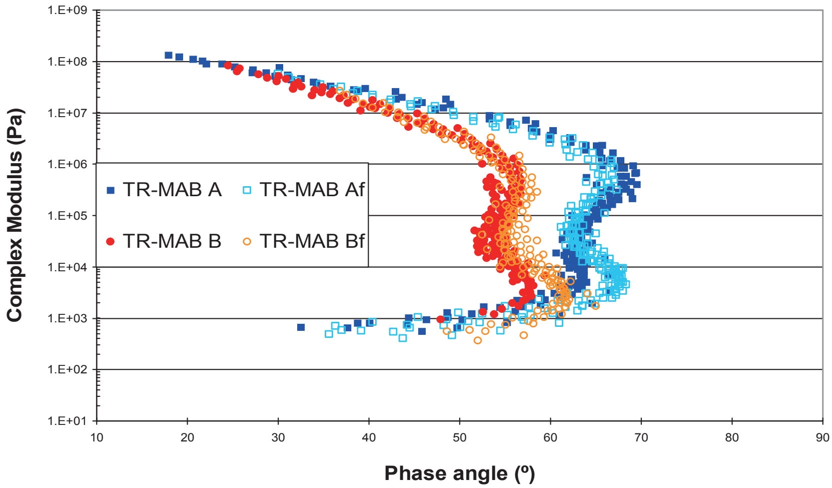


Figure 11. *Black diagrams for all the TR-MABs*

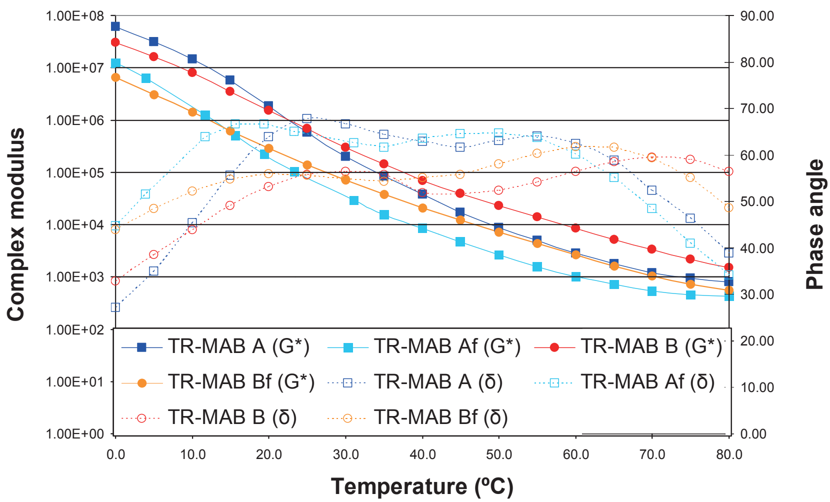


Figure 12. *Isochronal plots of complex modulus and phase angle at 0.4 Hz for all the TR-MABs*

4.3.3 Storage stability analysis

Another fundamental requirement that has been evaluated for the laboratory blended TR-MABs was their storage stability. In this analysis all the modified bitumens have been subjected to a hot storage test based on BS EN 13399, but modified due to the fast phase separation of the rubber. The procedure consisted of filling toothpaste tubes (three for each binder) with pre-heated modified binder (180°C) and then, once the tubes had been closed, placing them in a vertical orientation into an oven at 180°C. After two different periods (3 and 5 hours), one tube for each binder was taken out from the oven and was cooled down to ambient (room) temperature before placing into a freezer at -20°C. After that, the samples were cut into thirds and the top and bottom sections were saved to be further analysed.

In order to have an easily determinable parameter which could give a value of the storage stability, the softening points of both the modified binders for the top and bottom sections at three different periods of storage were measured (Tables 13 and 14). Softening point results show that the phase separation of the rubber starts to be relevant after three hours.

Table 13. *Softening points of TR-MAB A and TR-MAB Af before and after hot storage*

Softening point (ASTM D36-95)	**TR-MAB A**		**TR-MAB Af**	
	top	bottom	top	bottom
	°C			
before hot storage	60.5		49.8	
after 1h of hot storage	59.0	61.4	46.0	51.6
after 3h of hot storage	59.0	64.4	45.2	53.0
after 5h of hot storage	57.4	63.4	46.6	54.6

Table 14. *Softening points of TR-MAB B and TR-MAB Bf before and after hot storage*

Softening point (ASTM D36-95)	TR-MAB B		TR-MAB Bf	
	top	bottom	top	bottom
	°C			
before hot storage	71.0		60.5	
after 1h of hot storage	72.5	69.8	61.6	58.0
after 3h of hot storage	73.4	70.2	61.9	59.8
after 5h of hot storage	71.8	71.0	59.0	62.6

As found in previous research (Lu & Isacsson, 1997) the use of only the softening point to evaluate the storage stability of modified binders may be inadequate. Therefore, to have more information and more accuracy, a polymer dispersion analysis (PDA) and a rheological characterisation of the top and bottom sections of the modified binders, after three and five hours of hot storage, have been performed. PDA was performed using fluorescence microscopy which provided the images shown in Figures 13 and 16. These show, with a magnification of forty times, the binder dispersion of the crumb rubber before and after periods of storage. Lastly, the DMA was performed using the same conditions as used in Section 3.5 and presented in Figures 14, 15, 17 and 18.

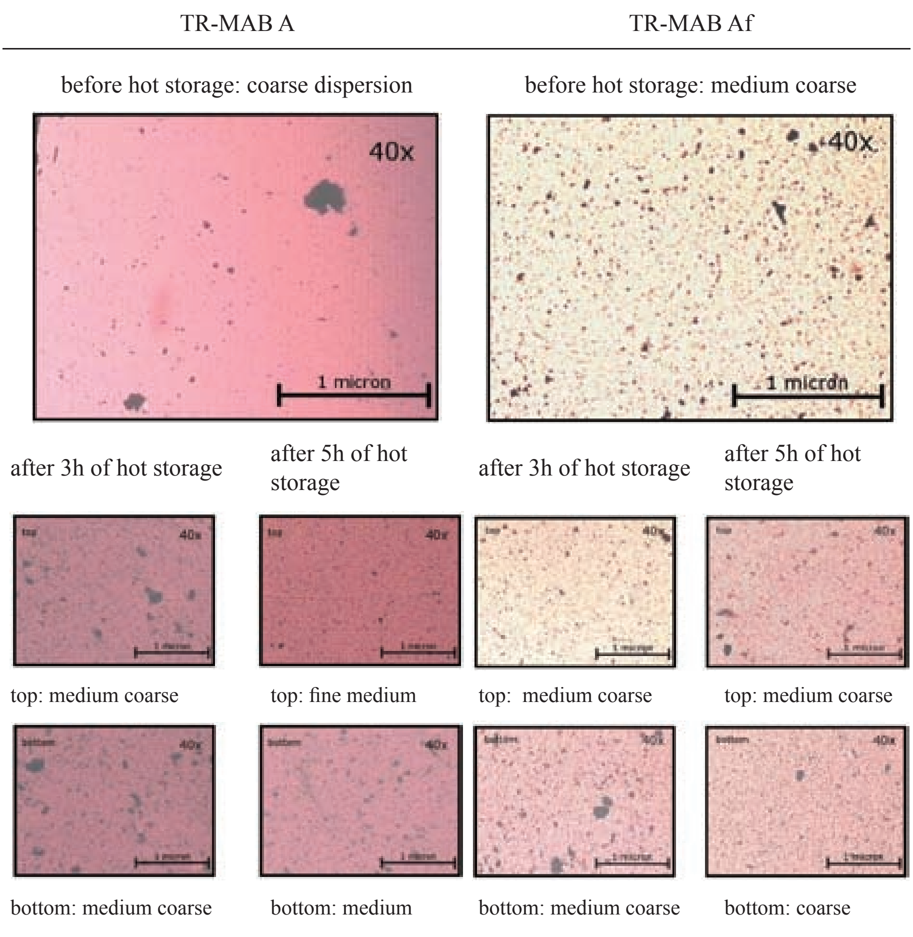


Figure 13. *Polymeric dispersion analysis of the top and bottom sections of "TR-MAB A" and "TR-MAB Af"*

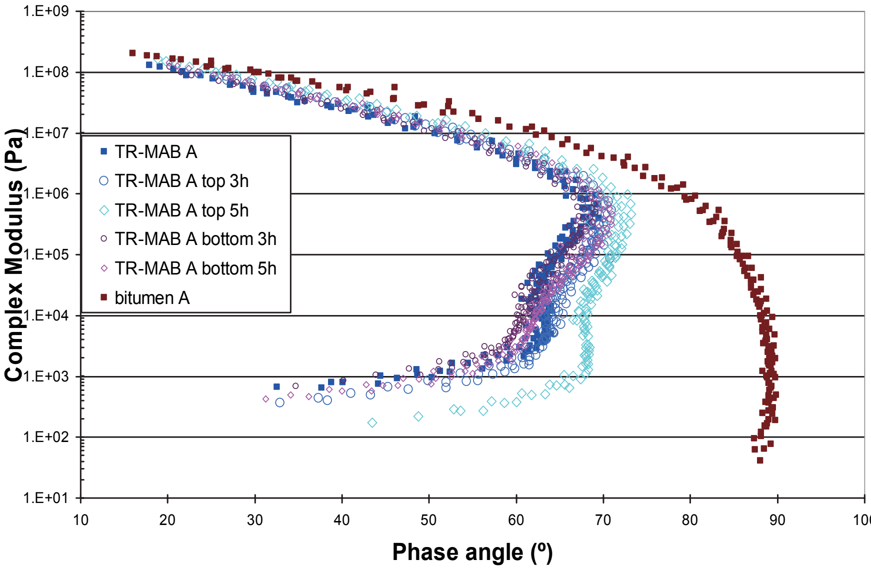


Figure 14. *Black diagrams of base bitumen A and TR-MAB A before and after hot storage; a) 3h hot storage, b) 5h hot storage*

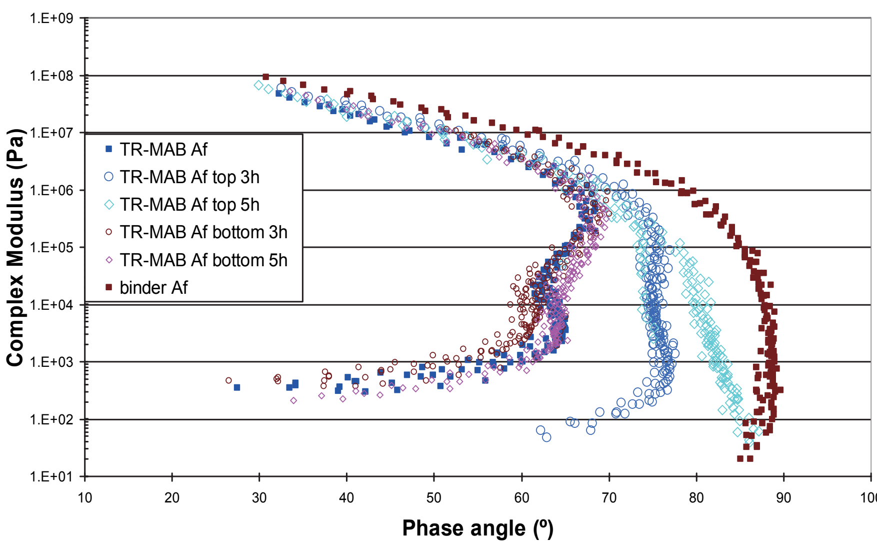


Figure 15. Black diagrams of base binder Af and TR-MAB Af before and after hot storage; a) 3h hot storage, b) 5h hot storage

The results of these analyses show that both TR-MABs originating from bitumen A (low asphaltenes content) begin to have a relevant phase separation after three hours of hot storage. As shown in Figure 13, the top section of TR-MAB A has a coarse dispersion of rubber which tends to settle toward the bottom. Comparing the images in Figure 13, TR-MAB A shows a lower variability between the top and bottom sections than the TR-MAB Af obtained after adding the flux to the same base bitumen. What it is suggested by the variability of the softening points has been confirmed by the rheological characterisation. The black diagrams in Figure 15 show how the finger print of the top section of TR-MAB Af differs considerable from that of the bottom. Moreover, it is interesting to note how, with increasing storage time, the top section tends to have a rheology closer to the base binder than the modified binder, while the bottom section becomes stiffer and more elastic. The same behaviour, but less emphasised, is shown by TR-MAB A in Figure 14.

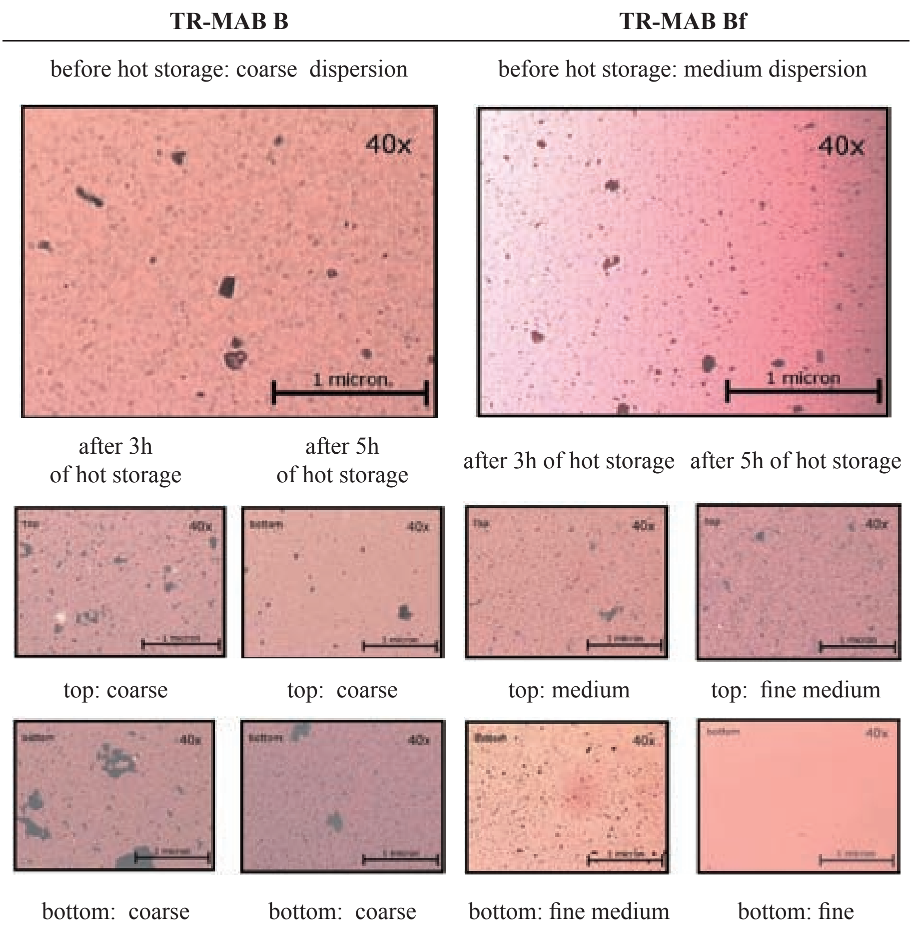


Figure 16. *Polymeric dispersion analysis of the top and bottom sections of "TR-MAB B" and "TR-MAB Bf"*

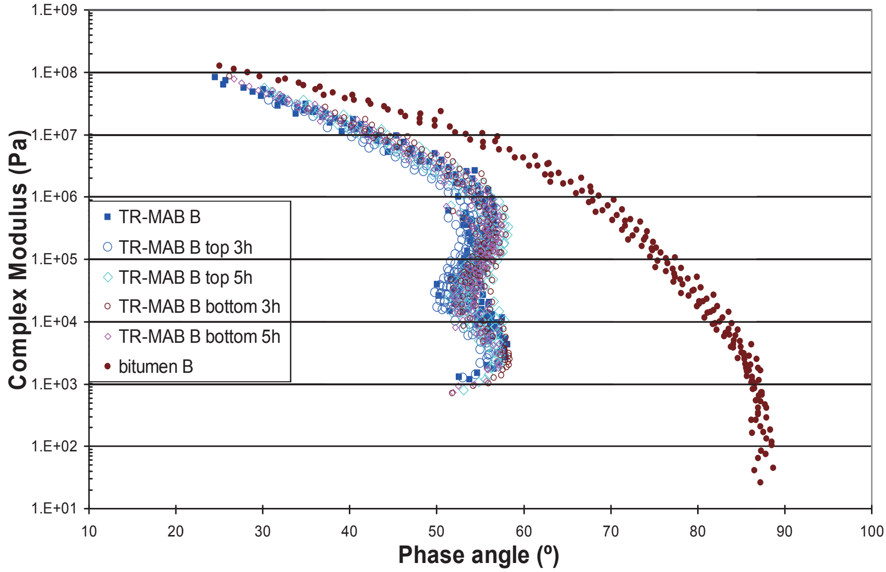


Figure 17. *Black diagrams of base bitumen B and TR-MAB B before and after hot storage; a) 3h hot storage, b) 5h hot storage*

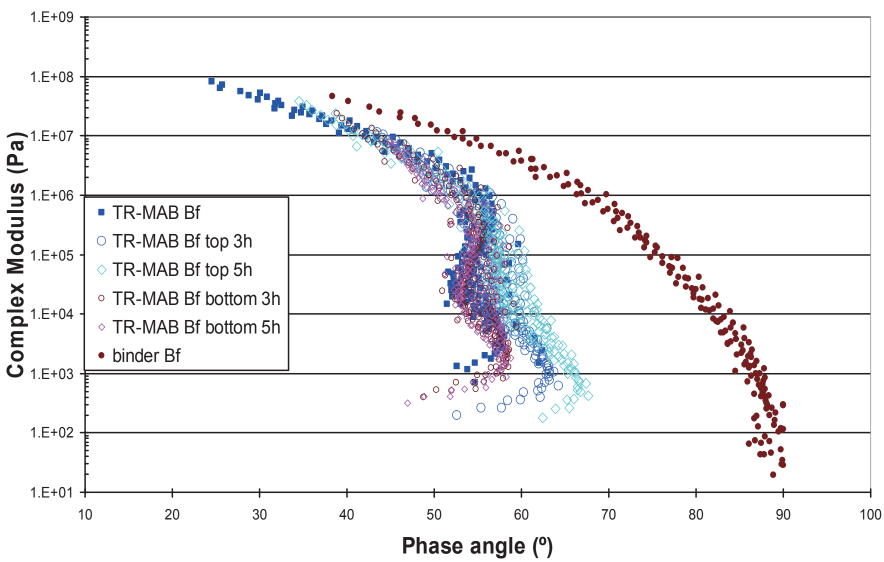


Figure 18. *Black diagrams of base binder Bf and TR-MAB Bf before and after hot storage; a) 3h hot storage, b) 5h hot storage*

PDA and rheological analyses for TR-MABs created from bitumen B (high asphaltenes content) confirm the results of the softening point tests (Table 14). These modified binders tend be much more stable than the ones obtained by modifying bitumen A. TR-MAB B has good stability showing no significant differences in terms of polymer dispersion between the top and the bottom sections after both periods of storage. It even shows a stable rheology (Figure 17) as well as having a constant softening point (Table 14). Rheological characterisation in Figure 18 shows how TR-MAB Bf is less stable than the base bitumen without flux. Contrary to what was expected, the laboratory blended TR-MABs produced with base bitumens without the addition of an oil extender tended to be more stable than the modified binders produced with the base binder (base bitumen plus flux) especially for the base bitumen with the lower asphaltenes content (bitumen A).

5. Conclusions

Four different Crumb-Rubber Modified Bitumens (TR-MABs), originating from two base bitumens with different asphaltenes content, have been blended and tested. Superpave performance grade, detailed rheological properties and storage stability analysis have been undertaken and used as the basis for the comparison. The results have indicated that all the modified binders have a considerable improvement in terms of their rheological, physical and performance properties compared to the original base bitumens (40/60 pen). However, fundamental differences have been found between the different blends:

- The TR-MABs obtained from the base bitumen with the lower asphaltenes content (bitumen A) show a greater degree of modification when mixed with crumb rubber. Their rheological properties demonstrate those typically found for elastomeric polymer modified bitumens.

- Despite this fact, the TR-MABs obtained from base bitumen B (higher asphaltenes content), show high viscosity, a better performance grade, lower temperature and frequency susceptibility and better storage stability.

- The addition of the oil extender in the TR-MABs softens the modified binder and also reduces performance range. In addition, the flux also appears to have a negative effect on storage stability.

- The overall effect of adding the oil extender is the same whether it is added solely to the base bitumen to form the base binder or used as a component in the production of TR-MABs. Furthermore, the addition of an oil extender to a low asphaltenes content bitumen (bitumen A) tends to have a detrimental effect on the modification in terms of the materials stability.

- It is possible to produce a high performance TR-MAB by mixing crumb rubber with a high asphaltenes content bitumen (bitumen B) without the need to add an oil extender, or at least not at the high content used in this study.

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Laboratory Performance Evaluation Of Gtr-modified Sma Mixtures With Fractionated Rap

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ABSTRACT. In 2006, the Illinois Tollway initiated a trial of hot mix asphalt (HMA) surface mixes modified with ground tire rubber (GTR). These mixes originally were intended to improve pavement noise, and they proved to have additional benefits for stone matrix asphalt (SMA) and open graded asphalt mixtures.

In 2007, the Tollway embarked on a field trial and laboratory pavement performance validation of an SMA asphalt surface mixture that uses fractionated recycled asphalt pavement (FRAP) and GTR. This highly recycled material was evaluated during mix design, field production, and laboratory performance testing .The Tollway selected a demonstration project site on a rural interstate highway with high truck traffic, then proceeded with mix designs, field production, and strict quality testing.

The mix design and field production documentation shows that GTR-modified SMA mixes with

FRAP can be designed and produced with success. The use of GTR eliminates the need for the fibers that often are required using other modifiers.

The laboratory evaluation of performance included such tests as dynamic modulus, beam fatigue, stability, rut testing, and tensile strength testing. The results of these tests indicate that the performance of the GTR-modified mixes is similar to other SBS modified HMA mixes commonly used in Illinois. These performance test data are the first validation of the use of GTR and FRAP in SMA mixtures under heavy interstate traffic.

KEYWORDS: Fractionated Recycled Asphalt Pavement (FRAP), Stone Matrix Asphalt (SMA), Ground Tire Rubber (GTR), Asphalt Fatigue, Asphalt Dynamic Modulus

1. Project Goals

To test fractionated recycled asphalt pavement (FRAP) materials, the Illinois Tollway, working through its contractors and consultants, developed and conducted a project on the applicability and feasibility of using increased amounts of recycled materials through FRAP. The goal of the program was to answer two main questions:

− Can the Tollway design, produce, and construct high-quality hot mix asphalt (HMA) pavements with high FRAP content mixes and specialty mixes with ground tire rubber (GTR)?

− Will these materials provide the same or better pavement performance as the standard mixes used by the Tollway and Illinois Department of Transportation (IDOT), and with performance that is consistent with pavement design procedures?

In the summer of 2007, a construction contract was awarded for advance pavement work on the Jane Addams Memorial Tollway (I-90) in the Rockford area. The timing, scope, and circumstances of the I-90 project provided a rare opportunity to evaluate several different HMA concepts directly via plant mixing and field trials.

This paper presents the results of the mixture testing. Performance testing included modulus, fatigue, stripping, and asphalt pavement analyzer analysis.

2. Introduction

In the summer of 2007, a construction contract was awarded for advance pavement work on the Jane Addams Memorial Tollway (I-90) near Rockford, Illinois. To test the new fractionated recycled asphalt pavement (FRAP) materials, Rock Road Companies and Rockford Blacktop, the Illinois Tollway, S.T.A.T.E. Testing, and Applied Research Associates, Inc. (ARA), with testing support provided by the Illinois Center for Transportation (ICT), developed and conducted a research project on the applicability and feasibility of using increased recycled asphalt pavement (RAP) contents through FRAP.

Hot mix asphalt (HMA) research testing usually is performed with laboratory-prepared mixes. For some circumstances, this is acceptable. For others, it may be all that is available. However, lab-prepared mixes usually are batched with oven-dried aggregates and mixed at relatively low oven temperatures. Lab procedures do an inadequate job of imitating the high temperatures and material handling that occurs in a plant. A plant-produced mix is better for answering questions like “how does the mixing of RAP with virgin aggregate and liquid asphalt in a plant affect the performance of the final mixture?”

The timing, scope, and circumstances of the I-90 project provided a rare opportunity to evaluate several different HMA concepts directly via plant mixing and field trials. This paper documents the activities involved in developing stone matrix asphalt (SMA) ground tire rubber (GTR) mix designs and producing/placing those mixes. The goal of the research program was to determine whether:

- The Tollway can design, produce, and construct high-quality HMA pavements with GTR mixes.

- These materials will provide the same or better pavement performance as the standard mixes used by the Tollway and Illinois Department of Transportation (IDOT), and with performance that is consistent with pavement design procedures.

2.1 Events surrounding this research project

The Tollway faces an unprecedented rehabilitation/expansion program for its highway network. The financial demand, market conditions, and desire to improve as much of the network as possible require the evaluation of options for minimizing construction costs. In the HMA industry, minimizing costs includes optimizing the selection of materials used in the mixes. The Tollway had been looking at small pieces of this puzzle—new liquid asphalt, different aggregates, and better performing mix designs. Nationally, fractionating RAP is becoming recognized as an efficient way to cut the cost of a new mix and reduce the inconsistencies of the high RAP mix properties without sacrificing quality. In May 2007, FRAP became a serious topic of interest at the Tollway when Don Brock, President of Astec Industries, made a convincing presentation to the Illinois HMA industry and various Illinois government agencies.

In the summer of 2007, a large contract for preliminary work on I-90 was awarded to a joint venture of Rock Road Companies and Rockford Blacktop. They readily agreed to work with the Tollway to test the FRAP concept by processing the mainline overlay grindings containing only high-quality manufactured aggregates into two fractions, category 1 fine portion FRAP (minus #4 sieve) and category 1 coarse portion FRAP (minus ½" sieve to #4 sieve); and by processing any other reclaimed asphalt pavement containing both lower quality natural and manufactured aggregates into two fractions, category 2 fine portion FRAP (minus #4 sieve) and category 2 coarse portion FRAP (minus ½" sieve to #4 sieve). This concept required additional processing and plant equipment and working with several additional material suppliers, all without an increase in their contract price. At the same time, the Tollway authorized S.T.A.T.E. Testing to proceed with designs for mixes which would include a significant percentage of FRAP which would include GTR. These mixes typically contained RAP contents 15% higher than the maximum percentages that the current IDOT or Tollway standard specifications allow.

During late summer and fall 2007, the joint venture produced these mixes. These mixes replaced standard mixes on the I-90 project. In most cases, each mix was placed over 2 or more days, allowing time for mix adjustments between production runs. The joint venture performed normal quality control (QC) testing. Quality assurance (QA) testing was performed by S.T.A.T.E. Testing. Ultimately, the mixes were sampled for HMA materials testing at IDOT and for more complex performance testing by the Illinois Center for Transportation at the University of Illinois.

2.2. New concepts

As with any research, new specifications, equipment, and procedures were required to conduct this project. These new concepts are noted below:

- A special provision was developed to implement FRAP with Illinois Tollway HMA mixtures. It included specifications for the source material and the production of the different FRAP products. It also included instructions on how to determine the specific gravity of the FRAP and how to accommodate design changes.

- Contractors acquired screening equipment and made modifications to their plants to accommodate the FRAP.

- Mix designs were created for the FRAP mixes that included non-traditional ingredient materials, notably terminally blended GTR, modified liquid asphalt, and coarse aggregates for the SMA mixes.

3. Materials tested

Nine HMA materials were plant produced and tested as part of this research effort, the details of which can be found in the project report (Vavrik 2008). These mixtures, their type, asphalt grade, and FRAP percentage are listed in table 1. The materials were selected because they are regularly used mixes in Tollway mainline and HMA shoulder expressway pavement sections.

The SMA mixtures included two surface mixes, one with trap rock coarse aggregate and another with steel slag coarse aggregate, and one SMA binder that used crushed gravel as the coarse aggregate source. The purpose of these mixtures was to evaluate the use of fine portion FRAP and to determine if the material properties and predicted performance of the resulting mixture were consistent with other SMA mixes previously produced in Illinois using virgin aggregate sources only.

Table 1. *FRAP SMA research mix matrix.*

Research Mix Number	PG grade	FRAP % Fine/Coarse	Coarse Agg. type
#1 – SMA Binder	PG 76-22	15 / 0	Crushed Gravel
#2 – SMA Surface	PG 76-22	15 / 0	Trap Rock
#3 – SMA Surface	PG 76-22	15 / 0	Steel Slag

4. Summary of field notes

The following notes summarize the characteristics of the trial mixes. These notes were compiled by S.T.A.T.E. Testing through their experiences from aggregates in mix designs and through their field observations of trial mix production and compaction. The complete mix design, mix design notes, and QA summary are provided in the project report (Vavrik 2008).

4.1 Mix 1 – SMA binder

- Quite likely the first Illinois SMA binder to include crushed gravel as the coarse aggregate. Dolomite is the customary coarse aggregate in most SMA binder mixes. Gravel is locally available and may better resist breakdown during compaction. The contractor used a blend of CM13 and CM14 from two different pits.

- The liquid asphalt was GTR-modified and complied with the PG 76-22 grade, with the exception of the tests on Residue From Rolling Thin Film Oven Test (AASHTO T 240).

- The FRAP proportion was 14% of the total mixture (fine portion category 1 FRAP). SMA mix designs typically require the use of a manufactured fine aggregate and require more liquid asphalt than conventional dense-graded mixes. The fine aggregate FRAP portion source in the SMA test mix was processed from mainline overlay grindings that consisted of high-quality crushed coarse aggregate and manufactured fine aggregate in the original overlay materials.

- Using GTR-modified liquid asphalt eliminated the use of fibers normally used to prevent liquid asphalt draindown during storage and transportation of SMA mixes produced with the SBS polymer modified liquid asphalt.

The contractor used two coarse aggregate products and feeders for control. Production took place over two nights. After the first night, minor proportioning adjustments were made to increase voids. The resulting volumetric properties (air voids, voids in mineral aggregate [VMA], and voids filled with asphalt [VFA]) were within specification tolerances. Limited production quantities restricted further adjustments and testing.

4.2. Mix 2 – SMA surface (trap rock)

- This is the first Illinois SMA surface mix that includes Diabase (trap rock) as the coarse aggregate. Diabase is a hard, non-absorptive aggregate with excellent friction properties. Most high-volume SMA surface mixes in Illinois have included steel slag as the coarse aggregate. The Diabase consisted of two different gradations—35% CM13 and 46% CM14.

- The liquid asphalt was GTR-modified PG 76-22.

- The FRAP proportion was 14% of the total mixture (fine portion category 1).

- The GTR liquid eliminated the customary use of fibers to prevent liquid draindown during storage and transportation.

The contractor used two coarse aggregate products and feeders for control. Production took place over two nights. Voids were slightly above target. VMA and VFA were within specification limits. Limited production quantities restricted further adjustments and testing.

4.3. Mix 3 – SMA surface (steel slag)

- This mix included steel slag as the coarse aggregate. The Heritage slag made up 80% of the aggregate blend—55% CM13 gradation and 25% CM11 gradation.

- The liquid asphalt was GTR-modified PG 76-22.

- The FRAP proportion was 15% of the total mixture (fine portion category 1 FRAP).

- The GTR liquid eliminated the customary use of fibers to prevent draindown during storage and transportation.

The contractor used two coarse aggregate products and feeders for control. Production took place during one day shift. The volumetric properties were within or close to specification tolerances. Limited production quantities restricted further adjustments and testing.

5. Performance evaluation methodology

The design of full-depth HMA pavements relies on two primary HMA material properties, the fatigue performance curve and the HMA modulus (stiffness). The dynamic modulus determines how much the pavement section flexes under the load of a heavy truck, resulting in strain in the asphalt layer. This strain is then evaluated against the fatigue performance curve, which relates the strain to the allowable loads that the pavement section can carry.

Other HMA material properties that are of concern include resistance to rutting and resistance to weathering. Relative rutting resistance is indicated by the dynamic modulus test, which is included in this testing program, and is controlled for most projects through the mix design process and by the selection of raw materials.

6. Asphalt mixture performance tests

Laboratory testing was performed to measure laboratory material properties and determine the expected performance of these mixes in field applications. The laboratory program was conducted on plant-produced mix on the second night of production. The materials were properly sampled, and the HMA tests were performed without incident. All of the identified mixtures were sampled at the plant and taken to the Advanced Transportation Research and Engineering Laboratory (ATREL) at the University of Illinois. ICT research staff reheated and split the mix, prepared test samples, and conducted all materials tests. The tests performed included flexural beam fatigue testing and dynamic modulus testing (AASHTO TP-321 and TP62-08, respectively).

6.1. Asphalt fatigue testing

Asphalt fatigue is a distress that develops with bottom-up cracking in a full-depth asphalt section. Fatigue test results also can be considered an indication of the fracture toughness of a mixture and its resistance to cracking that may develop from the top down. Asphalt fatigue testing requires the compaction of asphalt beams that are placed in a testing apparatus and subject to repeated bending. The equipment monitors the loads and deflections during this bending and identifies the failure point as when the sample loses half of its initial stiffness.

Asphalt fatigue tests were performed at between four and six strain levels, corresponding to strains that could be encountered in field conditions. These tests are used to develop a fatigue curve and fatigue slope that can then be compared with the fatigue curve assumed in pavement design. Fatigue curves with a higher fatigue slope than the design slope have the potential to provide a longer fatigue life. The asphalt fatigue slopes for the SMA GTR mixes are presented in figure 1.

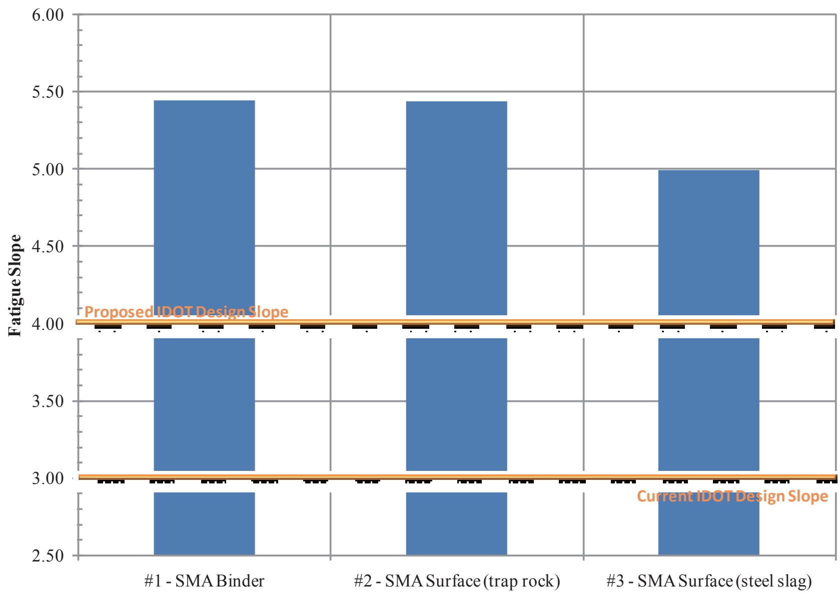

Figure 1. *Fatigue slopes for the study mixes plotted with the IDOT current and proposed design values.*

The test results show excellent fatigue performance for the SMA GTR mixes. This is consistent with the testing of normal SMA mixtures without RAP, as shown in testing conducted for IDOT on polymer and fiber modified liquid binder (Carpenter, 2007). The fatigue performance of the SMA mixtures can be attributed primarily to the GTR modified liquid that is used in these mixes, combined with the general characteristics of SMA materials.

6.2. Dynamic modulus (E)*

The modulus of HMA paving materials is one of the most important factors in establishing the pavement cross section and thickness. How to determine that modulus has changed over the years, but recent research has shown that the dynamic modulus is an appropriate method to develop comparative values.

The dynamic modulus test involves applying a compressive cyclical load to the HMA test sample. The deflection of the sample is measured by linear variable differential transformers (LVDT's) mounted on the HMA sample. The dynamic modulus is the maximum stress divided by the maximum strain.

The complete characterization of HMA modulus requires testing at 20℃, 4℃, and -10℃. The frequencies used varied from 0.01 Hz to 25 Hz. This combination of test parameters allows a characterization covering all but the extreme high temperature regime. All design temperatures and traffic speeds are covered by this testing program. Upon completion of the testing, a master curve is developed using time-temperature superposition techniques.

Dynamic modulus tests were performed on samples with 4% and 7% air voids at the temperatures and frequencies previously noted. These tests were used to develop master curves so that modulus values could be evaluated at a variety of temperatures and frequencies that are expected for in-service pavements. The HMA dynamic modulus at 20℃ for each of the mixes is shown in figure 2.

The dynamic modulus test data show that, for all mixes and all speeds, the modulus of the SMA GTR mixtures is higher than the values currently assumed for the IDOT mechanistic design procedure. This result demonstrates that SMA high FRAP GTR mixes will not require changes to the pavement thickness and that the performance of the roadway can be expected to be as good as current materials and designs. These values are very similar to values determined in testing of typical IDOT mixtures for an extended life pavement research project (Carpenter, 2007).

6.3. Additional testing

The IDOT Bureau of Materials and Physical Research was provided samples of the three SMA mixtures for evaluation of tensile strength ratio (TSR), stability, and testing in the asphalt pavement analyzer (APA) and in the Indenter. As detailed in the following sections, the SMA mixtures with FRAP and GTR have similar laboratory test results to other IDOT tested mixes and are therefore expected to perform similarly.

6.3.1. Tensile strength ratio

TSR testing (Illinois Modified AASHTO T-283) was performed on the SMA binder with gravel, SMA surface with steel slag, and SMA surface with trap rock. All tested samples passed the TSR test, with the gravel mix having the lowest TSR and the highest strength. Even after five freeze/thaw cycles, which generally will show a dramatic reduction of strength and TSR on susceptible mixes, the gravel SMA had good strength (103 psi). It had about the same rate of strength loss between the 140°F water bath strength and the freeze/thaw cycle strength as it had between no conditioning strength and the 140°F water bath strength. The TSR after five freeze/thaw cycles was 0.810, which nearly passed the Tollway criterion of 0.85

for 6-inch specimens after conditioning in the 140°F water bath only. The other two mixes had appropriate strengths and good TSR's. Figure 3 shows a chart of the strength and TSR data.

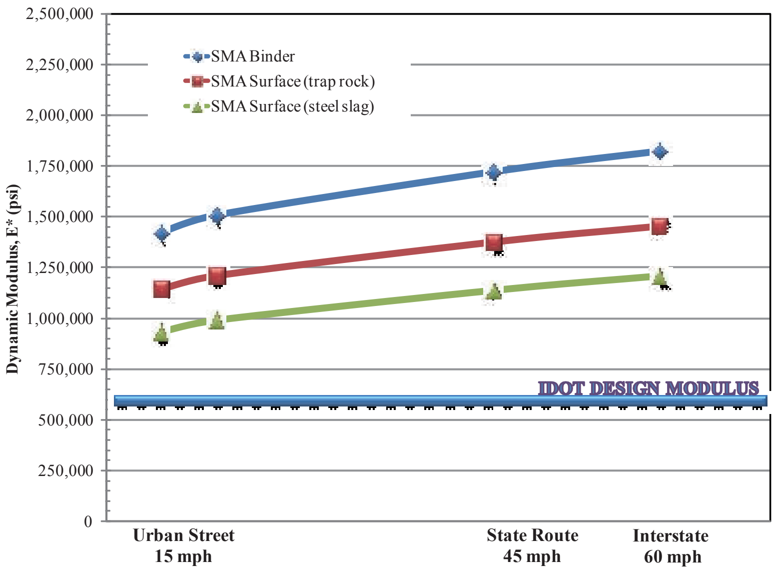

Figure 2. *Dynamic modulus test data at 20 and various highway speeds with the IDOT design range for northern Illinois.*

6.3.2. Stability

Both 4-inch Marshall and 6-inch Superpave gyratory stability tests were performed on the second sample of SMA binder with gravel and the trap rock samples. There was no significant difference between the two mixes. On the 4-inch samples, the stability of the gravel was slightly greater than that of the trap rock. For the 6-inch samples, the stability of the trap rock was somewhat greater than that of the gravel binder. Figure 4 presents the stability data.

6.3.3. Asphalt pavement analyzer

For the APA, IDOT evaluated the number of cycles per mm of rut depth, as well as the rut depth only, following the guidance in AASHTO TP 63-03. IDOT primarily uses the "cycles per mm" for the steel wheels when both the rut depth and the number of cycles until the end of the test vary. With the hoses and the steel wheels, these mixes easily lasted the full 8,000 cycles (for the hoses) and 20,000 cycles (for the submerged steel wheels test), so only the rut depth varied.

- Hose: The trap rock had slightly more cycles per mm of rut and a slightly lower rut depth. The steel slag had the least cycles per mm of rut and the greatest rut depth. All the results were close and were good.

- Steel Wheel: The gravel had slightly more cycles per mm of rut and, by a slight amount, had the lowest rut depth. The steel slag had the least cycles per mm of rut and the greatest rut depth. Again, all the results were close and were good.

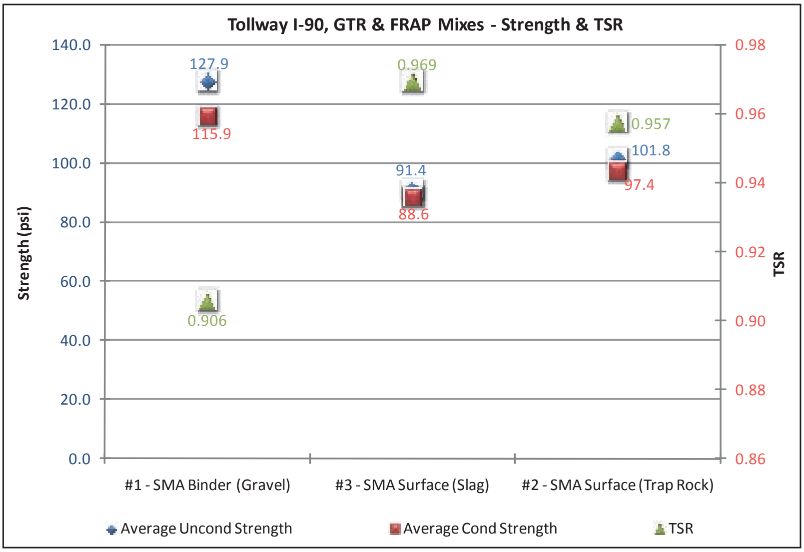

Figure 3. *Strength and TSR data for the SMA mixes.*

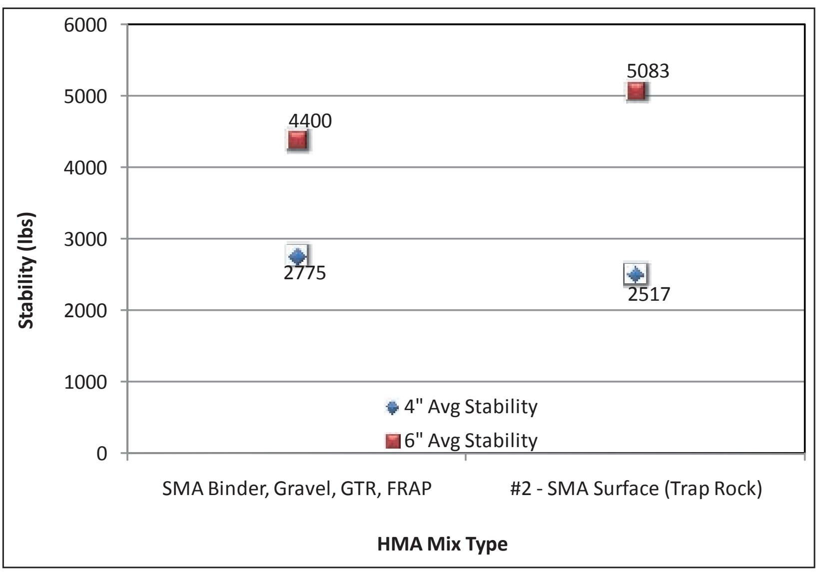

Figure 4. *SMA stability for 4- and 6-inch samples.*

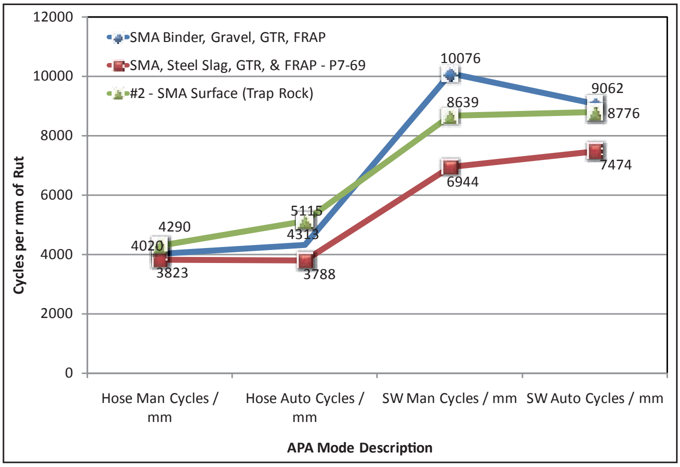

Figure 5. *APA performance data for the SMA mixtures.*

6.3.4. Indenter

The SMA gravel binder mix and the trap rock mix were tested in the Indenter, which measures deformation of an HMA gyratory specimen at an in-service temperature. All Indenter specimens start off at approximately 152 mm height (115-mm specimen and 36.7 mm for the Indenter). These specimens all ended up around 135 to 136 mm after 300 gyrations. Compared with all the other Indenter tests IDOT has run for the last 2 or 3 years, this is somewhere in the middle. The deformation/number of gyrations slope is fairly straight throughout the 300 gyrations. Some mixes deform quickly then level out. Since the deformation is fairly gradual, it is better than if it deformed quickly, even if it ended up at the same point.

7. Conclusions

Given the fatigue and modulus performance test data, two conclusions can be drawn at this time. First, the fatigue performance of all high FRAP GTR mixes is above the current design criterion. Second, the SMA materials with GTR and fine FRAP have material properties similar to those of other SMA mixes. It is expected that these high RAP GTR modified SMA mixes will perform similar to other SMA materials produced with virgin aggregate sources and the SBS polymer modified asphalt.

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Steady Shear Properties of Crumb Rubber Modified Bitumen

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ABSTRACT. The rheological properties of crumb rubber modified bitumen poses considerable challenges in terms of quantification. This investigation concerns the measurement of shear stress and normal stress difference when samples of crumb rubber modified bitumen were tested in a dynamic shear rheometer in a steady shear mode for a range of temperature and shear rates. Two commercially available crumb rubbers (I and II) were blended with an air blown bitumen. Three different dosage rates were used and the testing was carried out. It is seen that the material exhibit considerable normal stresses and normal stress difference when subjected to shear. A coarser graded crumb rubber (crumb rubber-I) results in considerable reduction in shear stress and normal stress difference for any dosage rate, shear rate and temperature. A finer graded crumb rubber (crumb rubber-II) results in considerable higher stress-overshoot in shear stress and normal stress difference for any dosage rate, shear rate, and temperature. For a given crumb rubber type and temperature, as the shear rate is increased, the normal stress differences increases at a much higher rate than the shear stress. These measurements clearly depict the difference in terms of crumb rubber type and dosage rates. These kinds of fundamental rheological experiments, when appropriately interpreted, shed considerable light on the interaction of crumb rubber with bitumen.

KEYWORDS: Steady Shear, Shear Stress, First Normal Stress Difference, Crumb Rubber, Air Blown Bitumen, Stress Overshoot

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1. Introduction

India is currently proceeding with one of the ambitious bituminous pavement building exercise. More than US$ 50 Billion is being spent on connecting the length and breadth of the country with four to six lane roads. The first phase of the road construction is completed and efforts are on to quantify the performance of the constructed facilities. It is interesting to note that considerable amount of crumb rubber modified bitumen is being used in the pavement construction.

Bitumen is processed in India through various methods. Depending on the crude source and the viscosity/penetration grade required, bitumen is air-blown or blended (IS73, 2006). In the air-blowing process, a conventional method of blowing compressed air by simple distribution rings is used. Air-blowing is also carried out in some refineries through a patented BITUROX unit. Considerable variability exists related to the crude source and processing methods across the country and efforts are underway to take into account the variability related to the physical, chemical and rheological characteristics.

Research on modification of bituminous binders with crumb rubber has gained significant interest world-wide due to the increased environmental concerns related to the disposal of scrap tire. It is claimed that addition of crumb rubber can increase the stiffness of the binder at high temperature and reduce the stiffening of the binder at low temperature thus effectively expanding the temperature range to which the binder can be used (Morrison and Hesp 1995). Interesting history related to the use of crumb rubber in bitumen and bituminous mixtures and the details related to dry and wet process can be seen from the reference of Morrison and Hesp (1995). Rheological investigations related to the use of devulcanized rubber tyre particles in bitumen and their fracture resistance are also discussed in Morrison and Hesp (1995). Considerable investigation and understanding related to crumb rubber – asphalt interaction is mainly through the studies of **Amirkhanian and co-workers (Amirkhanian 2003; Lee at al. 2006;** Putman and Amirkhanian 2006; Shen *et al*. 2009; **Thodesen et al. 2009a**).

It is well known that crumb rubber produced by the cryogenic process differ significantly when compared with the crumb rubber produced in the ambient conditions (Blumenthal 1994). The surface area for a given particle size is much higher for crumb rubber produced in ambient condition due to the mechanical tearing action. Complex reactions take place when crumb rubber is ingested with bitumen. Depending on the type of bitumen (high/low viscosity grade) and depending on the type of crumb rubber (content, particle size distribution and surface area characteristics), one can postulate a wide variety of reactions. Absorption of aromatic oils, swelling of rubber particles, possible dissolution of rubber components and de-volatilization and cross-linking in rubber can be some of the reactions (Shen *et al*. 2009). It is also postulated that the increase of apparent viscosity of crumb rubber modified asphalt is due to two main mechanisms, the first one is due to the swelling of crumb rubber particles resulting in reduced space for the particles to move around and the second is due to the stiffening of the binder (Abdelrahman and Carpenter 1999; Shen and Amirkhanian 2005).

Most of the work related to crumb rubber modified bitumen have been mainly related to the measurement of viscoelastic properties assuming that the material is linear viscoelastic. Hence, it is not uncommon to see investigations trying to elicit the performance grade properties of crumb rubber modified in terms of crumb rubber properties (see for instance Thodesen *et al*. 2009b). One need not overemphasise the necessity to characterize the behaviour of the material in terms of fundamental rheological tests and then build appropriate constitutive models towards the same. The experimental investigations reported in this study are aimed towards that step.

In the following we first discuss the rheology of steady shear flow. Two types of crumb rubber particles were blended with an air blown bitumen at different dosage rates and the blend was tested at four different temperatures for four shear rates. These details are given in section 3. In section 4, we summarize the results and discuss the possible reasons for such wide range of behaviour when two different types of crumb rubber are used.

2. Rheology of Steady Shear Flow

One of the most fundamental viscometric flows is the simple shear flow. The velocity fields for such a flow is given as follows,

$$v_x = \dot{\gamma}_{yx} y; \quad v_y = 0; \quad v_z = 0, \qquad [1]$$

where $\dot{\gamma}_{yx}$ is the velocity gradient and v_x, v_y, and v_z are the velocity fields. For steady shear flow, the shear rate (the absolute value of velocity gradient) is independent of time. This is one of the classic flows which are normally used for characterizing the rheological properties of viscoelastic fluids.

Constitutive equations relate the strains developed in the material to the stresses and bulk of the experimental work is conducted to establish these relationships. The constitution of the material and its mechanical response plays an important role in developing a proper constitutive model. For instance, if the fluid is Newtonian (viscosity independent of shear rate), the only component for the stress tensor which is non-zero is the shear stress τ_{yx} for the velocity field given in equation [1]. It is interesting to note here that when a viscoelastic fluid is subjected to shearing, one sees considerable complexity in terms of the components of the stress tensor. Here it is possible that all the six independent components of the stress tensor may be non-zero. The most general form of the stress tensor for the case of simple shearing flow for such fluids is given as follows (Bird *et al*. 1987),

$$\boldsymbol{\pi} = p\boldsymbol{\delta} + \boldsymbol{\tau} = \begin{bmatrix} p+\tau_{xx} & \tau_{yx} & 0 \\ \tau_{yx} & p+\tau_{yy} & 0 \\ 0 & 0 & p+\tau_{zz} \end{bmatrix} \qquad [2]$$

where $\boldsymbol{\tau}$ is the stress tensor, p is the thermodynamic pressure and $\boldsymbol{\pi}$ is the total stress. Now the stresses that are customarily used in characterizing the shear flow are essentially shear stresses given by τ_{yx} and the normal stress differences. As can be seen from equation [2], there are two normal stress differences, the first normal stress difference given by $(\tau_{xx} - \tau_{yy})$ and the second normal stress difference given by $(\tau_{yy} - \tau_{zz})$. Using the above basic definition, in non-Newtonian fluid mechanics literature, it is customary to define a non-Newtonian viscosity or shear-rate dependent viscosity (η) and two normal stress coefficients (Ψ_1, Ψ_2) and all these three functions are collectively called as viscometric functions (Bird *et al*. 1987).

In a dynamic shear rheometer normally used for characterizing the viscoelastic properties of bitumen, the top plate is rotated and the bottom plate is kept fixed. The torque and the normal force engendered during this motion are measured. Typically, in bitumen literature, the torque (and hence the shear stress) and the angular velocity (and hence the shear rate) are of main interest and different types of experiments are performed to elicit the mechanical characteristics of bitumen. However, over and above the torque and angular velocity measurements, the normal forces hold considerable information related to the rheological characteristics of the material. Typically, in a steady shear flow where the upper plate is subjected to constant angular velocity, one can measure in a dynamic shear rheometer the torque required to achieve this rotation as well as the total normal force required to maintain the disks at a separation of any given height. When a material like bitumen is sheared, considerable normal forces are developed in the material (equation [2]) and these forces tend to push the upper plate apart. Since the shearing is conducted at a constant height, the dynamic shear rheometer applies an equal and opposite force during shear to maintain this shear and this normal force along with the torque are the raw data which can be collected during measurement. An equipment specific material constant can be used to convert torque to shear stress, angular velocity to shear rate and the normal force to first and second normal stress differences.

The measurement of normal forces during shear is as old as experimental mechanics. In his seminal paper on granular materials, Reynolds (1885) discussed a quintessential feature concerning the response of granular solids, that of "dilatancy". Granular materials when sheared undergo a change in volume and in order to keep the volume constant, one has to apply a force, i.e., shearing engenders forces perpendicular to the plane of shear. This phenomenon is a characteristic shared in common by many non-linear materials and the underlying physics goes by different names in different fields. It is called the "Poynting effect" in non-linear elasticity (Poynting 1909), "Weissenberg effect" in non-Newtonian fluid mechanics (Weissenberg 1947) and "dilatancy" in the theory of granular solids (Reynolds 1885). Review of literature related to bitumen and bituminous mixtures show that scant literature exists related to measurement of this aspect (Stastna *et al*. 2003, Kasula *et al*. 2005, Wekumbra *et al*. 2007) let alone developing sophisticated constitutive models to predict such response.

It is noted here that the shear rate is not constant in the parallel plate rheometer. The velocity field for the parallel plate is given as $v_\theta = r\,w(z)$, $v_r = 0$ and $v_z = 0$, and the shear rate is $\dot{\gamma} = r\frac{dw}{dz}$. Here the angular velocity is given as w and it is seen that the shear rate varies across the plate. The shear rate at the periphery is maximum and this is the shear rate which is used in all the material parametric calculations. There are also considerable disadvantages due to inertia as well as edge fracture in the parallel plate assembly. It is also interesting to point out here that edge fracture due to the extremely high shear rate at the periphery is an artifact of the equipment and it is not related in any manner to the rheological properties of the binder. Interesting discussions related to whether one can quantify the edge fracture seen in oscillatory motion of a dynamic shear rheometer as depicting the fatigue and fracture response of asphalt films can be seen in the paper by Anderson et al. 2001.

Another important parameter during steady shear flow is the behavior of viscoelastic materials during transient shear (start-up) flow (Bird *et al*. 1987). Transient studies have revealed striking nonlinear phenomena, e.g., a shear stress overshoot after sudden imposition of a fixed finite shear rate. In fact, these nonlinear overshoot effects have proved to be extremely useful in studies concerned with developing models that depict the real material response. While this aspect has been studied extensively in polymer rheology, this is not studied at all in detail in bitumen rheology and very few experiments have been conducted to characterize the startup of the steady shear flow. In fact one of the investigations that reports few experimental results related to the stress growth coefficient in polymer modified asphalt is due to Wekumbura et al. 2007. In these experiments, essentially for very small shear rates only, the shear stress approaches its steady state monotonically (Bird *et al*. 1987). For large shear rates, the shear stress reaches a maximum and then approaches the steady state value.

In this investigation, two fundamental aspects of steady shear flow are investigated. The first one is the development of shear stress and normal stress differences when the material is subjected to steady shear. The second aspect is the overshoot of shear stress during steady shear conditions and the influence of shear rate on this overshoot.

From the discussion related to the mechanism of bitumen – crumb rubber interaction, it is seen that considerable swelling takes place when crumb rubber is ingested with bitumen. When this swelled fluid mixture is subjected to steady shear, the resistance of the particle during shear engenders normal forces perpendicular to the plane of shear. Hence, one should ideally notice considerable increase of normal forces as the dosage of crumb rubber is increased. As the normal forces are dependent on shear rate, one should also notice the incidence of higher normal forces as the shear rate is increased for a given crumb rubber percentage. In the following we show through experimental investigation normal stress differences and stress overshoot.

3. Experimental Investigations

3.1. Materials

Two types of commercially available crumb rubber were used. Table 1 show their gradation and table 2 gives their relevant properties. Both these crumb rubber were manufactured under ambient conditions.

Table 1. *Sieve Analysis of Crumb Rubber*

Sieve number (mesh size in micron)	Crumb rubber – I (percentage retained)	Crumb rubber – II (percentage retained)
30 (600)	0.48	0.20
40 (425)	3.46	1.77
60 (250)	50.02	34.76
100 (150)	40.95	54.82
200 (75)	4.17	6.26
Pan	0.92	2.19

Table 2. *Crumb Rubber Properties*

Property	Crumb rubber – I	Crumb rubber – II
Moisture + volatile content (up to 300 ℃)	0.27%	0.28%
Ash content	4.22%	10.78%
Toluene insoluble	84.7%	80.9%

The bitumen used was provided by Mumbai Refinery, India and confirmed to the VG-30 grade of IS: 73-2006. In terms of penetration grade, this material confirms to the 60/70 grade. Bitumen and the crumb rubber were prepared by mixing together at 180 ℃ and the blend was kept in the agitation with high speed homogenizer for 1 hour at 4000 rpm with temperature around 175-180 ℃. The samples were then stored in separate cups for further testing. During the start of each and every testing, the sample was slightly warmed up in a water bath maintained at 60 ℃ and a steel spatula was used for completely mixing and homogenizing the blend. Considerable variations are normally noticed when the testing was conducted without this step.

3.2. Steady Shear Experiments

Anton-Paar dynamic shear rheometer (MCR301) was used for measurement in all the investigations. In addition, these experiments were repeated at the R&D facility of Bharat Petroleum Corporation Limited in the Anton-Paar dynamic shear rheometer (MCR501) for reproducibility. Parallel plate measurement system of 25 mm diameter was used with a gap of 0.8 mm. The plate diameter and the gap were chosen taking into account the torque and normal force sensor capabilities of both the equipments. The material was sheared for 300 second with data points collected every 0.01 second. The test matrix used is given below in table 3.

Table 3. *Test Matrix*

Variables	Details
Crumb rubber type	I and II
Dosage rate	8, 10, 12 %
Shear rate (temperature)	1, 2, 5 and 8 s-1 (60 ℃) 5, 10, 15 and 20 s-1 (70 ℃) 30, 40, 50 and 60 s-1 (80 ℃) 40, 50, 60 and 70 s-1 (90 ℃)

4. Results and Discussions

Figures 1(a) – (d) show the results of the steady shear experiments conducted at 60 ℃ for the two types of crumb rubber used in this investigation. For the sake of illustration, only shear rates 1 and 8 s -1 are shown here. The influence of the size fractions of the two types of crumb rubber are clearly seen here. As the material is sheared, bitumen blend with finer crumb rubber fractions exhibit significant shear and normal stresses.

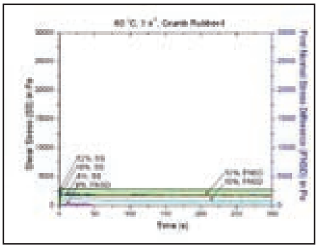

Figure 1-a. *Crumb Rubber – I at 1 s^{-1} for various dosages*

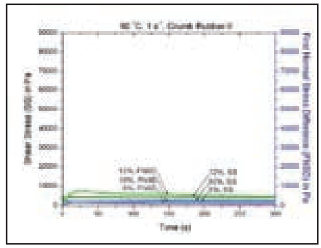

Figure 1-b. *Crumb Rubber – II at 1 s^{-1} for various dosages*

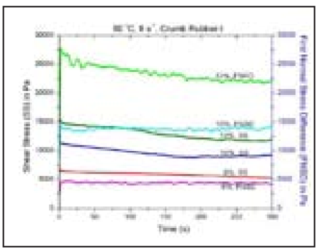

Figure 1-c. *Crumb Rubber – I at 8 s^{-1} for various dosages*

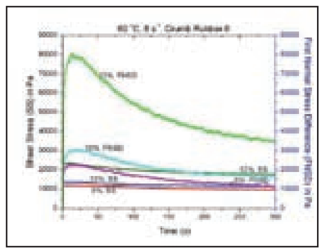

Figure 1-d. *Crumb Rubber – II at 8 s^{-1} for various dosages*

It is also interesting to note that at higher dosage rates and for the same shear rates, crumb rubber particles with finer particle size exhibit first normal stress difference higher than shear stress. As the numbers of particles are more for the given percentage, the particle surface area is increased resulting in increased capacity for absorption of aromatics. When this blend is sheared, the normal stresses also increases.

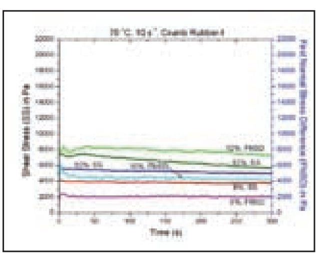

Figure 2-a. *Crumb Rubber – I at 10 s-1 for various dosages*

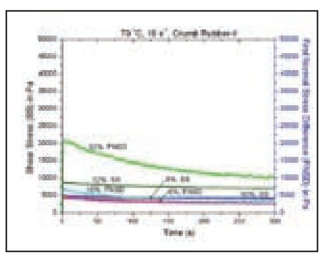

Figure 2-b. *Crumb Rubber – II at 10 s-1 for various dosages*

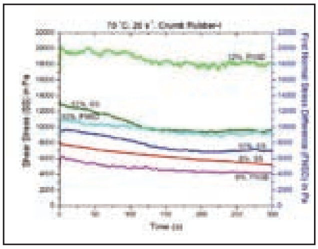

Figure 2-c. *Crumb Rubber – I at 20 s-1 for various dosages*

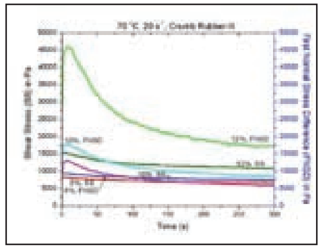

Figure 2-d. *Crumb Rubber – II at 20 s-1 for various dosages*

Figures 2(a)-(d) show the results for 70 ℃ for various dosage rates for the two crumb rubbers tested. The manner in which the shear stresses and first normal stress differences are evolving with time as the material is being sheared is clearly seen. It is also interesting to note the considerable amount of stress overshoot and decay in case of first normal stress difference for the finer crumb rubber particles. Due to the increased packing associated with finer particles, the resistance increases initially and as the material is being systematically sheared, this resistance systematically decays with time. The similar behaviour is also seen at 80 ℃ as shown in figures 3(a)-(d).

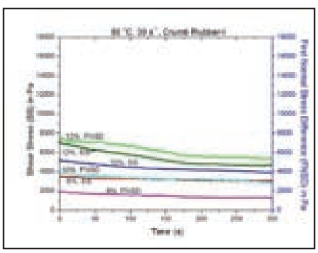

Figure 3-a. *Crumb Rubber – I at 30 s^{-1} for various dosages*

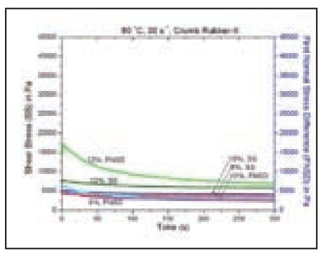

Figure 3-b. *Crumb Rubber – II at 30 s^{-1} for various dosages*

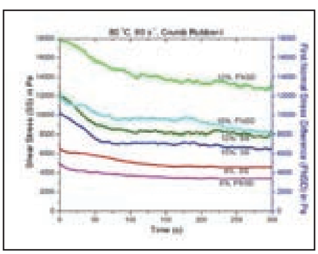

Figure 3-c. *Crumb Rubber – I at 60 s^{-1} for various dosages*

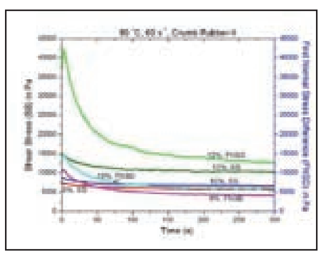

Figure 3-d. *Crumb Rubber – II at 60 s^{-1} for various dosages*

It is noted from figures 2(a)-(d) and 3(a)-(d) that as the shear rate is increased for the same temperature, the first normal stress difference is much higher than the shear stress for a higher dosage. However, the increased magnitudes of first normal stress difference with shear stress occurs at a lower dosage rate for the crumb rubber-II grade material as the particles in this case are much finer. Figures 4(a)-(d) show shear stress and normal stresses change for two shear rates for all the dosage rates at 90 ℃.

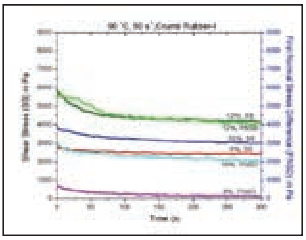

Figure 4-a. *Crumb Rubber – I at 50 s^{-1} for various dosages*

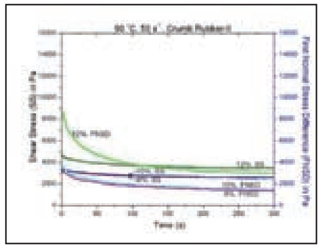

Figure 4-b. *Crumb Rubber – II at 50 s^{-1} for various dosages*

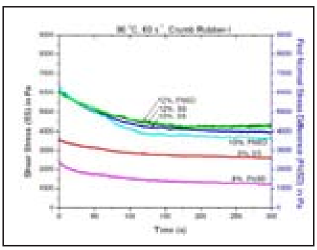

Figure 4-c. *Crumb Rubber – I at 60 s^{-1} for various dosages*

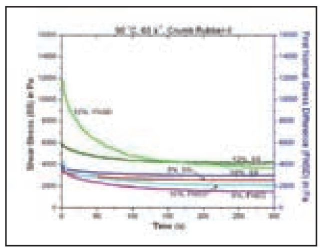

Figure 4-d. *Crumb Rubber – II at 60 s^{-1} for various dosages*

It is seen that as the temperature increases, the mobility of the particles to slide past each other increases and hence this results in the first normal stress difference always less than the shear stress. However, the blend with crumb rubber-II at a higher dosage rate of 12% still exhibits higher normal stress difference initially but decays to have value equal to that of the shear stress (figure 4-d).

One important manifestation of the non-linear behaviour of viscoelastic fluid is the tendency to exhibit stress-overshoot during the inception of steady shear flow. To highlight the stress-overshoot exhibited, two representative graphs are shown below in figures 5-a and 5-b. The influence of particle size distribution is more pronounced in terms of first normal stress difference. This is clearly evident from figure 5-b where one can see that the crumb rubber-I blend reaches its plateau much earlier than blend having crumb rubber-I for all dosage rates. In case of shear stress (figure 5-a), the magnitude of the shear stress shoot depends clearly on the dosage rate and on the type of crumb rubber used.

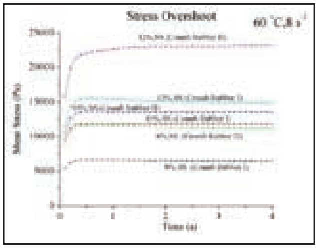

Figure 5-a. *Crumb Rubber – I at 60 s^{-1} for various dosages*

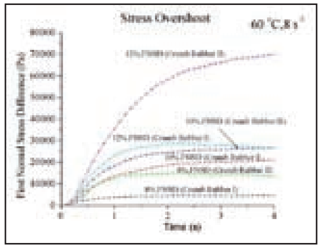

Figure 5-b. *Crumb Rubber – II at 60 s^{-1} for various dosages*

5. Conclusions

Interaction of crumb rubber particles with bitumen is quite complex. A large number of factors related to the crumb rubber particle shape, size, surface area, reaction time, processing temperature and the type of bitumen influence the rheological behaviour of the blend. Selective absorption of oil by rubber particles and their related swelling, increased stiffening of the parent binder and the reduction in mobility of rubber particles as the dosage rate is increased are possibly some of the cause for this complex behaviour. The present investigation reported on capturing the mechanical characteristics of this material when subjected to steady shear. Due to the inherent non-linearity, one sees considerable normal stresses when subjected to steady shear. Some of the important observations related to this study can be summarized below:

- A coarser graded crumb rubber (crumb rubber-I) results in considerable reduction in shear stress and normal stress difference for any dosage rate, shear rate and temperature.

- For a given crumb rubber type and shear rate, as the dosage is increased, the difference between the shear stress and normal stress differences keeps decreasing. However, as the temperature is increased, the onset of normal stress difference is considerably lower signifying the reduction of resistance to mobility among rubber particles.

- A Finer graded crumb rubber (crumb rubber-II) results in considerable higher stress-overshoot in shear stress and normal stress difference for any dosage rate, shear rate, and temperature.

- For a given crumb rubber type and temperature, as the shear rate is increased, the normal stress differences increases at a much higher rate than the shear stress.

The measurement of normal stresses and their difference along with shear stress hold the key to development of robust constitutive models for crumb rubber modified bitumen. The influence of the particle size distribution as well as the dosage rate was clearly captured during the steady shear motion reported here. Efforts are currently underway to quantify the influence of size and shape characteristics of crumb rubber particles and their relationship to the development of normal stresses and shear stresses.

Acknowledgements: J. Murali Krishnan acknowledges the support of Polymer Engineering Laboratory, IIT Madras on the conduct of experiments.

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Chapter 8

Invited papers on related aspects

(As invited papers the presentations in this Chapter have not been subjected to the Technical Committee blind peer review)

Experimental Study on Strength Developing Law of Epoxy Asphalt Mixture during its Curing Reaction

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ABSTRACT. Epoxy asphalt mixture (EAM) is widely used in the pavement engineering recently. As a thermosetting material, its strength developing law is different from thermoplastic paving material such as HMA, which lead to a more complex construction control. This paper presents an experimental program on the strength developing law of EAM. Firstly, the viscosities of epoxy asphalt binder (EAB) in different temperatures were tested and the viscosity-time curve was constructed to investigate its developing law with time and temperature. Then the EAM strength developing law with reserved time and curing time in different temperatures were studied respectively using Marshall test from the view of phenomenology. Finally, some important time and temperature points of EAM construction control were concluded according to the researches and case studies were conducted to verify the effectiveness of the conclusion.

KEYWORDS: epoxy asphalt mixture, strength developing law, viscosity-time curve, construction control

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1. Introduction

Epoxy Asphalt Mixture (EAM), mixed by epoxy asphalt binder (EAB) and high quality aggregate, has been widely used in the pavement engineering for its excellent performance. However, as a thermosetting material, EAM has been practically observed with different performances compared to the common thermoplastic materials as HMA, especially on the strength developing law, which affects the practical construction directly. Therefore, in order to instruct the construction of EAM, researches on the strength developing law in the curing process is needed.

As an irreversibly cure material, thermosetting material can be completed through heating and chemical reactions, by which the curing process transforms the resin into a plastic or rubber by a cross-linking process. Due to the added energy or catalysts, the molecular chains react at chemically active sites and are linked into a rigid, 3-D structure. Theoretically, the material will obtain a higher melting point when compared to the surrounding ambient temperature because of the larger molecular weight caused by the cross-linking process. However, uncontrolled reheating usually causes the material reaching the decomposition temperature before the melting point is obtained. Therefore, a thermosetting material cannot be melted and re-shaped after it is cured (Milton Keynes, 2000).

There have been many researches and studies being conducted to study the strength mechanism of paving materials. The Transportation Materials Research Center (TMRC) of the Michigan Technological University has researched the strength mechanism of the asphalt mixtures using the finite element and discrete element methods (Dai Q, 2007; You Z, 2007; Dai Q, 2006), Ranja Bandyopadhyay (Ranja Bandyopadhyay, 2007) and Martin H. Sadd (M.H. Sadd, 2005) have also investigated the strength formation of asphalt mixtures from microscopic view. However, these researches are all focusing on the thermoplastic material such as HMA; few studies are conducted on the thermosetting material. The Intelligent Transportation System Research Center of the Southeast University has taken a number of researches on EAM in recent years (Huang, 2003; Luo, 2006; Chen, 2006), nevertheless, most of the researches are aiming at the performances of EAM after curing, and further studies on the strength developing law of EAM is still needed.

The former studies show that the performances of EAM are influenced significantly by its reserved time and curing time. The reserved time is defined as the time from the mix of asphalt and aggregate to the compaction of the mixture, an optimum reserved time will lead to the best performance of EAM. Also, the strength developing law of EAM in different temperatures will help to determine the curing time after practical EAM construction. So, in this paper, an experimental program is put on to investigate the strength developing law of EAM. Firstly, a viscosity test is conducted to investigate the viscosity developing law of EAB; secondly, the performance of EAM under different reserved time and curing time in different temperatures is studied to investigate the strength developing law of EAM; finally, the test results are analyzed and case studies are conducted.

2. Materials and methods

2.1. Materials Preparation

Two main materials are involved in the test. The details of preparing for both materials and specimen are described as follows.

2.1.1. Epoxy Asphalt

The binder used in the test is 2910-type local epoxy asphalt, which is composed of two components marked as A and B. Component A is the epoxy resin while component B consists of petroleum asphalt and curing agent. The basic information of the material is given in Table 1.

Table 1. *Technical Index of 2910-Type Local Epoxy Asphalt*

Technical Indexes	Measured Value	Criteria	Test Method
Mass ratio（A：B）	100:290	100:290	
Tensile strength（MPa，23℃）	3.26	≥2.0	ASTM D 638
Fracture elongation（%，23℃）	242	≥200	ASTM D 638
Duration of viscosity up to 1Pa·s（min）	110	≥50	JTJ052-2000

2.1.2 Aggregate

Aggregates take about 93 to 94 percentage of the weight for the total mixture. Therefore, careful considerations should be given to the selection of type and quality of the aggregates. In this study, the basalt aggregate and the limestone powder special for steel bridge pavement are selected based on the practical engineering, and the max aggregate diameter is 13.2mm. The basic information of the aggregate is shown in Table2 and Table 3, the gradation curve for the aggregate is shown in Figure 1.

Table 2. *Technical Index of the Basalt Aggregate*

<table>
<tr><th colspan="2">Technical Indexes</th><th>Measured Value</th><th>Criteria</th><th>Test Method</th></tr>
<tr><td colspan="2">Compressive strength (MPa)</td><td>140</td><td>≥120</td><td>JTG E41-2005(T0221-2005)</td></tr>
<tr><td colspan="2">Los Angeles abrasion value (%)</td><td>11.5</td><td>≤22.0</td><td>JTG E42-2005(T0317-2005)</td></tr>
<tr><td colspan="2">Crushing value (%)</td><td>8.9</td><td>≤12</td><td>JTG E42-2005（T0316-2005)</td></tr>
<tr><td rowspan="6">Apparent density (g/cm³)</td><td>1#</td><td>2.924</td><td rowspan="6">≥2.65</td><td rowspan="6">JTG E42-2005（T0304-2005)</td></tr>
<tr><td>2#</td><td>2.983</td></tr>
<tr><td>3#</td><td>2.912</td></tr>
<tr><td>4#</td><td>2.900</td></tr>
<tr><td>5#</td><td>2.967</td></tr>
<tr><td>6#</td><td>2.803</td></tr>
<tr><td rowspan="2">Water sbsorption (%)</td><td>1#</td><td>0.93</td><td rowspan="2">≤1.5</td><td rowspan="2">JTG E42-2005（T0304-2005)</td></tr>
<tr><td>2#</td><td>1.22</td></tr>
</table>

Table 3. *Technical Index of the Limestone Powder*

Technical Indexes	Measured Value	Criteria	Test Method
Density (g/cm^3)	2.703	≥2.500	JTG E42-2005 (T0352-2005)
Hydrophilic coefficient (%)	0.63	≤1	JTG E42-2005 (T0353-2005)
Plasticity index (%)	3.2	4.0	JTG E42-2005 (T0354-2005)

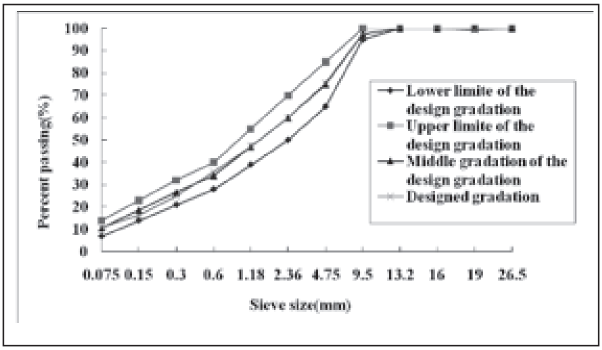


Figure 1. *Designed gradation in the test*

2.2. Specimen Preparation

Considering the convenience of the specimen forming, the Marshall test is adopted to investigate the performance of EAM, Marshall stabilities and air voids are also two main indexes in the practical EAM construction. Based on the Marshall mixture design test, a 6.5% binder content is determined as the optimum asphalt content, the Marshall mixtures are shaped using impact compaction method, five replicates are prepared for each testing.

2.3. Experimental program

Viscosity of binders has closed relationship with performance of the mixtures, however, there are many other clues that affect the performance of the mixture, so the strength developing laws of EAM with reserved time and curing time are also studied by investigating the Marshall stability of the curing specimens.

2.3.1 Viscosity test

110°C, 115°C, 120°C, 125°C and 130°C are selected as test temperatures according to the practical construction temperatures. The viscosity of EAB is measured using a Brookfield rotational dial viscometer with a rotating speed of 100 rpm (29#), then, the viscosity-time curves are constructed to investigate the viscosity developing law in different temperatures.

2.3.2 Study on reserved time

The epoxy asphalt and the aggregates are mixed at 110°C, 115°C, 120°C, 125°C and 130 °C respectively, and the reserved times are selected as 30min, 50min, 70min, 90min and 110min. The stability and air void of each specimen is measured and studied.

2.3.3 Study on curing time

The Marshall mixtures are formed in the same condition at temperature 120°C and reserved time 50min, then curing at the temperatures of 120°C, 125°C and 130°C respectively, the Marshall stability and air void at different curing time are tested. Meanwhile, other Marshall mixtures shaped in the same conditions are curing in the room temperature at about 23°C to simulate the actual condition in EAM construction. The Marshall tests are conducted as well at different curing time to investigate the strength developing law of EAM with the curing time.

3. Results and discussions

3.1. Viscosity test

The viscosity-time curves of EAB in different temperatures are shown in Figure 2.As shown in Figure 2, the viscosity of EAB increased as the reaction time increasing, and the viscosity increase slowly at the beginning of the test, while it increases faster and faster with the time passing. It is also observed that the higher the test temperature is, the faster the viscosity grows. This can be described using the dual-Arrhenius model (Min, 2006; Min2007) as

$$\ln \eta(t,T) = \ln \eta_{\infty} + \frac{E_{\eta}}{RT} + At\exp(-\frac{E_a}{RT}) \tag{1}$$

where

$\eta(t,T)$ = the viscosity,

η_{∞} = the Arrhenius Pre-factor,

E_{η} = the flow activation energy,

Ea = the reaction activation energy,

R = the gas constant,

T = the absolute temperature,

t = the time, and A is the factor.

The Equation (1) presents that the viscosity of EAB is a function of time and temperature, so the time and temperature construction process of EAM should be strictly controlled.

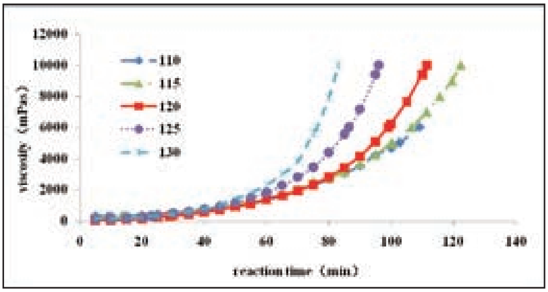
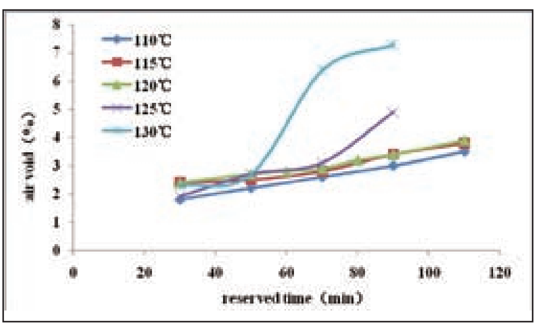


Figure 2. *Viscosity-time curve of EAB*

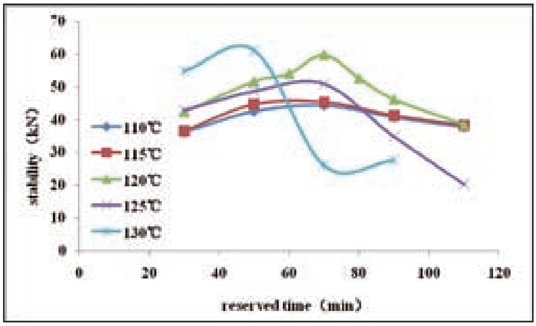
It can be observed in detail that before the viscosity of EAB reaches about 1000mPa·s, the viscosity grows slowly, this goes against the initial strength formation of EAM; after the viscosity of EAB reaches about 3000mPa·s, the viscosity grows rapidly, and this will affect the rolling effect of EAM. So, it can be drown that the viscosity range that suitable to the EAM construction is about 1000mPa·s to 3000mPa·s.

3.2. Study on reserved time

The tests results of EAM stability and air void are shown in Figure 3.The air void and Marshall stability controlled indexes in the practical EMA construction are under 3.0 and over 40kN respectively. It can be observed from Figure 3(a) that the air void of EAM grows with the reserved time increasing, when the test temperature is under 120°C, the air void increasing slowly as the times passing while the air void increasing rapidly when the test temperature is over 120°C, which make the air void difficult to control in rolling process. From Figure 3(b), it can be seen that there is an optimum reserved time corresponding to each temperature, the stability at the temperatures of 120°C and 130°C are higher than others. Taking a comprehensive consideration on the air void and stability, the optimum temperature of EAM construction is 120°C, and the reserved time is no longer than 70min.



(a) Air void – reserved time curve



(b) Stability – reserved time curve

Figure 3. *Experimental results of study on the reserved time*

A further study on reserved time of EAM is conducted and the reserved times corresponding to every temperature from 110°C to 130°C are concluded as shown in Table 4. The data in Table 4 indicates that there is a decrescent trend of reserved time as the temperatures increasing and the shortest and the longest reserved time are both decreasing. It's also can be seen that while the temperatures rising, the reserved time range of EAM becomes narrow, there is 48min for EAM construction at 110°C while it decreases to 25min when the temperature is 130°C. According to the comprehensive considering of the laboratorial results and the need of practical construction, it is better to construct EAM at the temperature from 115°C to 120°C.

Table 4. *The reserved time range of different temperatures*

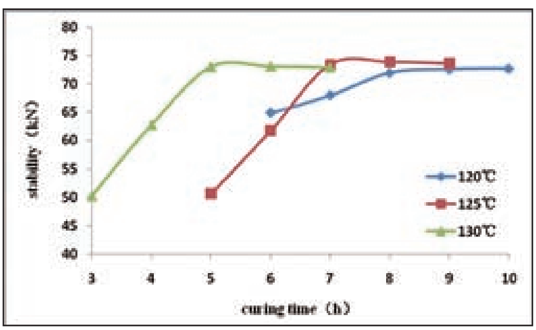
Temperature (℃)	Shortest Reserved Time(min)	Longest Reserved Time(min)	Reserved Time Range (min)
110	42	90	48
111	42	87	45
112	41	84	43
113	40	82	42
114	39	79	40
115	38	76	38
116	37	75	38
117	35	74	39
118	33	72	39
119	32	71	39
120	30	70	40
121	30	69	39

Temperature (℃)	Shortest Reserved Time(min)	Longest Reserved Time(min)	Reserved Time Range (min)
122	30	68	38
123	30	67	37
124	30	66	36
125	30	65	35
126	30	63	33
127	30	61	31
128	30	59	29
129	30	57	27
130	30	55	25

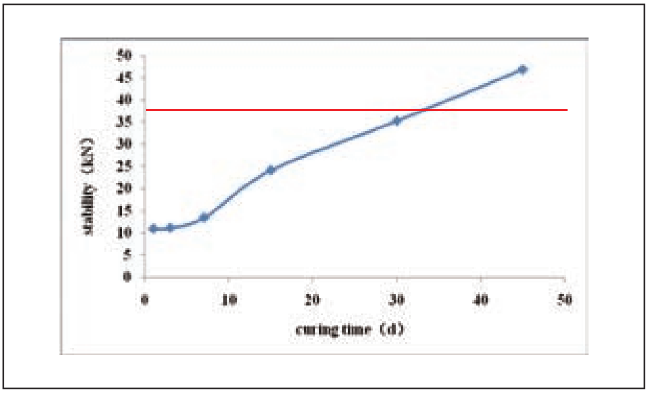
3.3. Study on curing time

The tests results of EAM stability and air void in different curing times are shown in Figure 4. Figure 4(a) presents the strength developing law of EAM in laboratorial curing condition, it can be seen that the stability grows as the curing time increasing until it achieves the maximum value, which means the curing reaction is finish. It also can be seen that curing time is shorter when the curing temperature is higher, 5 hours at 130°C is the optimum laboratorial curing condition.

Figure 4(b) shows the strength developing law of EAM at room temperature. It can be observed that curing process in this condition is much slower than in laboratorial condition; under this circumstance, the EAM need about 40 days to reach the stability of 40kN.



(a) EAM stability in the laboratorial curing conditions



(b) EAM stability at the room temperature

Figure 4. Experimental results of study on the curing time

4. Case study

4.1. Tianxingzhou Highway-railway Yangtze River Bridge

The Tianxingzhou Yangtze River Bridge is taken as a case to verify the key points of EAM construction control which are concluded through the experimental study. The Tianxingzhou Bridge is the first cable-stayed highway-railway bridge that contains 4 railway lines in the world, the total length of the bridge is 766m with a main span of 504m. For the excellent performance, EAM is selected as paving material of Tianxingzhou steel deck paving engineering which is started in November 2008.

According to the results of laboratorial study, the construction temperature of EMA is determined as 115°C to 120°C, and the reserved time range in Table 4 is abided. The core-drilling test was conducted after curing for three days, the test results show that the average air void of the core-drilling specimen is 2.0, less than the control standard 3.0, it proves that 115°C to 120°C is a suitable condition for EAM construction.

For the construction season is later autumn, the average temperature during the construction process is about 10°C, lower than the experimental temperature. So Marshall specimens were formed and cured in the natural environment, the Marshall test is conducted to monitor the strength develop of the EAM, the test result is presented in Figure5. It can be observed from Figure 5 that the strength of EAM grows more slowly than the experimental study for the low temperature, it takes over 100 days that when the stability reaches 40kN, and the stability reaches maximum value after curing for about 140 days.

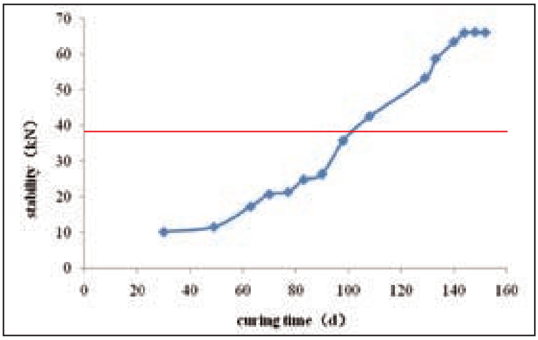


Figure 5. *Strength developing law of EAM in Tianxingzhou Bridge*

4.2. Shanghai Yangtze River Bridge

The Shanghai Yangtze River Bridge is taken as another case. The Shanghai Yangtze River Bridge is a 1430m cable-stayed bridge with a main span of 730m, which is the main bridge of crossing-river channels in Chongming of Shanghai. EAM is selected to be used in the steel deck paving project, which is the first time of EMA use in paving project of Shanghai. The project is started in June 2009, a hot summer in China.

As in the steel deck paving project of Tianxingzhou Bridge, the construction temperature of EMA is determined as 115°C to 120°C, and the reserved time range in Table 4 is abided also. The core-drilling test results show that the average air void of the core-drilling specimen is 1.8, less than the control standard 3.0, it can be concluded again that 115°C to 120°C is suitable for EAM construction.

During the construction of the EAM, Marshall specimens were formed and cured in the natural environment. The average temperature during the construction term is about 25°C, about 15°C higher than that of Tianxingzhou Bridge. The EAM Marshall test results of Shanghai Bridge are shown in Figure6. It can be observed from Figure 5 that the strength of EAM grows more quickly than that of Tianxingzhou Bridge, the stability of EAM reaches 40kN after only 18 days curing, about 90 days earlier than that in Tianxingzhou Bridge. It indicates that the curing time is affected a lot by temperature and it is better to construct EAM in hot summer.

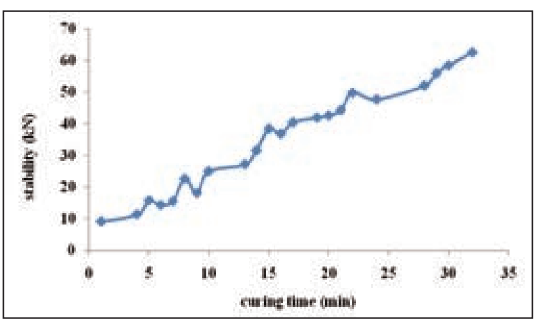


Figure 6. *Strength developing law of EAM in Shanghai Yangtze River Bridge*

5. Conclusion

This paper presents an experimental program to study the strength developing law of EAM, and conclusions can be drawn as follows:

1. The viscosity of EAB is a function of time and temperature, it increases with the reaction time passing, and it grows quicker when the reaction temperature is higher. It would be better to roll EAM when the viscosity of EAB is in the range of 1000mPa·s to 3000mPa·s.

2. The air void grows with the revered time increasing, and the optimum condition for EAM construction is 120°C, 70min according to the laboratorial study, which is also verified in the steel deck paving engineering project of Tianxingzhou Highway-railway Yangtze river Bridge and Shanghai Yangtze river Bridge.

3. The strength of EAM grows with the curing time going, and it reaches the maximum value more rapidly in a higher temperature.130°C, 5h is suitable for the completely curing of EAM in the laboratory.

4. The Phenomenological method is adopted to determine the curing time needed in the natural curing condition, the results show that it takes only about 18 days curing when the stability of EAM reaches 40kN in summer while the curing time is over 100days in later autumn. So it is better to construct EAM in hot summer.

Acknowledgment

The authors would like to thank the financial support to this research from the Chinese National Natural Science Foundation (No. 50878053) and Chinese New Century Excellent Talents Program of Education Ministry.

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Asphalt Rubber: policy disclosure in Italy

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ABSTRACT. In the present paper, it is presented a proposal for an evaluation systemic approach aimed at the characterization of Asphalt Rubber Hot Mixes (ARHM). This constituted the input to develop a careful scientific-technical assessment process to assure the quality control of ARHM. In particular, this kind of approach allows i) to take immediate action to technical production and laying of ARHM that should also guarantee "traditional " Italian road materials specifications ii) to develop AR expertise in terms of mechanical performance (stiffness, fatigue resistance and permanent deformation resistance) and functional properties (noise and skid resistance), trying to bridge the gap of knowledge with the international experience of forty years of successful use iii) to develop innovative scientific research to provide original contributions to engineering road materials investigation.

KEYWORDS:Evaluation systemic approach, Asphalt Rubber Hot Mixes, Wet process

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1. Introduction

Asphalt Rubber (AR) is a mixture of hot asphalt and crumb rubber derived from waste scrap tires (ASTM D-6114). This type of blending process is called “wet”. It was invented in the United States in the State of Arizona in the late 1960’s. AR is commonly used as binder for wearing course hot mix asphalts in order to improve smoothness and skid resistance and to reduce cracking and traffic noise. While in many countries all over the world it is used extensively and successfully in the highway paving industry, in Italy AR technology is quite new and first applications were made in 2006 (Santagata *et al*., 2008).

The study presented in this paper came about as a result of the ever-growing demand for high performances in the field of road constructions, which lead investigation into asphalt binder properties, surpassing the current traditional empiricist approach in favor of a more specific characterization. Particularly, the interest in developing this scientific investigation is aimed at driving the worth of industrial utilization of asphalt binders over traditional asphalts, and systemizing its classification, based on the most innovative specifications. Adding to all this, there is also an environmental concern linked to the possibilities of re-using a significant amount of reclaimed tires given by Asphalt Rubber Hot Mixes (ARHM).

This paper focuses on an evaluation systemic approach aimed at the characterization of ARHM, describing the first steps adopted to have Asphalt Rubber “made in” Italy: a global strategy which strongly involved the, at the moment, unique Italian producer of Asphalt Rubber binder *(Asphalt Rubber Italia)*, CIRS *(Experimental Interuniversity Road and Airport Research Centre)* representing the most important scientific entities in Italy and some local agencies.

2. Systemic approach

2.1. Formulation of Asphalt Rubber binder

Experience has shown that by properly combining the waste product of ground tire rubber (Crumb Rubber Modifier - CRM) with asphalt at high temperatures the resultant Asphalt Rubber binder will have many improved superior engineering properties. Such improved engineering properties include reduced fatigue and reflection cracking, greater resistance to rutting, improved aging and oxidation resistance and better chip retention due to thicker binder films (Partl *et al*., 2009; Santagata *et al*., 2007; Souza *et al*., 2005; Zborowski *et al*., 2004; Potgieter *et al*., 2002; Kaloush *et al*., 2003; Cook *et al*., 2006; Kumar *et al*., 2005; Bertollo *et al*., 2004). Also Asphalt Rubber pavements have demonstrated to have lower maintenance costs (Way, 2000; Jung *et al*., 2002), lower noise generation (Pasquini, 2009; Antunes *et al*., 2006a; Leung *et al*., 2006), higher skid resistance and better night-time visibility due to contrast in the pavement and stripping (Antunes *et al*., 2005).

Asphalt Rubber binders have specific and unique characteristics. Several studies have underlined that the enhanced performance depends on the higher percentage of asphalt rubber within the hot mix and on the percentage of crumb rubber used in the asphalt rubber binder prepared by the wet process (Way, 2003).

Conventional binder tests have shown how the CRM modified binders are extremely dependent on the processing conditions, particularly to what concerns the temperature and time of reaction. The time required to disperse, blend and react the crumb rubber is dependent on a number of factors including the chemistry of the asphalt cement and crumb rubber as well as the temperature of the blended material. The same tests put in evidence as the content of CRM is determinant in binder's consistency and elastic behavior (Antunes *et al*., 2004; Antunes *et al*., 2006b).

The main effort to manage the production of AR in Italy consisted in adapting the consolidated AR technology knowledge from abroad to the Italian reality (raw materials, weather, environment, specifications, etc), starting by mixing several bitumen-base (virgin asphalt) with several type and size of crumb rubber, before finding the "optimal" blend to be tested in an industrial scale.

In laboratory, several virgin asphalt (penetration classes: 40/50, 50/70 and 70/100) and high percentages of crumb rubber (higher than 15%) were blended into a homogeneous asphalt-rubber system at a temperature of 190°C, which then reacted at 180°C for a minimum of 45 minutes. Then, the blend characteristics were compared with the specification required by ASTM D-6114 which defines Asphalt Rubber binder (Table 1).

Table 1. *Requirements for Asphalt Rubber binder design (ASTM D 6114)*

Tests	Requirements	
Viscosity Brookfield @175ºC: cP (ASTM D2196)	Min. 1500	Max 5000
Penetration @25ºC: 1/10 mm (ASTM D5)	Min. 25	Max 75
Resilience @25ºC: % (ASTM D 5329)	Min. 20	
Softening Point, Ring&Ball: ºC (ASTM D36)	Min. 54.5	

These limits along with the results attained for each percentage of crumb rubber blended into the bitumen allow the determination of the "optimal" content of crumb rubber, which is usually the percentage that improves the softening point temperature, resiliency and ductility.

The results show that increasing the CRM percentage corresponds to increasing the softening point, resilience and viscosity values as depicted in Figures 1 and 2.

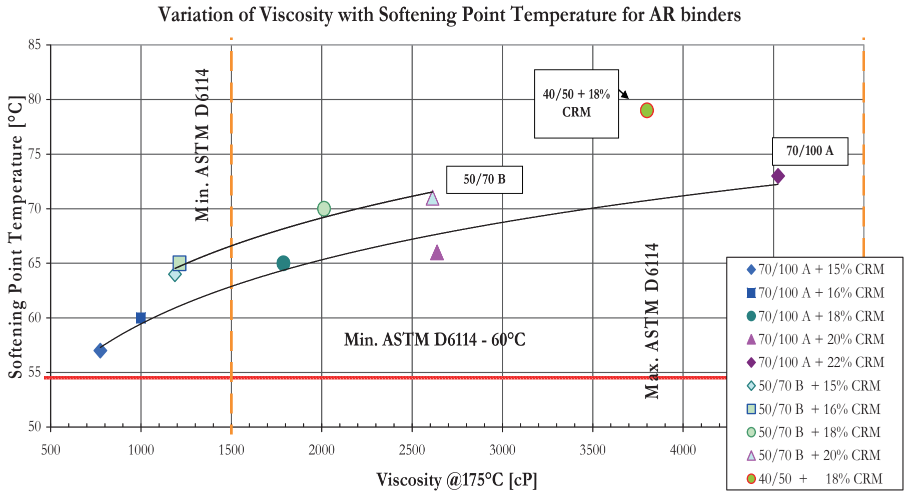


Figure 1. *Variation of viscosity with the softening point for Asphalt Rubber with different percentage of CRM*

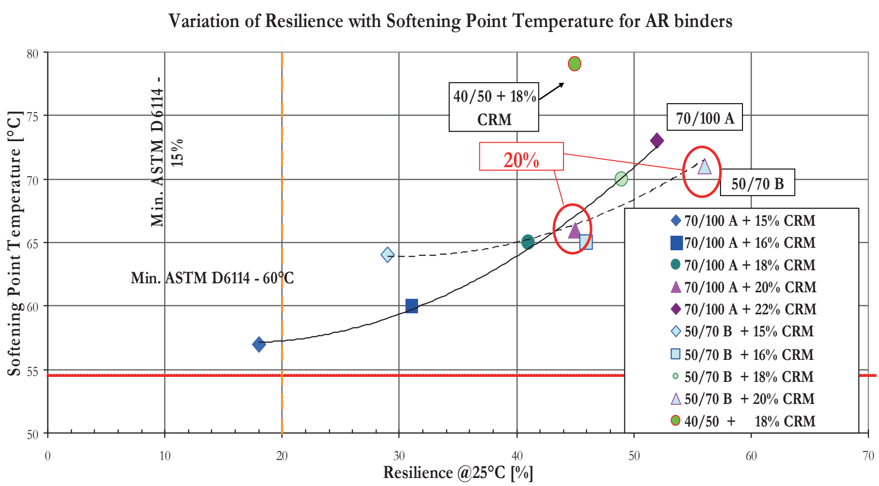


Figure 2. *Variation of resilience with softening point for Asphalt Rubber with different percentage of CRM*

It can be seen that some asphalts are more compatible with CRM than others and thus binder design pre-testing is necessary to determine that the proper asphalt grade is being used to formulate the asphalt rubber.

Taking in consideration the results above and the Italian weather and ambient contest, three blends are now commonly produced at the AR industrial producing plant: 40/50 with 18% CRM, 50/70 with 20% CRM and 70/100 with 20% CRM, depending on the use.

2.2. Characterization of Asphalt Rubber mixes

Following the Arizona Department Of Transportation (ADOT) specification, in Italy, the AR producer began experimenting with two asphalt rubber mixes: an open graded mixture (ARFC) and a gap graded mixture (ARAC) (Scofield, 1989). To fully utilize AR properties, two aggregate gradations, that would provide a high voids in the mineral aggregate (VMA), have been adopted as shown in Table 2.

Table 2. *Italian ARFC (Open Graded) and ARAC (Gap Graded) Gradations*

AR MIX	**Open Graded ARFC**		**Gap Graded ARAC**	
Sieves (mm)	**Min**	**Max**	**Min**	**Max**
16	100	100	100	100
12.5	93	100	83	97
10	88	100	68	82
8	68	82	54	68
4	23	37	25	37
2	3	15	12	24
0.5	2	10	7	15
0.063	0	3	0	3
AR Binder	8.5%	9.5%	7.5%	8.5%

To what concerns the study of ARHM, the adopted approach consisted in developing theoretical and experimental parallel activities, with the main purpose of:

- to disclose experimental findings to encourage the use of ARHM in the Italian road network in the awareness of potential technical, environmental and economic benefits;

- to develop innovative and relevant topics to provide original contributions to scientific research.

In particular, it was provided technical support during the production of asphalt rubber mixes to assure that ARHM respect Italian “traditional” standard specifications. This support was divided in time by monitoring the characteristics of several materials produced and implemented for many applications on the road in recent years. The intent is to gather as much information as possible to set up a large database, which allows the establishment of real performance-type specifications.

At the same time, advanced research on AR materials through innovative test methods have been developed.

3. Operative support for road applications

In this chapter the technical activities conducted within the routine characterization of asphalt rubber mixes are illustrated. This activity is geared to provide immediate answers to ARHM producers and potential users of these innovative materials in the Italian market. In that sense several productions of ARHM were analyzed in terms of composition (aggregate gradation and AR binder content), volumetric properties (void content) and mechanical performance (Marshall Stability, indirect tensile strength, loss of strength after immersion in water).

As an example, some results obtained during the characterization of AR gap graded – ARAC (Figures 3, 4, 5, and 6) and AR open graded – ARFC (Figures 7, 8, 9, 10) are shown.

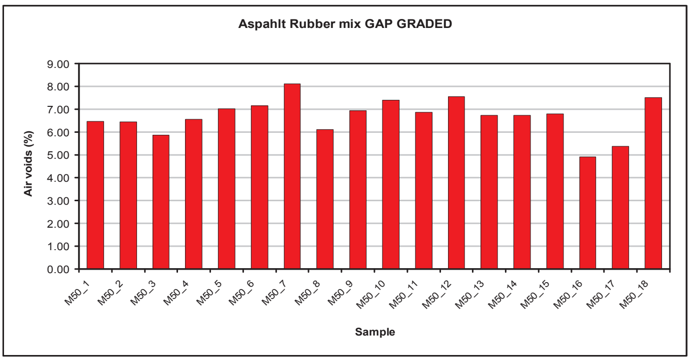


Figure 3. *Volumetric properties of a Gap Graded AR mix*

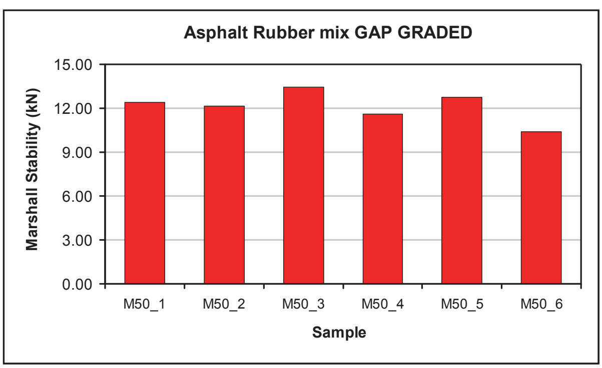


Figure 4. *Marshall Stability of a Gap Graded AR mix*

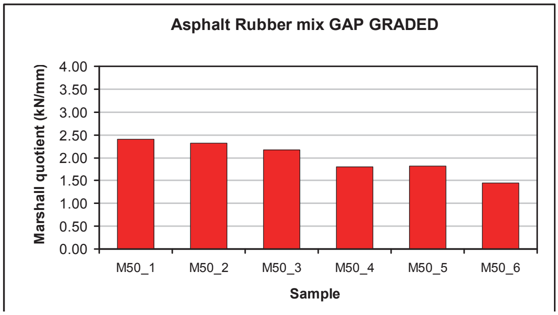


Figure 5. *Marshall Quotient of a Gap Graded AR mix*

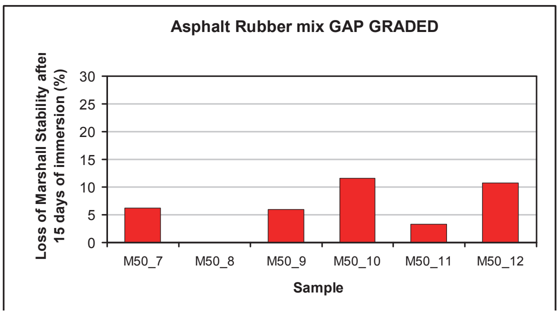


Figure 6. *Loss of Marshall Stability of a Gap Graded AR mix*

Obviously, the results presented refer only to two specific materials, gap and open graded, respectively, packaged with a particular compaction methodology and compaction energy. In that sense, this results represent only a model of the type of data obtained during this phase of the study. In particular, it is possible to notice the good repeatability of results for each tested material. These studies were repeated systematically over the years on the production in order to verify also the quality of the process.

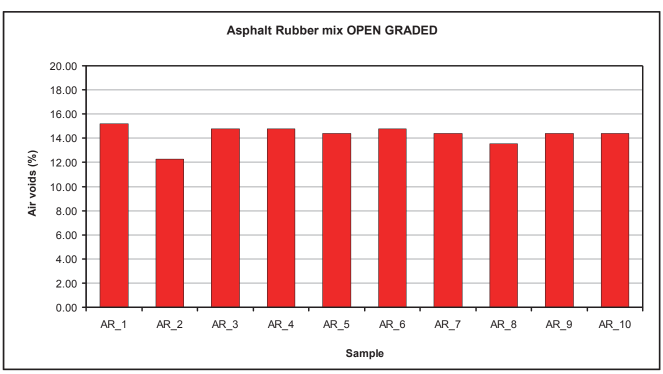


Figure 7. *Volumetric properties of an Open Graded AR mix*

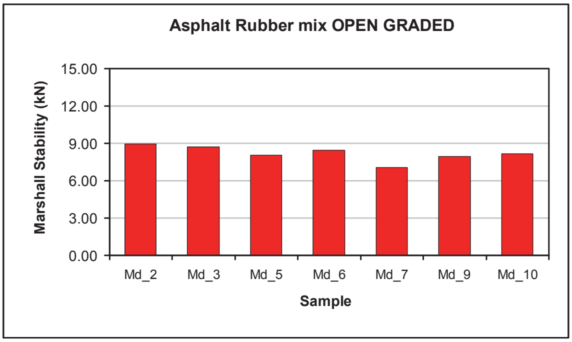


Figure 8. *Marshall Stability of an Open Graded AR mix*

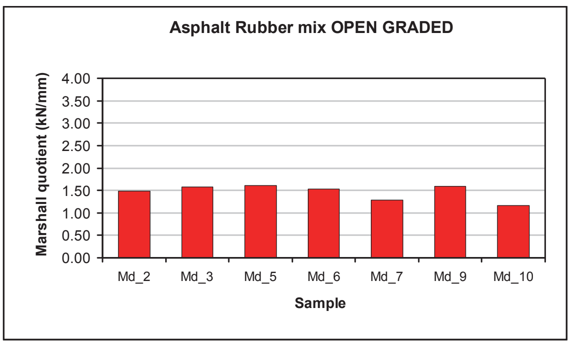


Figure 9. *Marshall Quotient of an Open Graded AR mix*

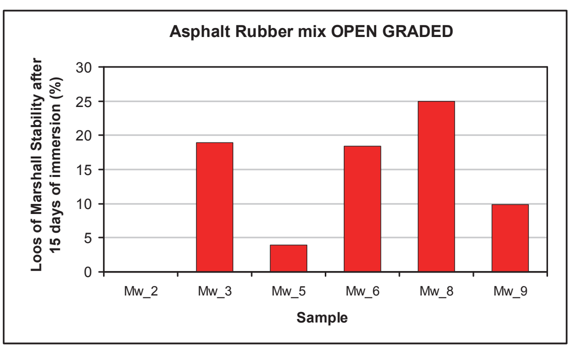


Figure 10. *Loss of Marshall Stability of an Open Graded AR mix*

The detailed and rigorous monitoring of these requirements allowed making a continuous optimization of ARHM and also AR binders. In addition, this extensive data enable to make a preliminary draft of a specification for ARHM following both Italian standard specification for “traditional” road materials and international AR requirements. Table 3 is an example of adaptation of different standards of specifications to develop AR gap graded (ARAC) mixture optimized taking into account the specificities of the Italian reality.

Italian specifications, even those more advanced, follow the current traditional empiricist approach and only provide a marginal concept of performance of a material (such as stiffness, fatigue and permanent deformation). This fact is mainly due to the lack of a database that has a significance level and statistical feedback in terms of real performance so it is not always possible to prescribe certain standards. Producers are not familiar with non empirical tests and it is often the same thing to for Agencies. The introduction of ARHM in the Italian market opened a new scientific approach to road materials and, with the technical and scientific support of CIRS, the first Standard Specifications for Italian ARAC and ARFC were made in 2007. These specifications are constantly changing since the monitoring of AR materials in laboratory and under real traffic flows is still in progress.

Table 3. *Italian Traditional requirements for ARAC (Gap-Graded)*

Test condition	**Test method**	**Unit**	**Requirements**	
			Binder course	**Wearing course**
Compaction	EN 12697-34	n° of blows	50	50
Requirements				
Marshall Stability	EN 12697-34	kN	> 9	> 9
Marshall quotient	EN 12697-34	kN/mm	1,5÷3,0	1,5÷3,0
Air voids	EN 12697-8	%	5 – 8	5 – 8
Loss of Marshall Stability after 15 days of immersion	CNR n. 149/92	%	≤25	≤25

Nowadays, it is considered that for an objective evaluation of the performance of road materials, specifications should refer to performance-related test methods that can effectively evaluate the real potential of the material. In the near future, therefore, the aim is that these rules begin to be understood as a requirement and replace those that are traditionally and empirically used. In this sense, it is extremely important to introduce the concept of “performance” of a material, which includes its mechanical and functional characterization.

In this light, the proposed specifications require to evaluate certain performance parameters of the ARHM taken from the site and compacted in the laboratory until reaching the voids produced in situ. These parameters must be representative of the visco-elastic properties and the cracking resistance of the mixes, according to EN 12697 series, in order to provide useful numerical elements for the design and reference values for non-destructive in situ testing.

4. Advanced performance characterization

As already introduced, there are many studies in the international literature focused on the mechanical performance (stiffness, fatigue and rutting) and functional properties (noise and friction) of ARHM. In order to bridge the gap of knowledge with the international experience, the same kind of approach was applied in Italy, and the performance achieved by these kind of materials was assessed experimentally through standardized testing protocols. This choice is considered appropriate in consideration of local particularities mainly related to the different characteristics of raw materials (bitumen and aggregates).

The laboratory and in situ collected data for AR mixes and the comparison with more traditional materials will allow the drawing up performance-related specifications with obvious impact to the quality of road infrastructure.

The advanced characterization concerns two main aspects: mechanical and functional properties.

4.1. Mechanical properties

The mechanical behavior of AR materials have been analyzed by means of the evaluation of three main properties: stiffness modulus, permanent deformation and fatigue behavior. Several productions of ARHM have been analyzed from this point of view and the study is still ongoing. As an example, the Authors show the results obtained during the performance characterization of some ARHM.

Materials under study were first characterized in terms of stiffness performance. In Figure 11 it is possible to observe the results for some of the investigated materials. These results were determined by means of indirect tensile configuration tests, according to European EN 12697-24, at 20 ℃.

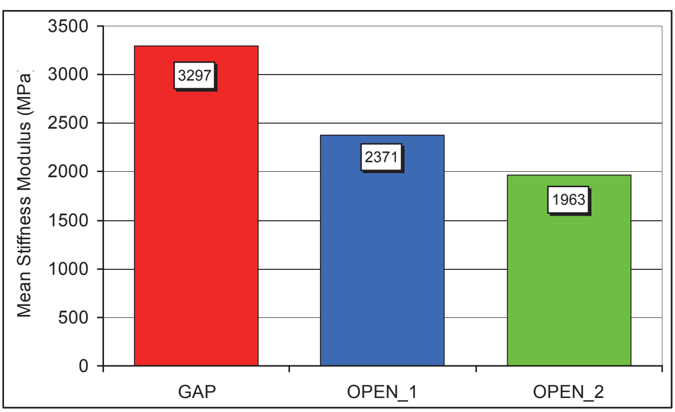


Figure 11. *Indirect Tensile Stiffness Modulus of several AR mixtures @ 20°C*

It should be noted that the ARAC (Gap Graded) mix is characterized by an air void content of about 6.5% while ARFC (Open Graded) mix was optimized in order to reach values between 15% and 20%. Bearing in mind also the great amount of binder that is used in ARHM, these mixes are characterized by a significantly high VMA (Voids in Mineral Aggregate) that further penalize the stiffness properties of ARHM. In this sense, the properties of the AR binder shall ensure that the entire ARHM can still have good rheological performance.

In particular, ARFC (open) shows obviously lower stiffness due to the higher percentage of voids. However, stiffness modulus values of Open Graded ARHM are always greater than those observed for conventional porous asphalts (Canestrari *et al.*, 2007), despite the higher binder content and VMA. This aspect confirms the relevant rheological properties of asphalt rubber binder.

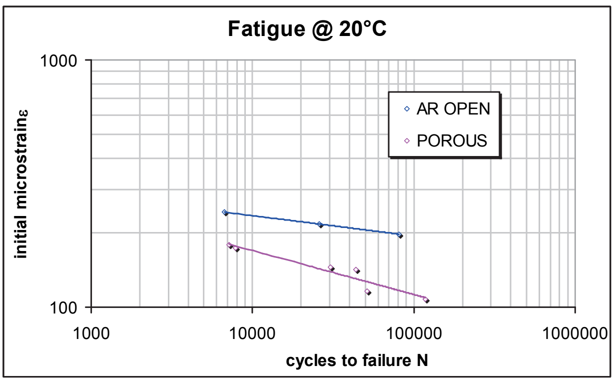


Figure 12. *Fatigue behavior of an ARFC Open Graded AR mixture*

For each tested material, the fatigue behavior was also analyzed on the basis of dynamic tests in indirect tensile configuration according to EN 12697-24. As an example, Figure 12 represents the fatigue curve of the ARFC (OPEN_1) previously introduced. The results are compared with those obtained in the case of a conventional porous hot mix (void content = 20%) prepared with SBS modified bituminous binder.

Even though porous layers provide a limited contribution to the fatigue resistance of the whole pavement, the results presented above clear indicate that the mix made with Asphalt Rubber binder offers significantly better performance than a material not prepared with AR binder and having similar volumetric properties. This fact seems to confirm the remarkable properties of asphalt rubber binder.

The advanced systemic approach described in this chapter finally provides the evaluation of rutting resistance. This property has been assessed by rutting tests performed according to BS 598:110 at 60 ℃. This evaluation is essential because asphalt rubber mixtures, having high binder contents, are potentially more prone to permanent deformation-related distresses. The assessments carried out on different materials have had a huge success: even in comparison with referenced materials with strong rutting resistance (Canestrari *et al*., 2007, Santagata *et al*., 2007), ARHMs have showed very high rutting resistance. Indeed, the deformation obtained after several hours test duration was found to be negligible (less than 1 mm), putting in evidence, once again, the valuable rheological properties that AR binder confers to the ARHM (Figure 13).

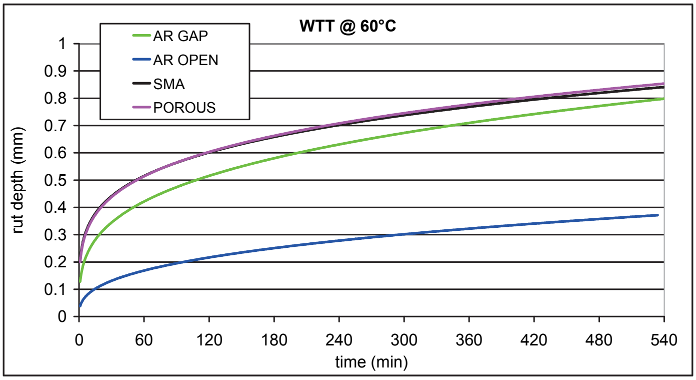


Figure 13. *Rutting behavior of AR mixtures*

4.2. Functional properties

In the evaluation of performance of AR mixtures, it is essential to monitor the also functional properties (friction and noise) by paying special attention to the evaluation of rolling noise reduction. This problem is nowadays an increasingly strong issue that often affects the choice of materials for road pavement wearing courses.

The evaluation of functional properties have been done both in situ, through the measurement of surface characteristics and the evaluation of noise levels along the side of the roadway, and in laboratory, through the determination of sound-absorption properties of materials studied.

As an example, it is interesting to show the results obtained during the acoustic characterization of ARHM, both gap and open graded, in two different trial sections.

4.2.1. Trial sections

A Gap Graded and an Open Graded AR mixes were laid down on an urban trial section about 700 m long in Florence. The thickness of both wearing courses was 30 mm. The section covered with ARAC (gap graded) is composed of two lanes (one for ordinary traffic and one reserved for buses) while ARFC (open graded) trial section has only one lane.

Another trial section realized with a Gap Graded AR wearing course was constructed on an urban way about 1000 m long in Imola. Asphalt Rubber material was staggered with some short sections of a traditional HMA wearing course selected as reference surface for traffic noise survey. The thickness of the new ARAC wearing course was 30 mm.

4.2.2. In situ acoustic analysis

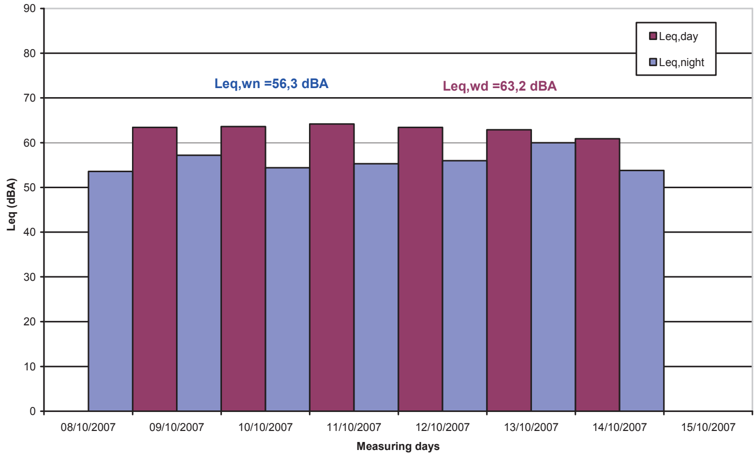
Noise levels recorded along the side of sections constructed with the studied materials were related with noise levels recorded at the same time on near different sections covered with traditional asphalt concretes and interested by quite the same traffic load.

The acoustic characterization of materials under study was carried out performing noise measurements surveys, each lasting one week, according to the Italian technical specifications. Measurement boxes, according to Class 1 of EN 60651 and EN 60804, were fixed at about 4 m high to lampposts situated along the side of the experimental roadway (Figure 14).

Measurement boxes were able to determine the A-weighted equivalent sound level Leq for each measuring hour and the results were summarized in one mean A-weighted sound pressure level for night (Leq,wn) and day (Leq,wd) periods, as shown in the example of Figure 15.

In order to make the comparative study of the “in situ” acoustic measurement objective, it was supported by traffic investigation. Each investigation was carried out for 24 hours once a week, during the traffic noise finding weeks, recording the number of vehicles and the corresponding speed and length.

Figure 14. *Measurements boxes*



In situ noise measurements (one week)

Figure 15. *In situ noise measurements (one week)*

Observing the experimental results obtained for the trial section of Florence (Table 4), it could seem that noise level recorded for the reference material was not comparable with that of asphalt rubber mixtures because of the not negligible difference of traffic flow and speed between the different sections. But actually, the measurement box corresponding to reference material was placed at a greater distance from traffic stream than those mounted along the asphalt rubber experimental sections in such a way that the distance counterbalanced the different traffic conditions. In fact, it is possible to estimate through, for example, the Italian CNR prediction model (Canale *et al.*, 1986) that the greater noise level recordable for the reference material due to the higher traffic flow and speed is roughly counterbalanced by the greater distance between measurement box and traffic stream.

Table 4. *"In situ" acoustic results – Florence trial section*

	ARAC (gap)	**ARFC (open)**	**Reference**
Leq,wd (dBA)	65.1	63.2	67.9
Leq,wn (dBA)	57.9	56.3	61.5
Traffic (vehicles/day)	6694	5656	8967
Mean speed, day (km/h)	37.7	40.8	48.3
Mean speed, night (km/h)	42.0	44.9	58.3
Heavy vehicles (%)	10.16	2.05	3.51

Thus, ARAC mix proved to be about 3 dB(A) quieter than a traditional dense graded asphalt concrete principally thanks to the asphalt rubber binder employed. It has to keep in mind that 3 dB(A) noise reduction corresponds to halving traffic flow or doubling the distance between the source and the receiver.

A further 2 dB(A) noise reduction was demonstrated to be achieved through the employment of open graded AR mixture that is able to combine acoustic benefits arising from AR binder with those obtainable from high air void content in terms of sound absorption capabilities. Moreover, this mix is characterized also by a reduced maximum chipping size that further enhanced rolling noise reduction properties.

Results obtained from in situ acoustic and traffic investigations for the trial section of Imola are presented in Table 5.

Table 5. *"In situ" acoustic results – Imola trial section*

	ARAC (gap)	**Reference**
Leq,wd (dBA)	66.5	72.7
Leq,wn (dBA)	60.4	66.4
Traffic (vehicles/day)	9305	12396
Mean speed, day (km/h)	74.3	74.2
Mean speed, night (km/h)	77.5	77.3
Heavy vehicles (%)	5.52	5.53

Differently to what happened for the previous case, the noise pressure levels recorded were not directly comparable. In fact, the two sections were characterized by the same mean vehicle speed and heavy vehicle content but different total traffic flow, 25% lower for ARAC material owing to an intermediate intersection.

Moreover, the sound measurement box corresponding to ARAC section had to be placed at a distance from traffic stream (Figure 16 on the right) sensibly greater than that of reference material road section (Figure 16 on the left).

Thus, in the same way as the previous situation, it is possible to estimate that the difference between noise level in ARAC section and in reference material section has to be reduced of about 3 dB(A) considering these two aspects. Taking into account the previous considerations, ARAC proved again to be about 3 dB(A) quieter than a traditional dense graded asphalt concrete having quite the same void content and maximum chipping size. As a consequence, this noise reduction, corresponding to halving the traffic flow, has to be imputed to the use of the asphalt rubber binder that confers more elastic properties to the bituminous mixture.

Figure 16. *Measurement boxes position – Imola trial section*

4.2.3. Laboratory acoustic results

The acoustic laboratory experimental program consisted in the determination of the sound absorption coefficient α of the investigated materials by means of the impedance tube (Figure 17) according to the EN ISO 10534-1.

Samples subjected to sound absorption investigation were 95 mm diameter cylindrical specimens cored from slabs prepared in laboratory with the Roller Compactor (EN 12607-33). Slabs were prepared with materials taken during the construction of the respective experimental road sections. The thickness and air void content of the slabs were preferably chosen equal to those obtained in situ.

Each sample was subjected to 7 different test frequencies: 400, 500, 630, 800, 1000, 1250 and 1600 Hz.

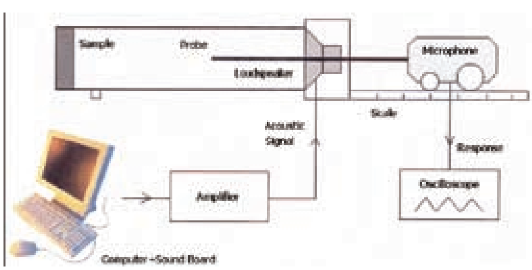


Figure 17. *Sound absorption test setup*

The Open Graded mixture showed a not pronounced sound absorption coefficient notwithstanding the 14% air void content (Figure 18). This fact seems to confirm that a void content lower than 15% is not able to guarantee good absorption properties because the pores are probably not totally interconnected between them. Moreover, the selected reduced maximum chipping size probably enhanced the air flow resistance of the material limiting sound absorption characteristics.

SOUND ABSORPTION COEFFICIENT α							
Specimen	Frequency (Hz)						
	400	500	630	800	1000	1250	1600
OG-ARsitu_1	0.17	0.29	0.38	0.38	0.23	0.10	0.07
OG-ARsitu_2	0.10	0.18	0.34	0.31	0.25	0.05	0.05
OG-ARsitu_3	0.19	0.20	0.35	0.32	0.14	0.03	0.04
OG-ARsitu_4	0.11	0.23	0.45	0.31	0.15	0.03	0.03
OG-ARsitu_5	0.16	0.25	0.34	0.24	0.09	0.05	0.06
mean values	**0.15**	**0.23**	**0.37**	**0.31**	**0.17**	**0.05**	**0.05**

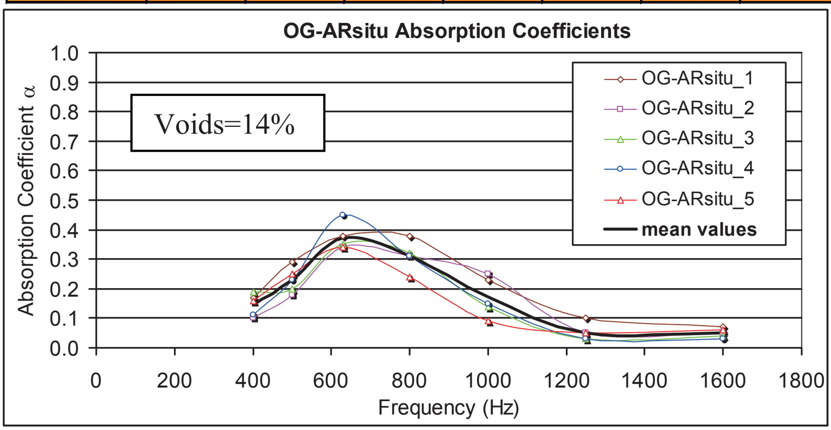


Figure 18. *Absorption coefficients of Open Graded AR mix*

Furthermore, it is very interesting to note that ARFC – Open Graded bituminous mixtures demonstrated a low peak frequency of absorption corresponding to 630 Hz (Figure 18). According to (Sandberg *et al*., 2002), this may be due to the higher tortuosity, i.e. a pores-shape parameter, that arise from the reduced maximum chipping size coupled with to the high binder content that created narrow channels linking up pores.

As a matter of fact, the not elevated sound absorption coefficients recorded for the ARFC material indirectly proved that the very good anti-noise performance demonstrated in situ by this mixture is principally due to the acoustic properties, in terms of reduction of rolling noise generation, arising from the adding of asphalt rubber binder.

Finally, low absorption coefficients were showed by the Gap Graded AR mixture (Figure 19). This fact proved once again that the asphalt rubber binder is the main author of the acoustic benefits demonstrated by this kind of mixture .

SOUND ABSORPTION COEFFICIENT α							
Specimen	Frequency (Hz)						
	400	500	630	800	1000	1250	1600
GG-AR_1	0.11	0.14	0.14	0.25	0.10	0.04	0.03
GG-AR_2	0.13	0.18	0.12	0.29	0.02	0.07	0.09
GG-AR_3	0.10	0.15	0.18	0.20	0.11	0.01	0.04
GG-AR_4	0.08	0.12	0.18	0.29	0.08	0.05	0.04
GG-AR_5	0.12	0.18	0.20	0.20	0.06	0.05	0.02
mean values	**0.11**	**0.15**	**0.16**	**0.25**	**0.07**	**0.04**	**0.04**

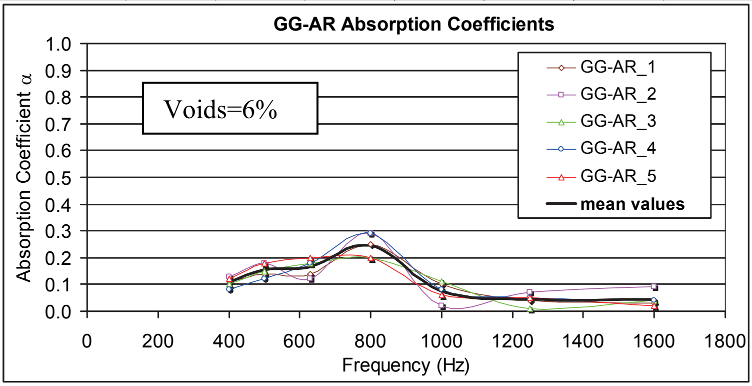


Figure 19. *Absorption coefficients of Gap Graded AR mix*

5. Innovative researches

The characterization of a road material should be completed with the development of advanced studies that will provide original contributions to scientific research. In this regard, several research items have been developed in the recent years, aimed at the evaluation of specific performance-related properties through innovative test methods.

As an example, it is possible to mention studies on the durability of Asphalt Rubber mixes with particular interest to water damage. This aspect is being investigated through two innovative testing methods (Pasquini, 2009, Partl *et al*., 2009, Santagata *et al*., 2009): the CAST, implemented in the EMPA laboratories; and the PATTI test, specifically modified for the determination of the adhesion and cohesion properties of bitumen-aggregate system.

In particular, CAST equipment allows testing of laterally deformation constrained donut shaped specimens under simultaneous action of cyclic mechanical loading, temperature changes and water exposure. Tests are performed in a conventional, temperature controlled, servo-hydraulic tension-compression machine. (Figure 20).

As an example, Figure 21 shows a typical output of a strain-controlled fatigue test performed at 10 Hz and with temperature cycles. The complex modulus E* and phase angle of the material are calculated based on a finite element model of the test.

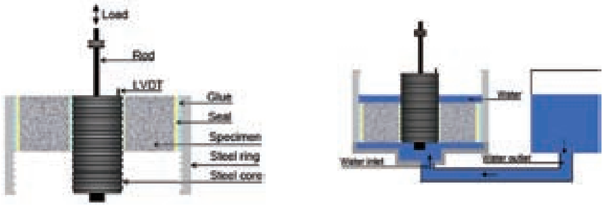


Figure 20. *CAST setup for dry test (left) and wet test (right)*

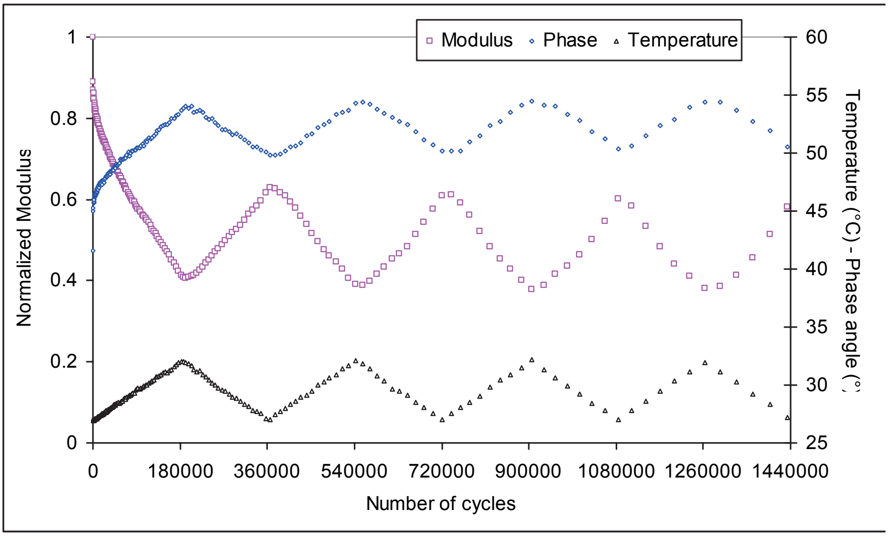


Figure 21. *CAST fatigue test experimental output*

The CAST results demonstrated that open graded mixtures with asphalt rubber binder had not only superior fatigue resistance but also significantly reduced moisture sensitivity as compared to traditional porous or semi-porous asphalt mixtures.

6. Conclusions

Progress in the use of recycled materials has been appreciable in the highway community over the last 20 years. However, further development is dependent on more cooperation among various disciplines: industry and government, highway engineers and environmental specialists.

In this paper, the Authors presented a complete proposal for an evaluation systemic approach aimed at the characterization of Asphalt Rubber mixtures (ARHM) and useful for areas where the Asphalt Rubber wet process technology has recently been introduced.

The selected approach provides parallel activities that give immediate answers for ARHM producers and applicators but that are also able to develop relevant scientific research.

The results so far obtained, and in part presented in this paper, are constantly in progress as there is also a continuous research for mixture production optimization. In any case, they clearly showed the enhanced performance of Asphalt Rubber mixtures in terms of mechanical and functional properties.

Despite the huge literature focusing on Asphalt Rubber study, this work shows how it is still possible and essential to enhance research addressing specific issues in order to implement and update the knowledge to date consolidated.

Acknowledgements

Part of this work is based on research activities funded by Asphalt Rubber Italia S.r.l. (Italy)

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De-icing Characteristics of Rubber Concrete Pavements

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ABSTRACT. The performance of rubberized asphalt has been investigated extensively and it is proven that rubberized asphalt is superior to conventional asphalt. The de-icing property of rubberized asphalt has attracted the attention of many Transportation Departments in the United States since early 1980. In few field test, significant reduction on stopping distance were observed during icy conditions as compared to control sections in Alaska, New Jersey, and Minnesota.

Simulated traffic (wheel passage) tests were used to study ice dis-bonding from the surface of rubberized asphalt, the results were somewhat encouraging. Ice dis-bonding were observed where the rubber particles where present. This results lead to the idea of "Chunk Rubber Asphalt Concrete" (CRAC).

The purpose of this paper is to demonstrate that de-icing is an inherent property of rubberized asphalt by correlating the de-icing property of rubberized asphalt to the interfacial surface tension between water and rubber modified bitumen and interpret the above contradictory results in terms of interfacial surface tension and rubber particle size.

KEYWORDS. De-Icing, Rubberized Asphalt, Interfacial Surface Tension

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1. Introduction

Current techniques to remove ice and compacted snow from pavement surfaces are costly because they needlessly crush or melt most of the ice layer. Also, in many cases they do not accomplish dis-bonding or clear the ice from the surface of the pavement. In addition, deicing chemicals currently in use (salts) are environmentally objectionable. More efficient alternatives to current physical and chemical methods are needed which will selectively apply environmentally acceptable energy to the ice - pavement interface to accomplish dis-bonding.

In 1980's and 1990's most of the Transportation Departments in the cold states installed rubberized asphalt in test sections in order to compare the performance of rubberized asphalt with conventional asphalt. It was believed that iced formed on the surface of rubberized asphalt will dis-bond from the surface of asphalt presumably because rubber acts as aggregates that flexes on the pavement surface under traffic. The flex helps to break the ice and the wind generated by passing vehicles moves the ice to the side of the road. In these experimental sections, from 1 to 3 percent granulated crumb rubber by weight of the total mix was added to the paving mix. The granulated rubber consisted of rubber particles ranging in size from 4.2 mm (1/4 in) to 2.0 mm (Number 10 sieve).

Alaska Department of Transportation installed experimental pavement sections using rubber with granulated rubber particle size ranging from 1.6 to 6.4 mm. On measuring vehicle stopping distances, significant reduction in distance were observed during icy conditions as compared to control sections (Takallou, *et al*., 1987). Some raveling was detected. Test section in route NJ41 in Cherry Hill, New Jersey showed improved skid resistance, however, some raveling were reported.

From test sections in other cold states that had used rubber with the above specifications it had been concluded that there is no difference between the performances of rubberized asphalt and conventional asphalt in cold climate (Stuart *et al*.,1991; Stuart *et al*.,1991; Stuart *et al*.,1991; Mogawer *et al*.,1989). Raveling was one of the major problems in these test sections.

Recently, the New Jersey Department of Transportation paved a section of Route 95 with asphalt rubber. In this project unlike previous projects fine crumb rubber (100 % passing 20 mesh screen), were used. Skid test showed that the rubber improved the skid resistance of the pavement and no raveling is reported so far (Asphalt 2008).

On laboratory level, Albert F. Wuori (Wuori,1993) studied the dis-bonding of ice from the surface of rubberized asphalt by simulated traffic method. Wheel passage tests of ice grown on the rubber-aggregate asphalt samples did not show significant ice dis-bonding from the surface, but the result where somewhat encouraging in a sense that ice-dis-bonding was observed on those areas of the surface that was covered by large rubber particles. The conclusion was that by increasing the percentage of larger rubber particles the potential for ice dis-bonding under wheel loading will increase. The technology referred to "Chunk Rubber Asphalt Concrete (CRAC)" used crumb rubber particles larger than 4.75 mm with rubber particle as large as 10 mm on the top of asphalt with the crumb rubber concentrations of 3, 6 and 12 percent. Simulated traffic (wheel passing) on the CRAC indicated that as rubber content increases, ice breaks faster from the surface. Ice dis-bonding was observed where ever the rubber particles were present, but the raveling was the major problem.

Raveling of rubber particles could be attributed to the available surface area of rubber particles for surface-surface interaction with bitumen and flexibility of rubber particles. For a constant weight of rubber, as the particle size of rubber decreases, its surface area increases. The surface area of 1 gram of 0.375 mm rubber particles is about 45 times larger than 1 gram of 10 mm rubber particles. Therefore the surface-surface bonding between rubber and asphalt of the latter is about 45 times stronger and particle flexibility in the latter practically is diminished.

The dis-bonding of ice from the surface of rubberized asphalt could be explained in terms of interfacial surface energy between water and the surface and flexibility of chunk rubber in CRAC technology. In order to demonstrate that interfacial surface tension and de-icing are correlated we will examine the interfacial surface tension of water-glass, water-Teflon and water-rubber.

The interfacial surface tension between water and glass is 144 ergs/cm^2. Water wets the surface of glass and ice formed on the surface of glass sticks to the glass and will not break down easily.

The interfacial surface tension between water and Teflon is 28 ergs/cm^2. Water does not wet the surface of Teflon and the ice formed on the surface of Teflon easily breaks down from the surface.

The interfacial surface tension between water and bitumen is 84 ergs/cm^2 and the interfacial surface tension of water-rubber is 48 ergs/cm^2. The ice formed on the surface of chunk rubber portion of CRAC is not bonded to the surface as strongly as to the bitumen portion of the asphalt; therefore, the ice formed over the rubber portion of CRAC will break under the weight of moving vehicles. The flexibility of rubber chunk helps the separation of ice from the surface the same way that flexibility of plastic in ice plastic trays helps the removal of ice cubes with a minor pressure.

Recently the surface free energy (SFE) characteristics of two Oklahoma aggregates with and without Styrene – Butadiene- Rubber (SBR) were evaluated for moisture induced damaged potential using a Universal Sorption Device (USD). SBR coating altered the aggregate surface from hydrophilic to hydrophobic .

The purpose of this paper is to demonstrate that de-icing is an inherent property of rubberized asphalt by correlating the de-icing property of rubberized asphalt to the interfacial surface tension between water and rubber modified bitumen and interpret the above contradictory results in terms of interfacial surface tension and rubber particle size.

2. Laboratory Experimental Design

The drop shape method is been used to measure the contact angle of a water drop formed on the surface of the sample. The interfacial surface energy is calculated by Young equation:

$$W_{S/L} = \gamma_{L/A} (1+\cos \Theta)$$

Measuring method is based on taking images of a steady drop on the surface of the sample. The images then were analyzed for contact angle measurements.

Two different rubber gradations were used in these investigations. Table 1 represents a typical gradation analysis for these two samples. The surface area of second sample is about 4 times more than the surface area of the first sample. Rubber was added to hot GP80-60 bitumen at 200 °C and mixed for 30 minutes. A drop of colored water was placed at room temperature on the surface of bitumen and bitumen modified with crumb rubber samples on the range of 2 to 16%. The images of the water drop on right angle to drop were recorded by a digital camera,

Table 1: *Gradation analysis for two types of crumb rubber*

Sieve size	**Sample 1**	**Sample 2**
	% Passing	% Passing
2.36-mm (#8)	100	100
2.00-mm (#10)	100	100
1.18-mm (#16)	50	100
600-µm (#30)	9	70
300-µm (#50)	3	25
150-µm (#100)	1	6
75-µm (#200)	0	1

The images were analyzed for contact angles measurements. The interfacial surface free energies were calculated for these samples by Young equation.

3. Laboratory Test Results

The interfacial surface tensions for between water drop and rubber modified bitumen are tabulated in table 2. The errors in interfacial surface measurements are attributed to two factors. The error related to the angle Θ measurements for the lack of a sharp boundary between the drop and the surface and the error related to the surface of the sample covered by rubber particles. The second sample covers more area and the coverage is more uniform. As the result the error in interfacial surface measurements in the second sample is less than the first one.

Table 2. *Interfacial surface tensions for bitumen modified rubber*

%Rubber	I.S.T ergs/cm^2 Sample 1	I.S.T ergs/cm^2 Sample 2
0	84±1	84±1
2	83±1	83±1
4	81±2	81±2
6	77±3	76±3
8	74±4	73±3
10	70±4	69±3
12	69±3	68±2
14	68±3	67±2
16	68±3	67±2

In both samples the interfacial surface tension decreases by adding rubber to bitumen and levels up at about 15% rubber. The interfacial surface tension of about 67-68 ergs/cm^2 is considerably less than that of bitumen (84 ergs/cm^2), but larger than that of rubber (48 ergs/cm^2), Therefore, it is expected that ice dis-bounding from the surface of asphalt modified with fine crumb rubber not to be as pronounced as dis-bounding from the surface of chunk rubber portion of CRAC, but to be more pronounced than none rubber portion of CRAC.

4. Conclusion

The interfacial surface tension of about 67 ergs/cm^2 of asphalt modified with 15-20% fine crumb rubber indicates that de-icing is an inherent property of rubberized asphalt. The disbanding of ice from such a surface will be uniform, the surface- surface interaction of rubber – bitumen will be much stronger and raveling practically will be diminished as been illustrated by many successful projects in states with warm climates and recent projects in New Jersey.

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