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Selection of reclaimed asphalt pavement sources and contents for asphalt mix production based on asphalt binder rheological properties, fuel requirements and greenhouse gas emissions

Ali Jamshidi, Meor Othman Hamzah*, Zulkurnain Shahadan

School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus, 14300 Nibong Tebal, Seberang Perai Selatan, Penang, Malaysia

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ABSTRACT

This paper characterizes the effects of reclaimed asphalt pavement (RAP) source on the rheological properties of virgin asphalt binders blended with 15% and 30% recovered binders. The recovered binders were extracted from three local RAP sources namely; the North-South Expressway (NSE), Damansara-Puchong Expressway (DPE) and Public Works Department (PWD) roads. The results showed that RAP source significantly influence the binder rheological properties at each aging state and test temperature. Environmental impacts were analyzed by estimating fuel requirements and Greenhouse Gas emissions in an asphalt mixing plant which was found to depend on RAP source and RAP content. Three scenarios were suggested to select the optimum RAP source and RAP content based on fuel requirement, emissions, and environmental condition.

1. Introduction

Waste materials from civil engineering construction and maintenance works have increasingly threatened the environment and public health. Environmental policy makers and project managers have tried to develop new technologies to recycle waste materials using energy efficient methods while simultaneously imposing minimum environmental loads. Therefore, factors related to sustainability and environmental impacts have equal if not greater importance for decision makers to consider in infrastructure construction. Currently, due to greater demands for natural resources conservation and sustainability, various construction and infrastructure development alternatives are assessed to select the most sustainable strategy using tools such as Life Cycle Assessment (LCA) and Leadership in Energy and Environmental Design (LEED) for all components and stages, including raw materials selection, method of construction, utility and recycling of a product, from cradle to the grave (Dovi et al., 2009; Millet et al., 2010; Huang et al., 2009; Optis and Wild, 2010; Bahr and Steen, 2004; Gabel et al., 2004; Manfredi et al., 2011). For example, Khoo et al. (2010) found that the type of energy provided to the product's life cycle has a significant effect on the total environmental impacts.

One alternative is the use of waste materials in infrastructure construction and rehabilitation. Pavements are one of the most important infrastructures that provide safe and efficient transportation for human communities while asphalt binder is the most prevalent material used in asphalt pavement construction. When an asphalt concrete pavement reaches the end of its design life, the road surfacing is milled, creating a milling waste material known as Reclaimed Asphalt Pavement (RAP) as indicated by arrow A in Fig. 1 (Al-Qadi et al., 2007). The RAP materials contain aggregate and asphalt binder which are transported to an asphalt plant for recycling (Arrow B in Fig. 1). RAP is 100% recyclable and has become a popular waste material in pavement construction and rehabilitation as illustrated by Arrow C in Fig. 1. For instance, in the United States, almost 100 million tons of RAP are produced annually, with about 60 million tons being reused in the construction of new asphalt pavements, while the remaining 40 million tons used in other pavement-related applications, such as aggregate road base (NAPA, 2009). Since aggregate materials are non-renewable natural resources, the primary benefit of using RAP is to reduce demand for extraction of new aggregate and helps to ease pressures on landfills (Chiu et al., 2008; Huang et al., 2007). Lee et al. (2010) showed that the use of RAP in pavement base and sub-base layers could reduce global warming potentials (20%), energy consumption (16%), water consumption (11%), life cycle costs (21%) and, hazardous waste generation (11%). Consequently, the use of RAP is in harmony with efforts to develop sustainable pavements based on the green design

* Corresponding author. Tel.: +60 45996210; fax: +60 45941009.
E-mail addresses: cemeor@yahoo.com, cemeor@eng.usm.my (M.O. Hamzah).

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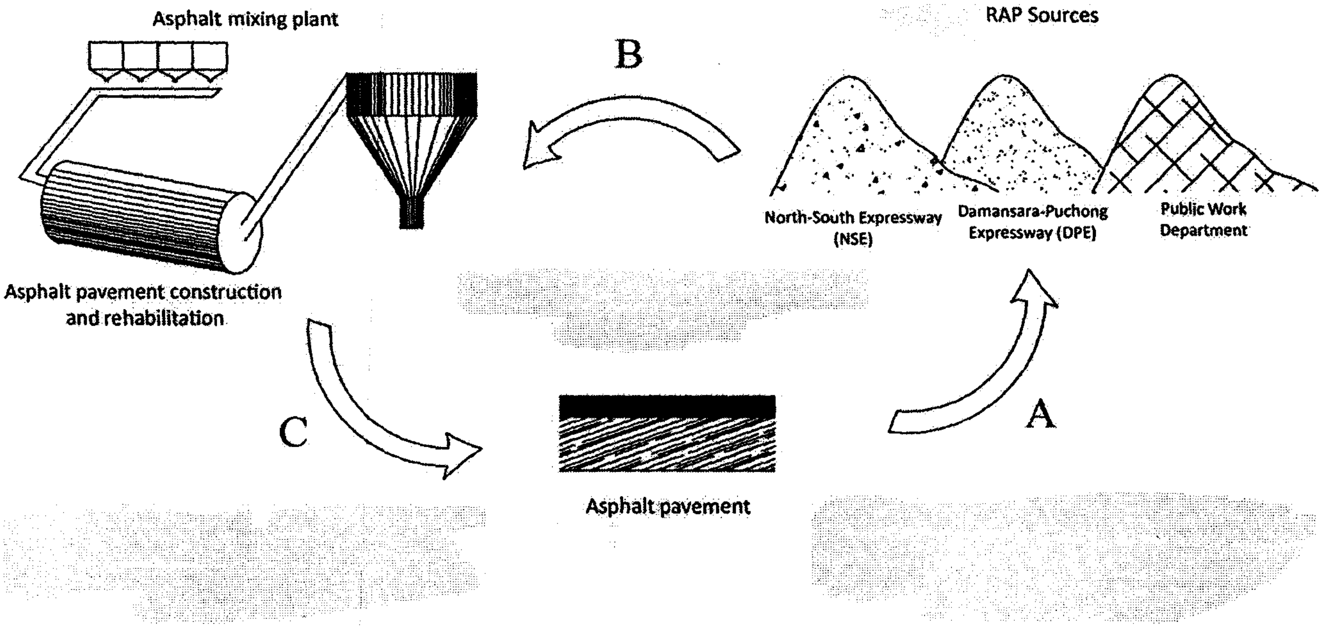


Fig. 1. Schematic life cycle of asphalt pavements.

concept since 40% of the global primary energy consumption and CO_2 emission is related to the production of materials (Hekkert et al., 2000).

There appear to be a lack of an extensive study focusing on the effects of different RAP binder sources and RAP contents on the rheological properties of virgin asphalt incorporating extracted RAP binder. Furthermore, there is no detailed investigation on increasing fuel requirement and Greenhouse Gas (GHG) emissions due to use of RAP from different sources during asphalt production. Therefore, the objective of this paper is to characterize the rheological properties of unaged and aged asphalt virgin binder blended with different percentages of extracted RAP binder from various RAP sources at high and intermediate temperatures. The paper also suggests an environmentally friendly method to select the optimum RAP binder content and source using the results of Superpave™ asphalt binder tests results depending on in-service temperature, and minimizing GHG emissions.

2. Materials and methods

2.1. Materials

Asphalt binder grading was based on Performance Grade (PG) as suggested by the Strategic Highway Research Program (SHRP). One of the products of SHRP is Superior Performance Asphalt Pavement (Superpave™) which outlines a new system for asphalt binder selection and specification. This new system incorporates performance-based asphalt binder properties with design environmental conditions to improve performance by controlling rutting, low temperature, and fatigue cracklings (Asphalt Institute, 2003). In this research, a PG64 was used as the base binder which means the asphalt binder properties must fulfill the requirements prescribed by Superpave™ at least up to 64 °C.

Table 1 summarizes its rheological properties of PG64. RAP milled from three Malaysian highways; the Damansara-Puchong Expressway (DPE), North-South Expressway (NSE) and Public Works Department (PWD) road were studied. Binder from the RAP samples were recovered and extracted via the Rotovapor method. In this method asphalt binder is extracted from the mix using an appropriate solvent. The Rotavapor Method is an established method to extract asphalt binder from the aggregate using an appropriate solvent and recovering the binder from the solvent without dramatically changing the asphalt binder's characteristics (ASTM D5404, 2011). Hence, the extracted asphalt binder can be tested using the same asphalt binder test as unaged binders. The rheological properties of the recovered RAP are presented in Table 2.

Table 1
Rheological properties of virgin PG64 binder.

Aging state	Test properties	Value
	Viscosity at 135 °C (Pa s)	0.465
	$G^*/\sin\delta$ at 64 °C (kPa)	1.23
	Failure temperature (°C)	66.4
Unaged binder (original state)	Specific heat capacity (J/kg/°C)	920
	Penetration (0.1 mm)	88
	Softening point (°C)	46
RTFO aged residue	$G^*/\sin\delta$ (kPa) at 64 °C	2.68
	Failure temperature (°C)	66
RTFO + PAV aged residue	$G^*\sin\delta$ at 25 °C (kPa)	2958.75

2.2. Methods

2.2.1. Sample preparation and designation

The extracted RAP binder was directly blended with the virgin binder at 140 °C in proportions of 15% and 30% by mass of asphalt

Table 2
Rheological properties of recovered RAP binder.

Source	Penetration test at 25 °C (dmm)	Softening point (°C)	Viscosity at 135 °C (Pa s)
NSE	14	67.5	3.06
DPE	13	67.5	4.33
PWD	11	72	2.14

Table 3
Rheological properties of virgin binders blended with 15% and 30% of extracted RAP binders.

Blends	Source	RAP modified binder	Penetration test at 25 °C (0.1 mm)	Softening point (°C)	Viscosity at 135 °C (Pa s)
PG64 + 15%	NSE	N15	67	49.5	0.45
PG64 + 30%		N30	51	54	0.58
PG64 + 15%	DPE	D15	63	50.5	0.49
PG64 + 30%		D30	46	56	0.7
PG64 + 15%	PWD	P15	63	50.5	0.45
PG64 + 30%		P30	48	55	0.58

binder. Their properties are shown in Table 3. As shown in Table 3, binder samples are designated according to their source (D for DPE, N for NSE and P for PWD), percentage of extracted binder blended (15% or 30%) with virgin binder and their aging states (U for unaged, S for short term and L for long-term aging). Where necessary, a letter M is incorporated to denote an asphalt mix. Hence, binder sample D15S denotes a virgin binder blended with 15% recovered RAP binder sourced from DPE subjected to short term aging, while asphalt mix D15M refers to an asphalt mix prepared with virgin binder blended with 15% extracted RAP binder sourced from DPE. Virgin binder blended with extracted RAP binder can also be generally described as RAP modified binder.

2.2.2. Aggregate gradation and mix design

Granite aggregate was used, while the aggregate gradation used was in accordance with the Malaysian PWD specifications (PWD, 2008) for asphaltic concrete mix type ACW14 designed based on the Marshall Method according to ASTM D1559 (2006) procedures. The mix specifications are shown in Table 4. The granite aggregate specific heat capacity at 25 °C is 790 J/kg/°C.

2.2.3. Binder aging protocol

Binder samples were aged using the Rolling Thin Film Oven Test (RTFO) and Pressure Aging Vessel (PAV) in accordance with ASTM D2872 (2006) and ASTM D6521 (2006), respectively, to simulate short-term and long-term aging.

2.2.4. Characterization at high temperature

Viscosity was used to evaluate the effects of RAP binder contents on the rheological characteristics of virgin binder blended with recovered RAP binder at high temperatures (125–165 °C) using a Brookefield Viscometer.

2.2.5. Characterization at intermediate temperature

The rheological properties of the virgin and RAP modified binders were measured using the Dynamic Shear Rheometer (DSR) at intermediate temperatures in the terms of Superpave™, rutting ($G^*/\sin\delta$) and Superpave™ fatigue ($G^*\cdot\sin\delta$) factors. Temperature sweeps were applied from 52 °C to 82 °C at 6 °C increments for evaluation of $G^*/\sin\delta$ according to Superpave™ recommendations. However, for $G^*\cdot\sin\delta$, temperature sweeps were performed from 19 °C to 31 °C at 3 °C increments (Asphalt Institute, 2003).

Table 4
Specifications of mixes designed to PWD specifications for mix type ACW14.

Properties of Mix	Value	Prescribed ranges by PWD (2008)
Binder content	5%	4–6%
Air void	3.90%	3–5
VFA[a]	75.60%	70–80
VMA[b]	15.37%	–
Density	2.38	–
Stability	20.64KN	More than 8.00KN

[a] Void filled asphalt.
[b] Void mineral aggregate.

3. Results and discussion

3.1. Characterization at high temperatures

3.1.1. Effects of RAP on viscosity

Viscosity behavior differs according to binder types and aging conditions. The relationships between viscosity and temperature of unaged, short term aged, and long term aged asphalt binder at the sweep temperatures are illustrated in Fig. 2.

Fig. 2 shows that the viscosity–temperature dependency exhibit a polynomial trend with R^2 greater than 0.9 at the sweep temperatures irrespective of RAP binder sources and sample aging conditions. It also shows that D30 sample exhibits the highest viscosity regardless of aging state. The results of analysis of variance (ANOVA) in Table 5 also show that RAP source and RAP content have significant effects on the viscosity.

3.1.2. Effects of RAP binder on construction temperatures

In asphalt mix production, the binder should be fluidic enough to completely coat the aggregate particles and bind them together after compaction has ceased. In this regards, the Asphalt Institute recommends that the mixing and compaction temperatures should, respectively, correspond to viscosity ranges 170 ± 20 mPa s and 280 ± 30 mPa s (Asphalt Institute, 2003). Using this guide, Table 6 shows the corresponding mixing and compaction temperatures of virgin and RAP modified binders.

From Table 6, the construction temperatures of RAP modified binder from NSE (N15 and N30) and PWD (P15 and P30) are identical. It can also be observed that D30 requires the highest mixing and compaction temperatures. Higher construction temperature can cause delay in opening pavement to traffic because the compacted mixes require more time to cool down. The P15 and N15 binders require the lowest construction temperature compared to the others.

3.2. Characterization at intermediate temperatures

3.2.1. Rutting parameter

According to Superpave™, rutting is controlled by limiting the $G^*/\sin\delta$ to a value greater than 1 kPa and 2.20 kPa for unaged and short term aged binders, respectively (Asphalt Institute, 2003). Table 7 shows the effects of RAP binder sources and contents on the rutting parameter of virgin and RAP modified binders at various temperatures. Table 8 presents the analysis of variance (ANOVA) for $G^*/\sin\delta$. The results indicate that RAP source and RAP content are significant factors affecting $G^*/\sin\delta$ at each aging state.

The $G^*/\sin\delta$ of unaged and short term aged RAP modified binders increases significantly irrespective of their RAP sources at each test temperature. This is due to the effect of aged binder which increased binder samples stiffness and contribute to upgrading the performance grade of RAP modified binders as shown in Table 9.

Table 9 shows that the binder upgrading depends not only on the amount of RAP binder but also on the RAP binder source. For example, N15 is not upgraded to higher binder performance grade, while the performance grade of D15 increases to PG70. Meanwhile, D30 is able to promote binder grading to PG76, while the corresponding value of RAP modified binders from PWD and NSE only upgrade the binder grading to PG70.

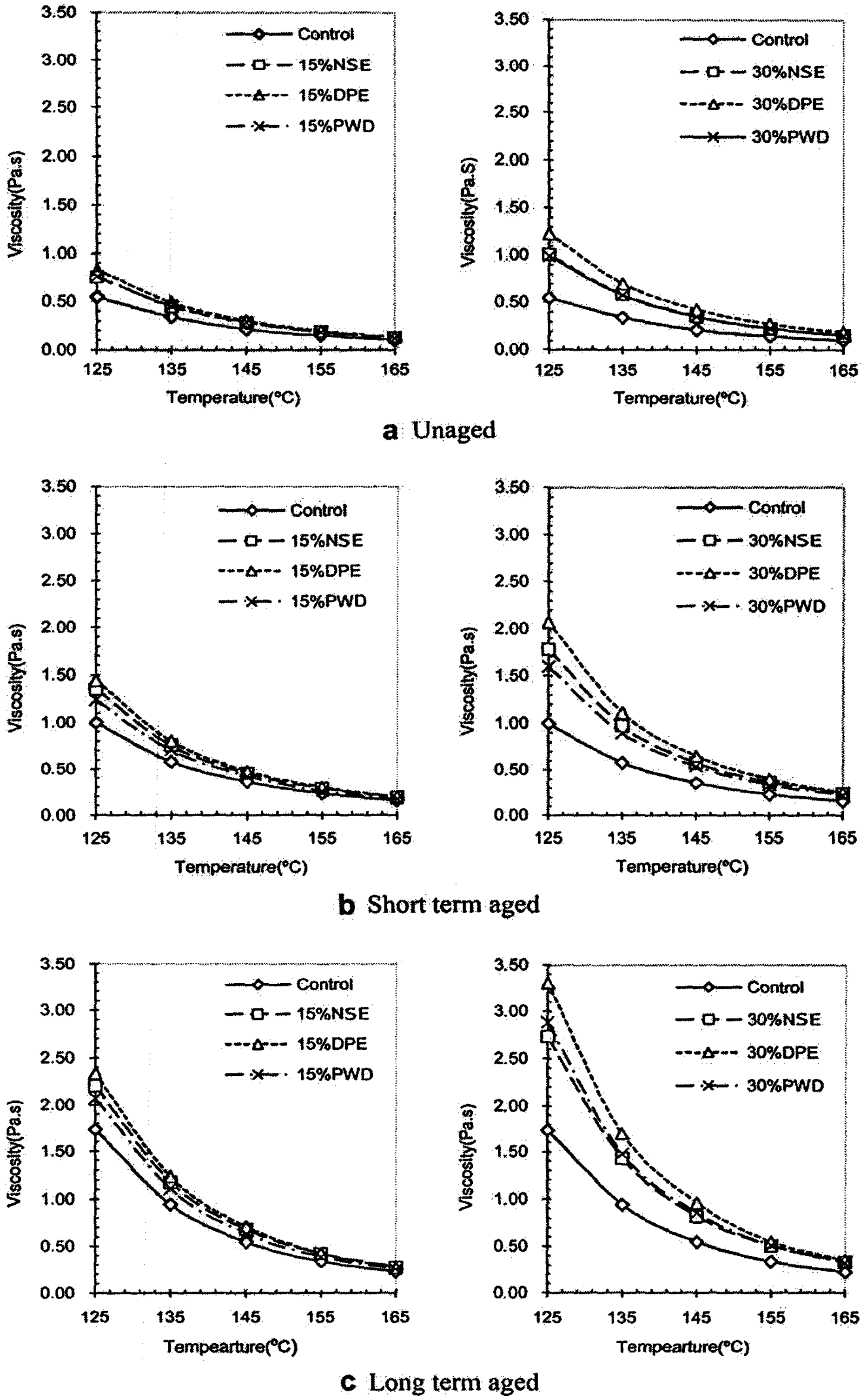


Fig. 2. Viscosity–temperature dependency of virgin and RAP modified binder.

3.2.2. Effects of RAP binder on aging factor

Aging changes the binder properties over time during construction and service life which makes the binder less adhesive, and becomes harder and brittle. To evaluate aging, the Aging Factor (AF) which is defined as the ratio between aged and unaged Superpave™ rutting parameter (Wasiuddin et al., 2007) is computed. Samples with lower AF values are the least susceptible to aging and are more desirable. The AF values for all RAP modified binders at different temperatures are presented in Table 10. From Table 10, two distinct trends can be observed. Firstly, AF increases

Table 5
Results of analysis of variance (ANOVA) for viscosity.

Source	Sum of squares	df	Mean square	F	p-Value	Significant
Intercept	149596262.296	1	149596262.296	464411.032	<0.001	Yes
Source	605067.480	2	302533.740	939.195	<0.001	Yes
Temperature	76570477.604	4	19142619.401	59426.910	<0.001	Yes
Aging	19979584.643	2	9989792.322	31012.605	<0.001	Yes
Content	2915891.280	1	2915891.280	9052.179	<0.001	Yes
Source * temperature	464449.089	8	58056.136	180.231	<0.001	Yes
Source * aging	73245.227	4	18311.307	56.846	<0.001	Yes
Temperature * aging	13803084.301	8	1725385.538	5356.338	<0.001	Yes
Source * temperature * aging	85382.941	16	5336.434	16.567	<0.001	Yes
Source * content	127102.117	2	63551.059	197.290	<0.001	Yes
Temperature * content	1881280.599	4	470320.150	1460.076	<0.001	Yes
Source * temperature * content	98846.584	8	12355.823	38.358	<0.001	Yes
Aging * content	365173.278	2	182586.639	566.827	<0.001	Yes
Source * aging * content	40527.460	4	10131.865	31.454	<0.001	Yes
Temperature * aging * content	293594.643	8	36699.330	113.930	<0.001	Yes
Source * temperature * aging * content	41225.701	16	2576.606	7.999	<0.001	Yes
Error	57981.670	180	322.120			
Total	266999176.915	270				
Corrected total	117402914.619	269				

*: Interaction; source: RAP source; content: RAP content.

from 52 °C and peaked at 64 °C for the virgin and RAP modified binder samples (N30). The same trend is observed for D30, P30, and P15 but at 58 °C. Beyond the peak, the AF decreases at the sweep temperatures. Secondly, AF decreases over the test temperatures (D15 and N15). The remaining RAP modified binder samples follow the second trend. Samples containing extracted RAP binder sourced from DPE (D15 and D30) exhibit the maximum AF value at each individual test temperature. It denotes that the characteristics of DPE RAP modified binder in terms of the AF based on $G^*/\sin\delta$ has changed significantly as a consequence of aging. Meanwhile, the minimum AF value takes place for NSE RAP modified binder (N15 and N30) in over the test temperatures. The AF values at this temperatures range are even less than the corresponding value of the virgin samples from 52 °C to 64 °C.

3.2.3. Fatigue parameter

Incorporating RAP binder increases its $G^*/\sin\delta$, making it stiffer and subsequently subjecting the asphalt pavement to fatigue failure especially at low temperature. It is also necessary to establish a balance between the amount of RAP and its performance at intermediate temperatures. In this respect, the asphalt binder fatigue factor ($G^*\sin\delta$) obtained from the DSR test results is used to determine the RAP modified binder performance at intermediate temperatures. According to Superpave™ specifications (Asphalt Institute, 2003), the value of $G^*\sin\delta$ should be less than 5 MPa or the area located below the dashed horizontal line shown in Fig. 3.

A lower value is more desirable from the viewpoint of resistance to fatigue failure. As anticipated, $G^*\sin\delta$ increases as the RAP binder content increases. However, the $G^*\sin\delta$ values differ according to RAP binder sources. For example, the highest percentage of extracted RAP binder which can satisfy the Superpave™ fatigue specifications at 22 °C is 5% sourced from NSE and DPE (Fig. 3(a) and (b)), while the corresponding value is 10% for extracted binder from RAP sourced from PWD at 22 °C (Fig. 3(c)). It can also be seen that P30 can fully satisfy Superpave™ fatigue requirements at 28 °C, while N30 can only satisfy this demand at 31 °C. However, D30 cannot even fulfill the Superpave™ requirements at 31 °C.

Therefore, RAP binder source can significantly affect the rheological property of the RAP modified binder in terms of the $G^*\sin\delta$ parameter. As RAP materials are used from various sources without considering their rheological properties, it is possible that asphalt mixes become inhomogeneous and the asphalt pavement cannot react uniformly under traffic and environmental loadings during its service life. It is also possible to develop patterns of the pavement failures that do not follow the predictions by pavement management system sectors. Under these circumstances, short term and long term pavement maintenance and rehabilitation policies to select cost-effective strategy, can become inefficient.

3.2.4. Analysis of fuel requirement and GHG emissions

It is very desirable for environmental policy makers and engineers to be able to measure the environmental loads of each RAP source to ensure cleaner asphalt production and field construction since asphalt binder and aggregate from different sources may exhibit different characteristics (Hamzah et al., 2010). Project managers and engineers should also quantify the fuel requirements, hence total cost of fuel consumption. The selected option

Table 6
Construction temperatures for virgin and RAP modified binders.

Parameter	Virgin	RAP modified binder					
		N15	N 30	D15	D30	P15	P30
Mixing, °C	155	161	165	163	168	161	165
Compaction, °C	145	151	155	153	158	151	155

Table 7
$G^*/\sin\delta$ for unaged and short term aged virgin and RAP modified binders.

State	Temperature (°C)	Virgin	$G^*/\sin\delta$ (kPa) RAP modified binder N15	N30	D15	D30	P15	P30
Unaged	52	5.80	14.30	22.09	12.10	23.10	12.00	19
	58	2.44	5.46	8.47	4.55	8.69	4.58	7.44
	64	1.14	2.28	3.53	1.96	3.81	1.96	3.12
	70	0.57	1.09	1.59	1.01	1.71	0.94	1.43
	76	0.29	0.52	0.78	0.45	0.84	0.45	0.67
	82	0.18	0.26	0.39	0.22	0.41	0.23	0.34
STA[a]	52	13.09	28.31	37.34	37.54	76.84	28.81	47.09
	58	5.33	10.61	16.88	14.02	36.11	11.43	18.27
	64	2.68	4.31	7.35	5.71	15.48	4.51	7.37
	70	1.11	1.85	3.17	2.45	6.54	1.94	3.13
	76	0.38	0.84	1.41	1.10	2.80	0.88	1.38
	82	0.29	0.41	0.67	0.53	1.24	0.42	0.65

[a] Short-term aging.

Table 8
Results of analysis of variance (ANOVA) for $G^*/\sin\delta$.

Source	Sum of squares	df	Mean square	F	p-Value	Significant
Intercept	15594673255.967	1	15594673255.967	31217.653	<0.001	Yes
Source	591842545.813	2	295921272.906	592.380	<0.001	Yes
Temperature	23184211903.445	5	4636842380.689	9282.101	<0.001	Yes
Aging	2994949568.535	1	2994949568.535	5995.335	<0.001	Yes
RAP content	1235878467.200	1	1235878467.200	2474.000	<0.001	Yes
Source * temperature	767727330.761	10	76772733.076	153.685	<0.001	Yes
Source * aging	547793190.597	2	273896595.298	548.290	<0.001	Yes
Temperature * aging	4327345560.076	5	865469112.015	1732.509	<0.001	Yes
Source * temperature * aging	735868058.092	10	73586805.809	147.307	<0.001	Yes
Source * content	297041857.068	2	148520928.534	297.311	<0.001	Yes
Temperature * content	1567694820.293	5	313538964.059	627.647	<0.001	Yes
Source * temperature * content	355020747.229	10	35502074.723	71.069	<0.001	Yes
Aging * content	303338196.540	1	303338196.540	607.227	<0.001	Yes
Source * aging * content	189137239.567	2	94568619.783	189.309	<0.001	Yes
Temperature * aging * content	320823466.882	5	64164693.376	128.446	<0.001	Yes
Source * temperature * aging * content	226387040.328	10	22638704.033	45.319	<0.001	Yes
Error	71934713.607	144	499546.622			
Total	53311667962.000	216				
Corrected total	37716994706.033	215				

*: Interaction; source: RAP source; content: RAP content.

Table 9
Upgrading of PG64 binder due to incorporating RAP binder.

Parameter	RAP modified binder					
	N15	30N	D15	D30	P15	P30
A	PG66	PG73	PG70	PG77	PG70	PG73
S	PG64	PG70	PG70	PG76	PG70	PG70

A: actual grading; and S: standard grading.

should consider binder rheological properties, fuel requirements and the GHG emissions so that asphalt production by the mixing plant can be made more environmental friendly. In this regards, the required amount of fuel to heat up aggregate and binder from 25 °C (the assumed ambient temperature at the quarry) up to the mixing temperatures as specified in Table 6 are calculated using Eq. (1).

$$Q = \sum_{i=n}^{j=n+1} mc\Delta\theta \tag{1}$$

where Q is the sum of required heat energy (J), m is the mass of materials (kg), c is the specific heat capacity coefficient (J/(kg/°C)), $\Delta\theta$ is the difference between the ambient and mixing temperatures (°C), and i and j indicate different material types. The specific heat capacity changes with temperature. The specific heat capacity values at the range of temperature, from ambient temperature to mixing points, were calculated using equations developed by Hamzah et al. (2010) and Waples and Waples (2004).

The mix density and the other volumetric properties are based on the values for the control samples (Table 4) and the total materials required, including asphalt binder and aggregate, to pave a 10-km dual carriageway road with 3 lanes per direction and

Table 10
AF values for virgin and RAP modified binders.

Temperature (°C)	Virgin	Aging factor					
		RAP modified binder					
		N15	N30	D15	D30	P15	P30
52	2.26	1.98	1.69	3.10	3.32	2.40	2.47
58	2.18	1.94	1.99	3.08	4.15	2.49	2.45
64	2.35	1.89	2.08	2.91	4.06	2.30	2.36
70	1.95	1.69	1.99	2.63	3.82	2.06	2.18
76	1.31	1.62	1.80	2.44	3.33	1.95	2.05
82	1.61	1.57	1.71	2.41	3.02	1.82	1.91

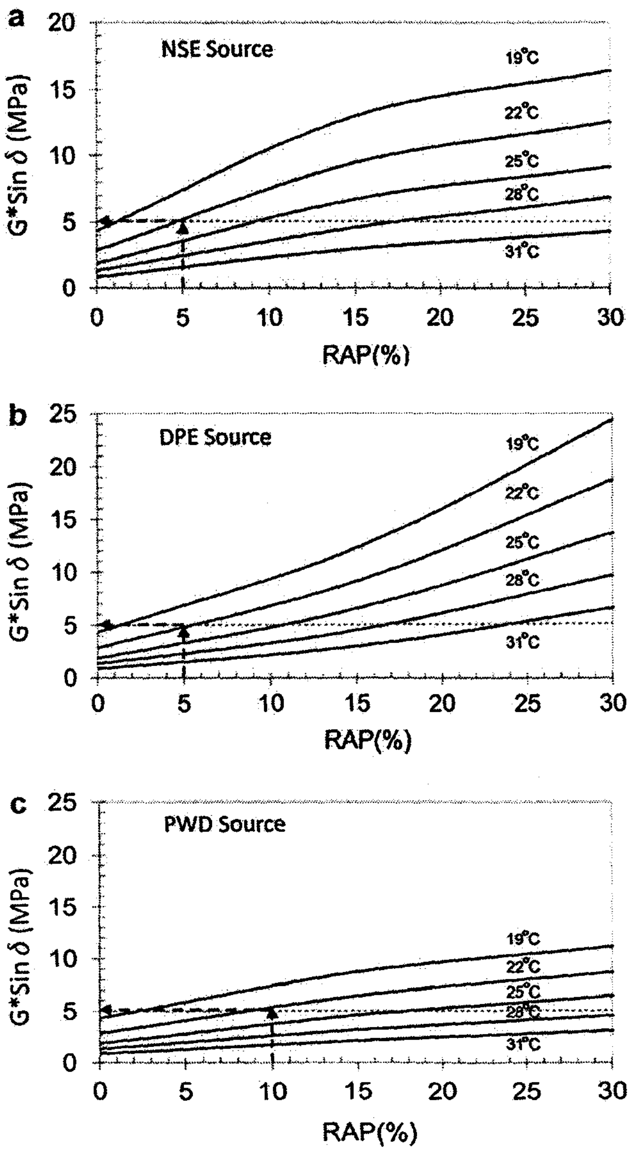


Fig. 3. Relationship between $G^*\sin\delta$, temperature, RAP binder source and content.

Table 11
Conversion factors for different fuel types (DTI, 2006).

Fuel Type	Unit	Coefficient
Coal	ton/MJ	0.00004263
Natural gas	m^3/MJ	0.02610824
Diesel	ton/MJ	0.00002181

Table 12
Conversion factors for GHG (DEFRA, 2010).

Fuel Type	CO_2 (kg CO_2/Unit)	CH_4 (kg CO_2e/Unit)	N_2O (kg CO_2e/Unit)
Coal	2295.3	1.8	39.4
Natural gas	2.023	0.003	0.0012
Diesel	3164.3	1.8	35

Table 13
Fuel requirements based on RAP source and RAP content.

Mixes	Q_{agg} (TJ)	Q_b (TJ)	Q_T (TJ)	Fuel type			Increase in fuel (%)
				Coal (ton)	Natural gas (m^3)	Diesel (ton)	
Virgin	2.95	0.161	3.11	133	81,197	68	
N15M	3.10	0.169	3.27	139	85,374	71	4.7
N30M	3.20	0.174	3.37	144	87,985	74	8.5
D15M	3.15	0.171	3.32	142	86,679	72	6.5
D30M	3.27	0.178	3.45	147	90,073	75	10.6
P15M	3.10	0.169	3.27	139	85,374	71	4.7
P30M	3.20	0.174	3.37	144	87,985	74	8.5

TJ: tera joule; Q_{agg}: Required energy to heat up aggregate; Q_b: required energy to heat up asphalt binder; and Q_T: required energy to heat up aggregate and asphalt binder.

a 5 cm thick wearing course. Assume that coal, natural gas and diesel are the industrial fuels used in the asphalt plant. The required heat energy is converted to the required fuel types and GHG emissions (CO_2, N_2O, and CH_4) using conversion factors given in Table 11 (DTI, 2006) and Table 12 (DEFRA, 2010), respectively. Tables 13 and 14 show required fuel types and the GHG emissions based on RAP source and RAP contents, respectively.

From Tables 13 and 14, the highest fuel requirements and the GHG emissions hence the most hazardous, are for asphalt mix impregnated with D30M corresponding to the highest construction temperatures (Table 6). Higher construction temperature can lead to emissions of more fumes in the asphalt mix plant and paving site. The fumes contain polycyclic aromatic hydrocarbons (PAHs), some of which are carcinogenic and hence their emission can make paving site hazardous for asphalt paving crews (Tsai et al., 2004; Karkaya et al., 1999; Kuo et al., 2008). The most environmental friendly options are asphalt mixes incorporating N15M and P15M. It can also be seen that the increase in fuel requirement and GHG emissions are different based on RAP source and RAP content irrespective of fuel type. In the asphalt mixing plant, the actual amount of GHG emissions and fuel requirement is enormous since both are calculated for large-scale asphalt production for huge road construction projects. Tables 13 and 14 can be used as guidelines for a preliminary evaluation in the choice of RAP sources and contents. The fuel requirement and the GHG emissions can be different based on the efficiency of an asphalt plant.

4. Selection of RAP source and RAP content

To choose a preliminary optimum RAP content and source, three critical parameters are selected to evaluate binder performance namely construction temperatures, Superpave™ fatigue, and aging factors. These parameters can be very advantageous because the chosen construction temperatures enable energy and environmental policy makers to assess increase in energy or fuel requirement and GHG emissions from different RAP sources since energy saving and reduction in GHG emission has been major current challenge in the world (Klemes et al., 2010; Jeong and Mo, 2009; Burandt and Barth, 2010) (Tables 13 and 14). If engineers and asphalt researchers need to evaluate binder performance and aging potentials of asphalt binder samples at the pavement in-service temperatures, then the Superpave™ fatigue, rutting and aging factors, respectively are useful (Fig. 3, Tables 7 and 10). Under these circumstances, the following viable scenarios can be suggested to select the preliminary optimum RAP binder content and source using the results of Superpave™ asphalt binder tests.

4.1. Scenario 1

If the in-service temperature of the asphalt pavement is high and the pavement is subjected to heavy traffic volume, then asphalt mix designated as D30M is recommended because incorporating such binder can upgrade the binder performance grade to PG76, which in turn increases its rutting resistance (Tables 7 and 9). However, it should be noted that the fuel requirement and the GHG emissions from asphalt mix D30M is the highest when compared to other mixes as shown in Tables 13 and 14.

4.2. Scenario 2

If the minimization of energy requirements and GHG emissions are of paramount importance due to the necessity to preserve the environment, then the most environmental friendly option is to adopt N15M because the aging factor, the corresponding fuel requirement, and the GHG emissions are the least as shown in Tables 10, 13, and 14, respectively.

4.3. Scenario 3

If the in-service temperature is as low as 22 °C, then resistance to fatigue must be considered. In this case, the highest permissible

Table 14
GHG emissions[a] based on RAP source and RAP content.

Mixes	Coal			Increase in GHG emission (%)	Diesel			Increase in GHG emission (%)	Natural gas			Increase in GHG emission (%)
	CO_2 (kg CO_2/t)	N_2O (kg CO_2e/t)	CH_4 (kg CO_2e/t)		CO_2 (kg CO_2/t)	N_2O (kg CO_2e/t)	CH_4 (kg CO_2e/t)		CO_2 (kg CO_2/m^3)	N_2O (kg CO_2e/m^3)	CH_4 (kg CO_2e/m^3)	
Virgin	305274.9	5240.2	239.4		215,172	2380	122		164,262	98	244	
N15M	319046.7	5476.6	250.2	4.51	224,665	2485	127	4.41	172,712	103	256	5
N30M	330523.2	5673.6	259.2	8.27	234,158	2590	133	8.82	177,994	106	264	8. 5
D15M	325932.6	5594.8	255.6	6.76	227,829	2520	129	5.88	175,352	104	260	6.8
D30M	337409.1	5791.8	264.6	10.52	237,322	2625	135	10.29	182,218	108	270	11
P15M	319046.7	5476.6	250.2	4.51	224,665	2485	127	4.41	172,712	103	256	5
P30M	330523.2	5673.6	259.2	8.27	234,158	2590	133	8.82	177,994	106	264	8.5

[a] The direct GHG emissions based on scope 1 in the GHG protocol (DEFRA, 2010).

RAP content and the best source to satisfy the fatigue criteria are about 10% and sourced from PWD (P10M), respectively, as shown in Fig. 3.

The above suggestions on RAP source and content selections are based on the binder test results. It is recommended to carry out mix performance tests for a more comprehensive mix evaluation.

5. Conclusion

A comprehensive rheological characterization of virgin asphalt binder blended with RAP binder using Superpave™ binder tests at high and intermediate temperatures as well as different aging states showed that RAP source and RAP content have significant effects on the asphalt binder properties. Changes in asphalt binder performance gradation also depended on RAP source and RAP content which the maximum asphalt upgrading was PG76 for 30D, while the maximum upgrading was PG70 using the identical amount of RAP from the other RAP sources. The maximum construction temperature was also for 30D.

The rheological characterization of virgin asphalt binder blended with RAP is beneficial for the assessment of preliminary fuel requirements and the GHG emissions in selecting the optimum RAP source and RAP contents while considering the construction temperatures aging, rutting and fatigue criteria. This is very advantageous in the context of the environment and sustainability to produce cleaner asphalt mixes using RAP material.

References

Al-Qadi, I.L., Elseifi, M., Carpenter, S.H., 2007. Reclaimed Asphalt Pavement – a Literature Review. FHWA-ICT-07-001. Illinois Center for Transportation, USA.

Asphalt Institute, 2003. Superpave™ Performance Graded Asphalt: Binder Specifications and Testing, SP-1. The Asphalt Institute, Lexington, KY, USA.

ASTM D1559, 2006. Test Method for Resistance of Plastic Flow of Bituminous Mixtures Using Marshall Apparatus. ASTM, West Conshohocken, PA, USA.

ASTM D 2872, 2006. Standard Test Method for Effect of Heat and Air on Moving Film of Asphalt (Rolling Thin-Film Oven Test). ASTM, West Conshohocken, PA, USA.

ASTM D6521, 2006. Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV). ASTM, West Conshohocken, PA, USA.

ASTM D5404, 2011. Standard Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator. ASTM, West Conshohocken, PA, USA.

Bahr, B.V., Steen, B., 2004. Reducing epistemological uncertainty in life cycle inventory. J. Clean. Prod. 17 (2), 369–388.

Burandt, S., Barth, M., 2010. Learning settings to face climate change. J. Clean. Prod. 18 (7), 659–665.

Chiu, C.T., Hsu, T.H., Yang, W.F., 2008. Life cycle assessment on using recycled materials for rehabilitating asphalt pavements. Res. Conserv. Recycl. 52 (3), 545–556.

Department for Environment, Food and Rural Affairs (DEFRA), 2010. Guidelines to GHGs calculation, Version 1.2.1. DEFRA.

Dovi, V.G., Friedler, F., Huisingh, D., Klemes, J.J., 2009. Cleaner energy for sustainable future. J. Clean. Prod. 17 (10), 889–895.

Department of Trade and Industry (DTI), 2006. Digest of United Kingdom Energy Statistics. DTI.

Gabel, K., Forsberg, P., Tillman, A.-M., 2004. The design and building of a life cycle-based process model for simulating environmental performance, product performance and cost in cement manufacturing. J. Clean. Prod. 12 (1), 77–93.

Hamzah, M.O., Jamshidi, A., Shahadan, Z., 2010. Evaluation of the potential of Sasobit® to reduce required heat energy and CO_2 emission in the asphalt industry. J. Clean. Prod. 18 (18), 1859–1865.

Hekkert, M.P., Joosten, L.A.J., Worrell, E., 2000. Reduction of CO_2 emissions by improved management of material and product use: the case of transport packaging. Res. Conserv. Recycl. 30 (1), 1–27.

Huang, Y., Bird, R.N., Heidrich, O., 2007. A review of the use of recycled solid waste materials in asphalt pavements. Res. Conserv. Recycl. 52 (1), 58–73.

Huang, Y., Bird, R., Heidrich, O., 2009. Development of a life cycle assessment tool for construction and maintenance of asphalt pavements. J. Clean. Prod. 17 (2), 283–296.

Jeong, I.-T., Lee, Mo K., 2009. Assessment of the ecodesign improvement options using the global warming and economic performance indicators. J. Clean. Prod. 17 (13), 1206–1213.

Karkaya, A., Yusecoy, B., Turhan, A., Erdem, O., Burgaz, S., Karkaya, A.E., 1999. Investigation of some immunological functions in a group of asphalt workers exposed to polycyclic aromatic hydrocarbons. J. Toxicol. 135 (1), 43–47.

Khoo, H.H., Tan, R.B.H., Chng, K.W.L., 2010. Environmental impacts of conventional plastic and bio-based carrier bags Part 1: life cycle production. Int. J. Life Cycle Assess. 15 (3), 284–293.

Klemes, J.J., Varbanov, P.S., Pierucci, S., Huisingh, D., 2010. Minimising emissions and energy wastage by improved industrial processes and integration of renewable energy. J. Clean. Prod. 18 (9), 843–847.

Kuo, C.-Y., Yang, Y.-H., Chao, M.-R., Hu, C.-W., 2008. The exposure of temple workers to polycyclic aromatic hydrocarbons. J. Sci Tot. Environ. 401 (1–3), 44–50.

Lee, J.C., Edil, B.E., Tinjum, J.M., Benson, C.H., 2010. Quantitive assessment of environmental and economy benefits of recycled materials in highway construction. Transportation Res. Rec.: J. Transportation Res. Board 2158, 138–142.

Manfredi, S., Tonini, D., Christensen, T.H., 2011. Environmental assessment of different management options for individual waste fractions by means of life-cycle assessment modeling. Res. Conserve. Recycle 55 (11), 995–1004.

Millet, D., Bistagnino, L., Lanzavecchia, C., Camous, R., Poldma, T., 2010. Does the potential of the use of LCA match the design team needs? J. Clean. Prod. 15 (4), 335–346.

National Asphalt Pavement Association (NAPA), 2009. Black and Green: Sustainable Asphalt, Now and Tomorrow. Special Report 200. NAPA, Maryland, USA.

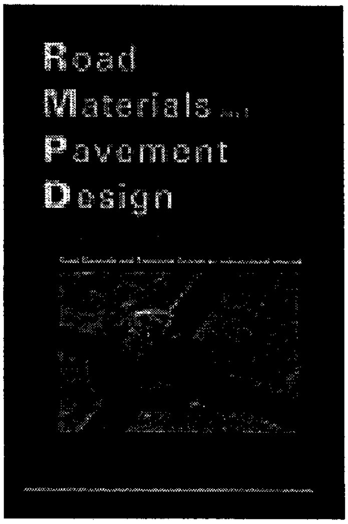
Optis, M., Wild, P., 2010. Inadequate documentation in published life cycle energy reports on buildings. Int. J. Life Cycle Assess. 12 (4), 644–651.

Public Works Department (PWD), 2008. Standard Specification for Road Works in Malaysia, Section Four: Flexible Pavements, Malaysia. Public Works Department.

Tsai, P.-J., Shin, T.-S., Chen, H.-L., Lee, W.-J., Lai, C.-H., Liou, S.-H., 2004. Assessing and predicting the exposures of polycyclic aromatic hydrocarbons (PAHs) and their carcinogenic potencies from vehicle engine exhausts to highway toll station workers. J. Atmos. Environ. 38 (2), 333–343.

Waples, D.W., Waples, J.S., 2004. A review and evaluation of specific heat capacities of rocks, minerals, and surface fluids, Part 1: mineral and non porous rocks. Nat. Resour. Res. 13 (2), 97–122.

Wasiuddin, N.M., Selvamohan, S., Zaman, M.M., Guegan, M.L.T.A., 2007. Comparative laboratory study of Sasobit® and aspha-min additives in warm mix asphalt. Transportation Res. Rec. J. Transportation Res. Board 1998, 82–88.



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Parameters to characterise the effects of Sasobit® content on the rheological properties of unaged and aged asphalt binders

Meor Othman Hamzah [a] , Ali Jamshidi [a] , Kunnawee Kanitpong [b] & Mohamad Yusri Aman [a]

[a] School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus, 14300 Nibong Tebal, Seberang Perai Selatan, Penang, Malaysia

[b] School of Engineering and Technology, Asian Institute of Technology, Thailand

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Parameters to characterise the effects of Sasobit® content on the rheological properties of unaged and aged asphalt binders

Meor Othman Hamzah[a]*, Ali Jamshidi[a], Kunnawee Kanitpong[b] and Mohamad Yusri Aman[a]

[a]*School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus, 14300 Nibong Tebal, Seberang Perai Selatan, Penang, Malaysia;* [b]*School of Engineering and Technology, Asian Institute of Technology, Thailand*

Incorporation of warm asphalt additives in conventional asphalt binder affects its rheological properties. This paper presents two Superpave™ binder parameters that were used to characterise the rheological properties of PG64 and PG70 asphalt binders blended with various Sasobit® contents and subjected to high and intermediate temperatures as well as under different ageing conditions. The parameters used to characterise the binder rheology at high and intermediate temperatures were the non-dimensional viscosity gradient ($\nabla\eta_S$) and non-dimensional Superpave™ rutting parameter (NSRP), respectively). The results indicated that the $\nabla\eta_S$ trends differ depending on binder types, test temperatures and ageing conditions. The NSRP exhibited a significantly different trend for each Sasobit® content, depending on binder type and ageing conditions.

Keywords: Sasobit®; rheology; ageing; viscosity; rotational viscometer; dynamic shear rheometer

1. Introduction

Warm mix asphalt (WMA) is a relatively new technology, developed in response to the escalating price of crude oil, hence asphalts binders and mixes. Many warm binder additives have been tested, tried and are commercially available in the market. It is therefore necessary to formulate parameters that will enable asphalt technologists to evaluate the performance of asphalt binder blended with warm binder additives. The parameters should be sensitive to variations in ageing conditions, test temperatures and additive content. The parameters can be formulated based on a unit percentage of warm binder additive incorporated in an asphalt binder under various conditions including binder type and source. The parameter is expected to be sensitive to sweep temperatures and hence reflecting the rheological trends of warm modified asphalt binders. These trends can be used to quantify the effects of ageing and various warm binder additive contents on asphalt binder rheological properties. Currently, detailed information on such parameters is not available in the literature. This paper attempts to fill the above gap in knowledge. The new parameters are used to compare and characterise the effects of Sasobit® content on the rheological properties of asphalt binders under unaged and aged conditions at high and intermediate temperatures using Superpave™ asphalt binder tests. Sasobit® is a type of synthetic wax which has been used to produce warm mix asphalt. It is an asphalt binder additive produced by Fischer-Tropsch (FT)

*Corresponding author. Email: cemeor@eng.usm.my

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paraffin wax. The effects of wax depend on its chemical composition, amount of wax, crystalline structure and binder rheological properties (Edwards & Isacsson, 2005a, 2005b).

2. Materials and test programmes

2.1. *Asphalt binder*

The rheological properties of the asphalt binders used in this study are shown in Table 1.

2.2. *Test programmes*

2.2.1. *Ageing state*

All asphalt binder samples were artificially conditioned via the rolling thin film oven (RTFO) and pressure ageing vessel (PAV) in accordance with the procedures outlined in ASTM D2872 (2006) and ASTM D6521 (2006), respectively.

2.2.2. *Dynamic shear rheometer*

The effects of Sasobit® content on the rheological properties of the unaged and aged warm binder samples at intermediate temperatures were investigated in terms of the Superpave™ rutting factor (G*/Sinδ), obtained from the results of the Dynamic Shear Rheometer (DSR) test. In the test, temperature sweeps were applied from 46°C to 82°C at 6°C increments for the unaged and short term aged samples in accordance with Superpave™ requirements (Asphalt Institute, 2001).

2.2.3. *Brookfield rotational viscometer*

The effects of Sasobit® content on the viscosity of unaged and aged asphalt binder samples at elevated temperatures (120°C to 160°C at 10°C increments) were evaluated using a Brookfield rotational viscometer (RV).

3. Result and discussion

3.1. *Effects of Sasobit® content on viscosity*

Viscosity is a fundamental characteristic that describes the resistance of fluids to flow. In practice, it is necessary to ensure that asphalt binder has adequate viscosity to ease pumping and able to coat each aggregate particle during mixing. The relationships between viscosity and Sasobit® content of PG64 and PG70 binders subjected to different temperatures and ageing conditions are shown in Figure 1.

Table 1. Rheological properties of the binders used.

Ageing state	Test properties	PG64 value		PG70 value
Unaged binder	Viscosity at 135°C (mPa.s)	465		575
	G*/Sinδ at 64°C (kPa)	1.23	at 70°C (kPa)	1.04
	Failure temperature (°C)	66		70.04
RTFO aged residue	G*/Sinδ (KPa) at 64°C	2.68	at 70°C (kPa)	2.23
	Failure temperature (°C)	65		71
RTFO+ PAV aged residue	G*/Sinδ at 25°C (kPa)	2958.75		4550

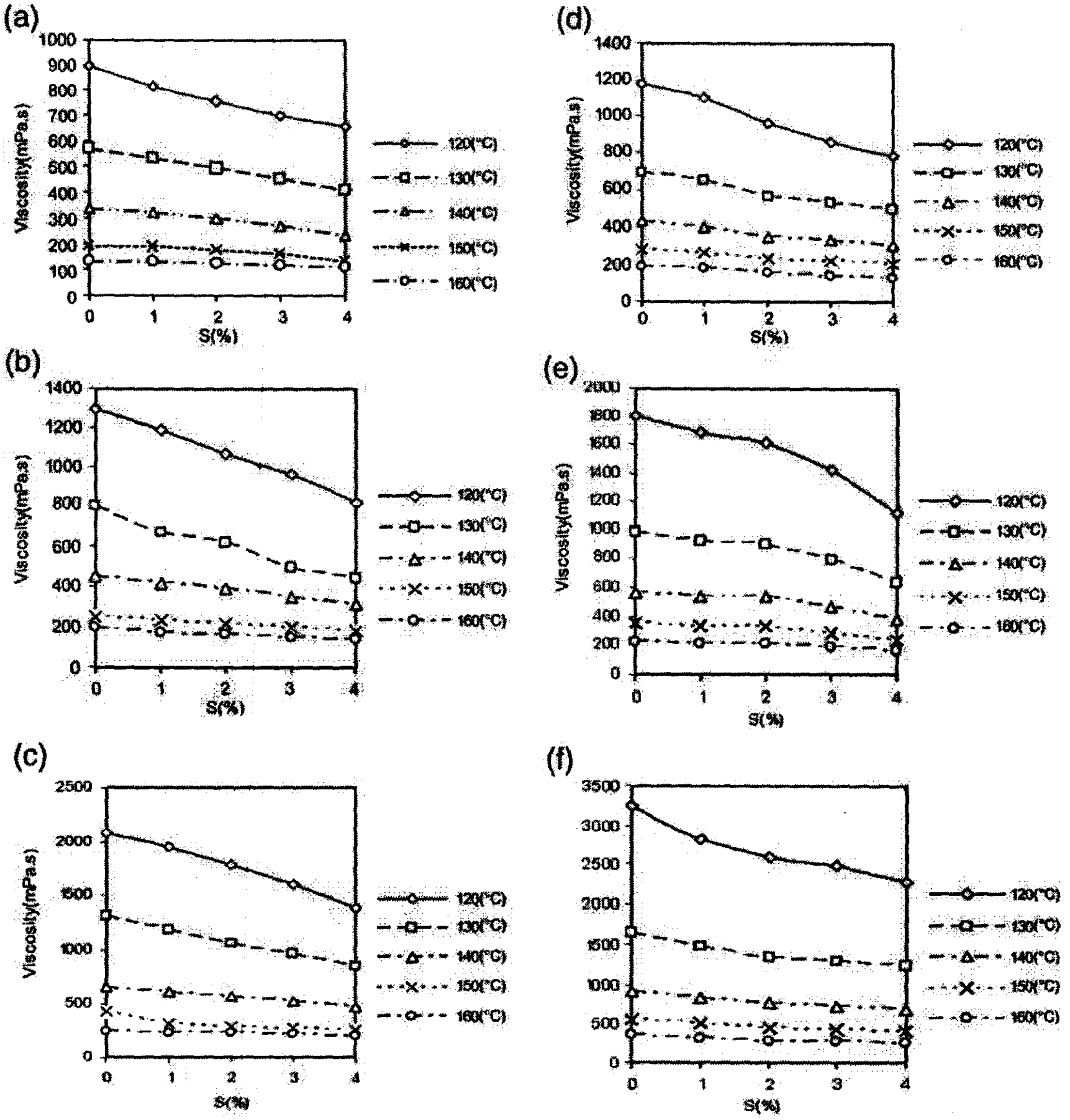

Figure 1. Viscosity–Sasobit® content dependency for (a) unaged PG64, (b) short term aged PG64, (c) long term aged PG64, (d) unaged PG70, (e) short term aged PG70, (f) long term aged PG70.

Figure 1 shows that the addition of Sasobit® linearly decreases the binder viscosity of the asphalt binders tested and similar pattern can be observed for binders that are subjected to short term and long term ageing. This decrease in viscosity is more pronounced when the binders are blended with 3% and 4% Sasobit® and tested at 120°C and 140°C for both binder types regardless of ageing conditions.

To characterise the effects of Sasobit® in reducing the amount of relative viscosity to a unit Sasobit® content (1%) incorporated in asphalt binder at each test temperature, a non-dimensional viscosity gradient ($\nabla\eta_S$) is defined and is expressed in Equations (1) and (2).

$$\eta_S = (\nu/\nu_0) \tag{1}$$

$$\nabla\eta_S = \left[\frac{\partial\eta_S}{\partial S}\right] = \left[\frac{\Delta\eta_S}{\Delta S}\right] = \left[\frac{\eta_{Si+1} - \eta_{Si}}{S_{i+1} - S_i}\right] \tag{2}$$

Table 2. Analysis of variance (ANOVA) for $\nabla\eta_S$.

Variables	DF	Sum of square	Mean of square	F	P	Significant
Model	30	539.45^a	179.820	348.79	0	
A	2	19.91	9.960	19.31	<0.01	Yes
T	4	39.46	9.870	19.13	<0.01	Yes
A^{ast}B*T	8	26.53	3.320	6.43	<0.01	Yes
Error	60	30.93	0.516			
Total	90	5425.40				

Notes: A, ageing; *, interaction; B*, binder type; T, test temperature. $^a R$ squared = 0.99; adjusted R squared = 0.99.

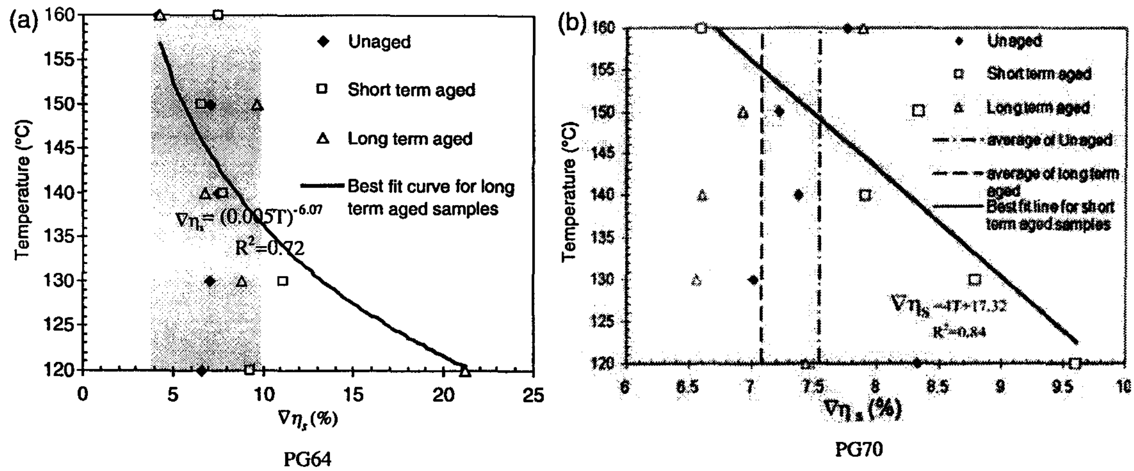

Figure 2. Temperature versus non-dimensional viscosity gradient ($\nabla\eta_S$) relationship.

where η_S is the relative viscosity of the binder sample, S is the Sasobit® content, ν is the binder viscosity and ν_0 is the binder viscosity at initial or control condition. Table 2 shows the summarised analysis of variance (ANOVA) results of $\nabla\eta_S$ as a function of Sasobit® content, binder type, ageing conditions and test temperature.

Table 2 indicates that although $\nabla\eta_S$ is a non-dimensional value, it is sensitive to ageing condition, binder type and test temperature. Therefore, $\nabla\eta_S$ can be used as a parameter to evaluate the rheological trends of Sasobit® modified binder at elevated temperatures. Figure 2 illustrates the relationship between temperature and $\nabla\eta_S$ for each base binder tested. From Figure 2(a), two different trends can be observed for PG64. For the unaged binders, at test temperatures varying from 120°C to 140°C, $\nabla\eta_S$ is approximately 7%. This implies that, within this range of temperature, the relative viscosity of the binder sample reduces by 7% for every 1% Sasobit® content added. At higher temperatures ranging from 150°C to 160°C, the value of $\nabla\eta_S$ reduces to 4.1%.

When subjected to short term ageing, $\nabla\eta_S$ ranges from 7.1% to 9.2%. Figure 2(a) shows that there is no obvious trend for $\nabla\eta_S$ for both unaged and short term aged binders. It can be stated that the values of $\nabla\eta_S$ for both unaged and short term aged samples lie within 4.1% to 9.2% as indicated by the hatched area shown in Figure 2(a). The second trend can be observed on samples subjected to long term ageing. Clearly, $\nabla\eta_S$ can reach as high as 21% at 120°C but reduces as the test temperature increases. From 130°C to 150°C, the value fluctuates from 8.8% to 9.6%. Based on the observed results, set Equation (3) of a multiple function is valid for PG64 binder. From set Equation (3), the trend for each ageing condition is different. This phenomenon can be due to ageing conditions and Sasobit® contents.

$$\nabla\eta_S = f(T,S,A) = \begin{cases} 4.1\% \leq \nabla\eta_S \leq 7\% & \text{(Unaged condition)} \\ 7.1\% \leq \nabla\eta_S \leq 9.2\% & \text{(Short term aged condition)} \\ (0.005T)^{-6.07} \quad R^2 = 0.72 & \text{(Long term aged condition)} \end{cases} \tag{3}$$

where A is the ageing condition, T is test temperature and S is the Sasobit® content.

Figure 2(b) illustrates two trends of $\nabla\eta_S$ for PG70. As shown in Figure 2(b), $\nabla\eta_S$ values are largely scattered around the average values, 7.53% and 7.08% for unaged and long term aged binders, respectively, while a linear trend is observed for short term aged samples. Therefore, set Equation 4 is developed for PG70 binder.

$$\nabla\eta_S = f(T,S,A) = \begin{cases} 7.53 & \text{(Unaged condition)} \\ -0.0648T + 17.32 \quad \mathrm{R}^2 = 0.84 & \text{(Short term aged condition)} \\ 7.08 & \text{(Long term aged condition)} \end{cases} \tag{4}$$

Set Equations (3) and (4) indicate that the observed trends of $\nabla\eta_S$ values for PG64 are markedly different from PG70. For instance, $\nabla\eta_S$ fluctuates at constant value around 7.08 for PG70 subjected to long term ageing, while $\nabla\eta_S$ follows a power trend for PG64 samples at the same ageing state. It can be inferred that the difference stems from the effects of binder types on $\nabla\eta_S$.

3.2. *Effects of Sasobit® content on rutting*

According to Superpave™, rutting is controlled by limiting the G*/Sinδ at a designated test temperature to a value greater than 1.0 kPa and 2.2 kPa for unaged and short term aged binders, respectively (Asphalt Institute, 2001). Table 3 shows the effects of Sasobit® content on the rutting parameter of unaged and short term aged binders tested at various temperatures.

To investigate the effects of Sasobit® content on G*/Sinδ trends, the non-dimensional Superpave™ rutting parameter, as defined in Equation (5), is used.

$$\mathrm{NSRP} = [\mathrm{G}^*/\mathrm{Sin}\delta]_n = \frac{\left[\dfrac{G^*}{Sin\delta}\right]_S}{\left[\dfrac{G^*}{Sin\delta}\right]_C} \tag{5}$$

Table 3. Summary of G*/Sinδ of unaged and short term aged binders at different temperature.

		S (%)									
		PG64					PG70				
	T (°C)	0	1	2	3	4	0	1	2	3	4
Unaged	46	17.77	21.67	25.07	30.60	35.60	30.70	34.20	36.63	45.76	53.20
	58	2.93	3.49	4.09	4.89	5.58	4.97	5.91	6.82	8.80	10.39
	70	0.71	0.9	1.08	1.26	1.43	1.04	1.25	1.53	2.10	2.56
	82	0.18	0.25	0.32	0.39	0.47	0.27	0.32	0.42	0.59	0.77
Short term aged	46	37.18	46.19	54.76	60.45	65.62	52.37	61.36	71.95	88.84	107.5
	58	5.33	6.83	8.11	8.99	9.66	9.33	11.14	12.49	16.76	20.77
	70	1.11	1.45	1.73	1.89	1.98	2.23	2.24	2.70	3.50	4.56
	82	0.29	0.38	0.46	0.52	0.57	0.42	0.49	0.59	0.79	1.08

Table 4. Analysis of variance (ANOVA) for NSRP

Variables	DF	Sum of square	Mean of square	*F*	*P*	Significant
Model	12	1000.39[a]	8.930	1369.18	<0.001	Yes
A	1	0.540	0.540	81.94	<0.001	Yes
B	1	0.750	0.750	115.19	<0.001	Yes
S	3	46.850	15.620	2393.87	<0.001	Yes
T	6	16.740	2.790	427.67	<0.001	Yes
Error	24	1.461	0.007			
Total	36	1001.860				

Notes: A, ageing; B, binder type; S, Sasobit; T, test temperature; * interaction. [a]R squared = 0.99; adjusted R squared = 0.99.

where NSRP is the unitless or non dimensional Superpave™ rutting parameter, subscript n is non-dimensional, $[G^*/\mathrm{Sin}\delta]_S$ identifies Superpave™ rutting parameter of the samples incorporating Sasobit® and $[G^*/\mathrm{Sin}\delta]_C$ is the control sample without Sasobit® or initial condition at each test temperature. The NSRP and its trends change with Sasobit® content and ageing conditions. Table 4 shows the summarised ANOVA results for NSRP.

Table 4 shows that ageing state, binder type and test temperature have significant effects on NSRP. Therefore, NSRP can be adopted as a suitable parameter to study Sasobit® modified binder rheology at different ageing states at intermediate temperature. Using the NSRP results above, Equations (6) and (7) are developed to characterise the rheological properties of PG64.

For the unaged condition:

$$NSRP = \left[G^*/\mathrm{Sin}\,\delta\right]_n = 0.004T + 0.1675S + 0.9084 \quad 46 \le T \le 82 \quad and\, 1 \le S \le 4 \qquad (6)$$

For the short term ageing condition:

$$NSRP = \left[G^*/\mathrm{Sin}\,\delta\right]_n = 0.0041T + 0.293S + 0.77 \quad 46 \le T \le 82 \quad and\, 1 \le S \le 4 \qquad (7)$$

Error averages of the equations are 2.68% and 1% for the unaged and short term aged conditions, respectively. Figure 3 illustrates the minimum and maximum boundaries of NSRP. The minimum boundaries are the same for each ageing condition but the maximum boundary of short term aged samples are lower compared to the unaged samples. The effect of ageing is to decrease the

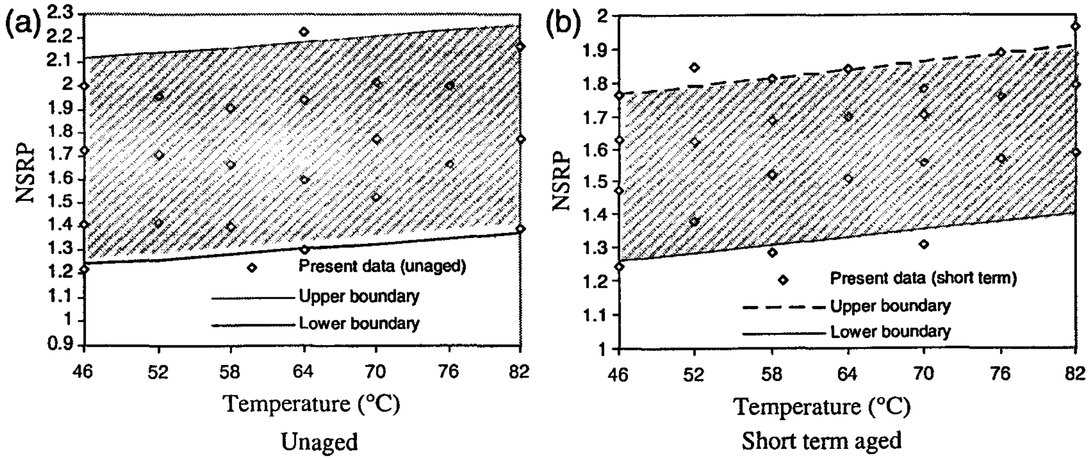

Figure 3. Valid boundaries of NSRP for PG64 binder tested.

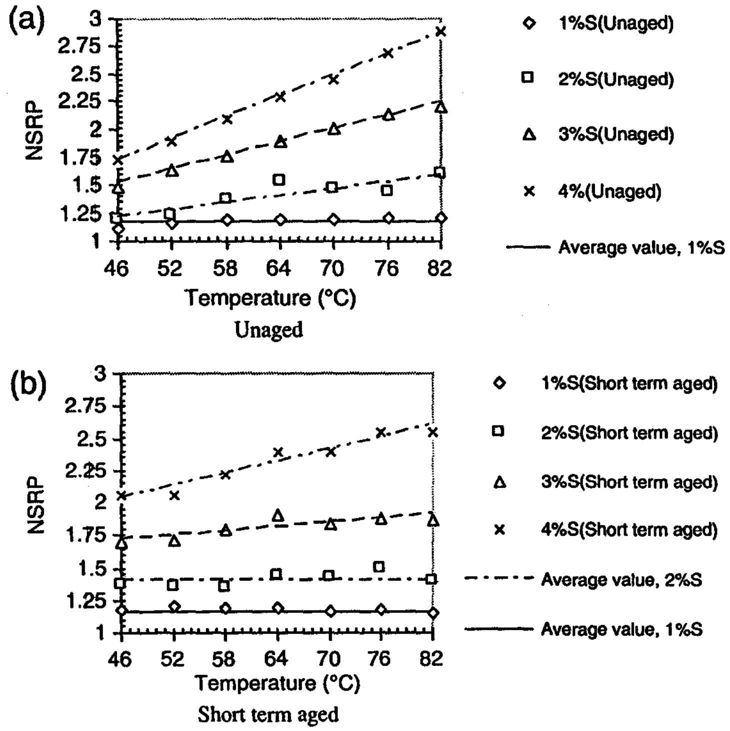

Figure 4. NSRP trends for PG70 binder.

maximum boundary of short term aged PG64 samples by 16.7% in comparison with the maximum boundary of the unaged samples.

Developing relationships similar to Equations (2) and (3) for PG70 is not a straightforward task due to variations in NSRP versus temperature relationships at each ageing condition as illustrated in Figure 4.

From Figure 4(a), the NRSP appears constant at about 1.18 for unaged binders incorporating 1% Sasobit®, while a linear trend is observed for samples with 2%, 3% and 4% Sasobit® at the same ageing state.

For short term aged samples blended with 1% and 2% Sasobit®, the NSRP values appear to fluctuate at a constant value of 1.17 and 1.4, respectively (Figure 4(b)). The increasing trends can be seen for samples with 3% and 4% Sasobit®. Therefore, two sets of Equations 8 and 9 can be developed based on the ageing conditions for PG70.

For the unaged condition:

$$\text{NSRP} = [\text{G}^*/\text{Sin}\,\delta]_n = \begin{cases} 0.03T + 0.23, & S = 4\%, R^2 = 0.9 \\ 0.02T + 0.59, & S = 3\%, R^2 = 0.9 \\ 0.01T + 0.74, & S = 2\%, R^2 = 0.8 \\ 1.18, & S = 1\% \end{cases} \quad 46 \le T \le 82 \tag{8}$$

For short term aged condition:

$$\text{NSRP} = [\text{G}^*/\text{Sin}\,\delta]_n = \begin{cases} 0.016T + 1.31, & S = 4\%, R^2 = 0.94 \\ 0.0005T + 1.48, & S = 3\%, R^2 = 0.68 \\ 1.40, & S = 2\% \\ 1.17, & S = 1\% \end{cases} \quad 46 \le T \le 82 \tag{9}$$

where T is temperature and S denotes Sasobit® content.

Differences in slope of the line and the intercept at the NSRP vertical axis for each equation at each individual ageing condition are parameters that can be used to evaluate the effects of Sasobit® content. For instance, from set Equation (8), the slope of the line for asphalt samples incorporating 3% Sasobit® is two times higher than samples with 2% Sasobit®, and the absolute value of the difference between NSRP intercept is 0.59–0.74 or 0.15. Furthermore, the observed differences in the mathematical relations in set Equations (8) and (9), including the equations and constant values, between unaged and short term aged conditions can be considered as the effects of ageing associated with Sasobit® content for PG 70. The role of binder type on the rheological properties of the asphalt samples can be observed by comparing Equations (6) and (7) with set Equations (8) and (9). It is evident that the observed trends changes when different asphalt sources and binder additives are used.

4. Conclusions

At high temperatures, a non-dimensional viscosity parameter ($\nabla\eta_S$) was developed using rotational viscometer results as a parameter to evaluate the amount of reduction of relative viscosity to a unit Sasobit® content (1%) at each test temperature. For the unaged PG64, $\nabla\eta_S$ is found to be approximately 7% at test temperatures varying from 120°C to 140°C, while it reduces to 4.1% when tested between 150°C to 160°C. $\nabla\eta_S$ values and observed trends are influenced by binder type, ageing and test temperatures.

At intermediate temperatures, the non-dimensional Superpave™ rutting parameter (NSRP) was used to characterise trends of effects of Sasobit® content on binder properties. The NSRP trends were significantly different depending on Sasobit® content, binder type and ageing state. Equations are presented to characterise the NSRP dependency trends to Sasobit® contents for each binder type and ageing condition.

References

ASTM D2872. (2006). *Standard test method for effect of heat and air on moving film of asphalt (rolling thin-film oven test).*

ASTM D6521. (2006). *Standard practice for accelerated aging of asphalt binder using a pressurised aging vessel (PAV).*

Edwards Y., & Isacsson U. (2005a). Wax in bitumen. Part I: Classifications and general aspects. *Journal of Road Materials and Pavement Design*, *6*(3), 281–309.

Edwards Y., & Isacsson U. (2005b). Wax in bitumen. Part II: Characterizations and effects. *Journal of Road Materials and Pavement Design*, *6*(4), 439–468.

Asphalt Institute. (2001). *Superpave performance graded asphalt binder specification and testing* (3rd edn). Superpave Series No. 2 (SP-1). Lexington, KY: Asphalt Institute.

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Effects of Sasobit® on the Required Heat Energy and CO_2 Emission on Blended Asphalt Binder Incorporated With Aged Binder

M.O. Hamzah
School of Civil Engineering, Universiti Sains Malaysia
Engineering Campus, 14300 Nibong Tebal, Pulau Pinang, Malaysia
E-mail: cemeor@eng.usm.my
Tel: +604-5995999; Fax: +604-5941009

A. Jamshidi
School of Civil Engineering, Universiti Sains Malaysia
Engineering Campus, 14300 Nibong Tebal, Pulau Pinang, Malaysia

Z. Shahadan
School of Civil Engineering, Universiti Sains Malaysia
Engineering Campus, 14300 Nibong Tebal, Pulau Pinang, Malaysia

Abstract

Increasing emission of greenhouse gases has worsened the global warming and a great concern to curb this problem from further damage to the environment and living things have captured the attention of many parties. In the asphalt industries, emergence of warm-mix technology has significantly reduced mixing and compaction temperature which consequently lowers the emission from the production process. This paper presents the effects of Sasobit®, one of warm-mix additives, on the required heat energy and CO_2 emission on mixing blended binder between pure and aged binder at different proportions of aged asphalt and Sasobit® content. The viscosity-temperature dependency was also investigated which provided fundamental information on temperature control particularly in determination of mixing and compaction temperatures. In this study, asphalt binder grade 80/100 was used and the binder was subjected to long term aging to simulate recovered asphalt binder from reclaimed asphalt pavement. The result shows that addition of Sasobit® in the blended asphalt binders incorporated with aged binder can reduce mixing and compaction temperatures, the required heat energy and CO_2 emission.

Keywords: Warm-Mix, Reclaimed Asphalt Pavement, Sasobit®, Required Heat Energy, Carbon Dioxide

1. Introduction

Reclaimed asphalt pavement (RAP) is one of the largest recycled materials produced by the asphalt industries. Utilizing reclaimed asphalt pavement or milling waste in asphalt mixes has offered many advantages which includes the preservation of existing road profile and the environment, conservation of asphalt binder and aggregate resources, conservation of energy and reduction in life-cycle cost (Kandhal and Mallick, 1997). Materials present in old hot-mix asphalt pavement still retain considerable value which can be incorporated into the virgin asphalt mixture. Furthermore, several studies have shown that the recycled mixtures have performed equally or better than the virgin mixture (Widyatmoko, 2008; Li et al, 2008; Huang et al, 2005). Inclusion of RAP in an asphalt mixture has somewhat changed the properties of the hot mix asphalt. The change in the recycled asphalt mixture properties is primarily due to the introduction of aged binder as part of the RAP in the asphalt mixture. Consequently, the addition of RAP increased the binder stiffness and decreased its shear strain (McDaniel et al, 2001).

As a current practice in asphalt plants, asphalt mixtures containing RAP material are mixed and compacted at higher temperatures due to the stiffer aged binder in RAP. Fortunately, a new technology known as warm-mix asphalt (WMA) is capable of reducing mixing and compaction temperatures of asphalt mixtures containing RAP. WMA technology utilizes various types of mixture additives of which one of them is Sasobit®. Test results showed that Sasobit® increases rutting parameter ($G^*/\sin\delta$) of binders, as well as decreases binder viscosity and zero shear viscosity (ZSV) regardless of binder sources (Gandhi et al, 2009; Biro et al, 2009; Wasiuddin et al, 2007; Edwards et al, 2007). Mallick, Bradley and Bradbury (2007) showed that Sasobit® can potentially decrease the construction temperature of mixes containing 75% RAP content, yet producing mixes with similar air voids as in virgin mixes. Literature review has shown that no extensive research investigating the effects of different aged binder and Sasobit® contents on emission of CO_2 and energy required for heating asphalt binder containing virgin and aged binders before mixing. This paper addresses the issue in detail.

2. Materials and Methods

2.1. Asphalt Binder

The virgin or conventional asphalt binder used for this study was an 80/100 penetration grade base bitumen (AC80/100) produced by PETRONAS, a Malaysian oil company. Their rheological characteristics showed in Table 1.

Table 1: Rheological characteristics of AC80/100 Binder

Aging State	Test parameters	Value
Unaged	Viscosity at 135°C (m.Pa.s)	465
	G*/Sinδ (kPa) @ 64°C	1.23
	Failure temperature (°C)	66.4
	Specific heat capacity @ 25°C(J/Kg/°C)	920
RTFO aged residue	G*/Sinδ (kPa) @ 64°C	2.68
	Failure temperature (°C)	66
RTFO+PAV aged residue	G*.Sinδ (kPa) @ 25°C	2958.75

2.2. Warm Mix Additive

Sasobit®, a product of Sasol Wax Company, South Africa, was the warm binder additive used. Table 2 reveals the rheological properties of Sasobit®.

Table 2: Rheological characteristics of Sasobit® (Sasol Wax, 2007)

Test parameter	CP	P_{25}	P_{65}	LM	BV_{135}
Maximum	100	-----	-----	75	10
Minimum	----	1	13	115	14

2.3. Tests and Methods

2.3.1. Rotational Viscometer

A Brookfield rotational viscometer (RV) was used to determine the mixing and compaction temperatures of the asphalt sample. The No. 27 spindle was used at rotational speed 6.81/s based on SUPERPAVE™ recommendations (The Asphalt Institute-SP1, 2001). Three readings were recorded for each test while the mean value was recorded as the final result.

2.3.2. Aging Protocol

To evaluate the effects of Sasobit® on the viscosity of aged binder, asphalt binder samples were aged in the laboratory. The samples were conditioned in the rolling thin film oven test (RTFO) and pressure aging vessel in accordance with ASTM D2872 ASTM (2006a) and ASTM D6521 ASTM (2006b), respectively.

2.3.3. Sample Preparation

A propeller mixer was used to blend Sasobit® with binder at 150°C. The Sasobit® was added in proportions of 1%, 3%, and 4% by mass of asphalt binder to modify the conventional asphalt binder. To ascertain the effects of Sasobit® content on the required heat energy in the blended asphalt binder containing pure and aged binders, two scenarios were considered:

- Scenario 1: This scenario considers only unaged Sasobit® modified asphalt binder. Mixes G2, G3 and G4 fall within this scenario while G1 is the control sample.
- Scenario 2: This scenario focuses on Sasobit® modified asphalt binder in aged binder. This scenario refers to mixes G5, G6, and G7.

The scenarios that define combinations of various percentages of unaged and aged asphalt binders incorporating different Sasobit® contents are covered under 7 mix groups and 35 mix types as shown in Table 3.

Table 3: Defined mix types for different content of Sasobit® and aged asphalt binder

Mix Group	Mix Type	Composition			
		(%)[a]	Vb[b] (%)	(%)[c]	RB[d] (%)
Group 1	T11	0	100	0	0
	T12	0	75	0	25
	T13	0	50	0	50
	T14	0	25	0	75
	T15	0	0	0	100
Group 2	T21	1	100	0	0
	T22	1	75	0	25
	T23	1	50	0	50
	T24	1	25	0	75
	T25	1	0	0	100
Group 3	T31	3	100	0	0
	T32	3	75	0	25
	T33	3	50	0	50
	T34	3	25	0	75
	T35	3	0	0	100
Group 4	T41	4	100	0	0
	T42	4	75	0	25
	T43	4	50	0	50
	T44	4	25	0	75
	T45	4	0	0	100
Group 5	T51	0	100	1	0
	T52	0	75	1	25
	T53	0	50	1	50
	T54	0	25	1	75
	T55	0	0	1	100
Group 6	T61	0	100	3	0
	T62	0	75	3	25
	T63	0	50	3	50
	T64	0	25	3	75
	T65	0	0	3	100
Group 7	T71	0	100	4	0
	T72	0	75	4	25
	T73	0	50	4	50
	T74	0	25	4	75
	T75	0	0	4	100

[a]: Sasobit® content in virgin asphalt binder; [b]: Virgin binder content; [c]:Sasobit® content in aged asphalt binder; [d]:Aged binder content

3. Result and Discussion

3.1. Effect of Sasobit® on Reduction of Mixing and Compaction Temperatures

The role of binder during mixing process is to coat the aggregates and produce a homogenous blend. To do so, the asphalt binder should be sufficiently fluidic such that all aggregates are uniformly coated and then strongly adhere the aggregate particles together. According to the Asphalt Institute-SP1 (2001), the recommended mixing and compaction temperatures should respectively correspond to viscosity ranges 170 ± 20 cP and 280 ± 30 cP . Using this as the basis, Table 4 shows the mixing and compaction temperatures of virgin and aged asphalt binders modified by Sasobit®.

Table 4: Reduction of mixing and compaction temperatures of asphalt binder using Sasobit®

Aging State	Sasobit® Content (%)	0 (Pure binder)	1	2	3	4
Unaged Asphalt	Mixing Temperature (°C)	160	155	152	150	145
	Compaction Temperature (°C)	150	145	142	140	137
Long Term Aged (RTFO+PAV)	Mixing Temperature (°C)	175	168	165	160	154
	Compaction Temperature (°C)	158	153	151	150	149

From Table 4, Sasobit® can decrease mixing and compaction temperatures of unaged and aged asphalt binders. For unaged binder without Sasobit®, the mixing and compaction temperatures are 160°C and 150°C, respectively. For Sasobit® modified binder, the lowest mixing temperature is 145°C.

Aging of asphalt binders takes place primarily due to loss of oils (volatilization) and reaction with oxygen in the environment (The Asphalt Institute-MS1, 2001). This phenomenon begins to take place during the mixing process at the asphalt plant and continues in the pavement while in service. The long-term aged asphalt binder possesses high viscosity which would require high temperature, hence requiring more energy input to attain the desired mixing and compaction viscosities. The results show that the mixing temperature of aged asphalt binder sample modified by 3% Sasobit® is equal to the mixing temperature of unaged pure asphalt binder, while 4% Sasobit® makes the aged asphalt binder softer than the unaged pure asphalt binder.

3.2. Required Heat Energy for Asphalt Binder Containing Unaged and Aged Binder

In order to calculate the required heat energy to heat up pure binder and aged binder, the total mass of asphalt binder to pave an assumed road carriageway is determined. From the mix design results, the optimum binder content and mix density were respectively 5.0% and 2.38 g/cm^3. Therefore, the mass of asphalt binder needed to pave a 10 km dual carriageway with 3 lanes per direction and 5 cm thick wearing course is 1350 tons.

The amount of heat energy is a function of aged binder and Sasobit® contents. The effects of different percentages of aged binder with Sasobit® are investigated by quantifying heat energy required to heat up asphalt binder to mixing temperature using Equation 1.

$$Q = \sum_{i=n}^{j=n-1} mc\Delta\theta \qquad (1)$$

where Q is the sum of heat energy (J), $\Delta\theta$ is the difference between the ambient and mixing temperatures (°C), c is specific heat capacity coefficient (J/(kg/°C)), m is the mass of material (kg), and i and j indicate different material types.

Two parameters are used to study and compare the trends observed from the heat energy of the scenarios described in Section 2.3.3, namely energy difference parameter (EDP) and energy gradient to aged binder $(\partial E/\partial R)$. The EDP can be expressed as in Equation 2.

$$EDP = \Delta E_i = E_{M_i} - E_C \qquad (2)$$

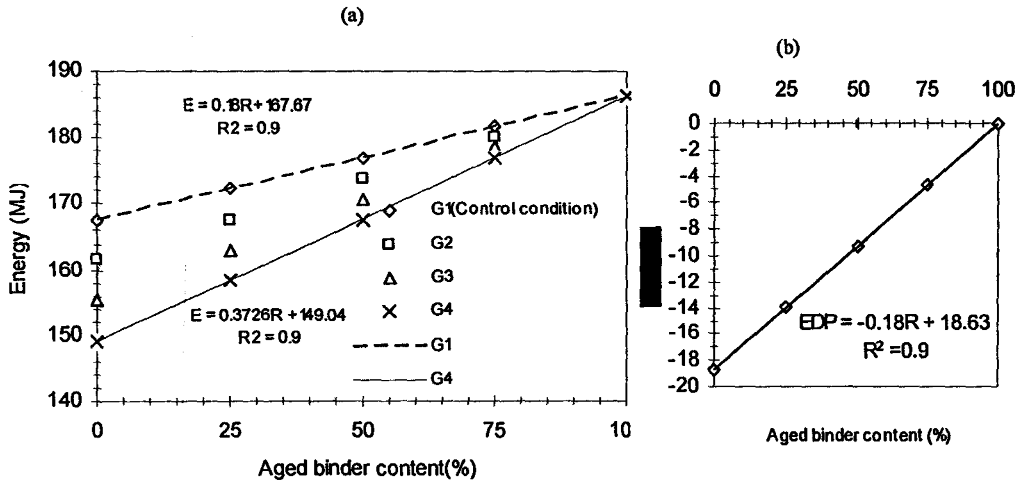
where E_{M_i} is the energy required to raise the temperature from ambient to mixing temperature for a particular mix type (Table 4), while E_C is the energy required to heat up the control mix (G1) containing the same percentage of aged binder without Sasobit®.

The purpose of EDP is to investigate the required heat energy of mix types containing the same aged binder content but modified by different Sasobit® contents. The energy gradient to aged binder $(\partial E/\partial R)$ is the slope of the linear relationship between the required heat energy and aged binder content. This parameter is defined to evaluate the heat energy versus aged binder content dependency of each mix group.

3.3. Scenario 1: Incorporation of Sasobit® in Unaged Binder

Fig 1(a) shows the trend of energy required in scenario 1 where several percentages of Sasobit® were blended in unaged binder. It can be seen that the EDP has decreased from 47.6 MJ at zero percent aged binder to 0 MJ at 100 percent aged binder when unaged binder incorporated with 4% Sasobit®. As the percentage of aged binder increases, the energy required increases as well until unaged binder with all percentages Sasobit® contents converge to a same energy required at 100% aged binder content. Furthermore, $(\partial E/_{\partial R})$ has increased two times in G4 which indicates that Sasobit® reduces the required heat energy for samples without aged binder, while heat energy input is subsequently increased as the aged binder content increases. However, the magnitude of energy required is less compared to the control group. This phenomenon is further illustrated in Fig 1(b) the EDP increases as the percentage of aged binder content increases.

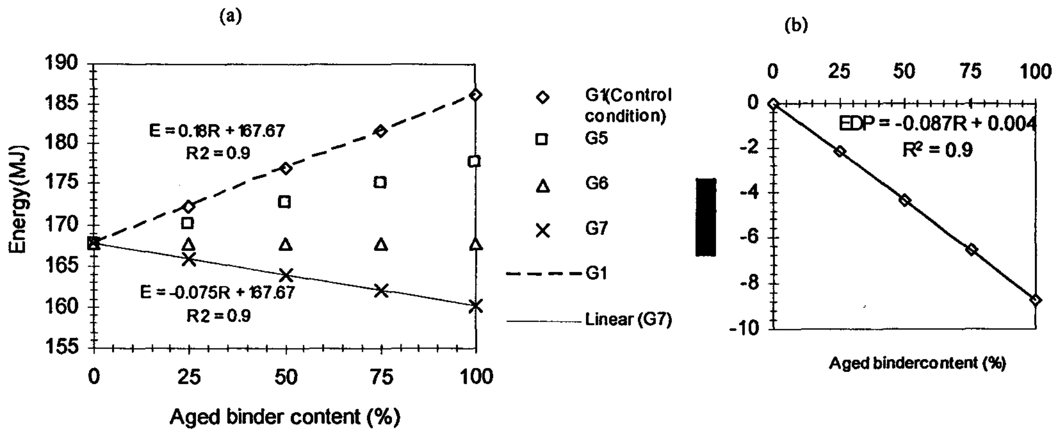
Figure 1: Effects of aged binder contents under scenario 1- (a) Heat energy versus aged binder content (b) Energy difference parameter versus aged binder content



3.4. Scenario 2: Incorporation of Sasobit® in Aged Binder

Scenario 2 shows the decreasing trend of energy required which contradict to the trend in scenario 1. It can be seen that in Fig 2(a), the energy lines are diverging downward as the aged binder content increases, the EDP has decreased to -21.8 MJ at 100% aged binder content containing 4% Sasobit® modified unaged binder. In other words, the effect of Sasobit® in reducing heat energy required is more pronounced when the aged binder contained Sasobit® to the extent that the aged binder is softer than the pure binder. Fig 2(a) also indicates that the presence of Sasobit® decreases the $\partial E/_{\partial R}$ to approximately zero in G6. This takes place because the presence of 3% Sasobit® in aged binder makes the aged binder as soft as the unaged asphalt binder without Sasobit®. From Table 4, the 3% Sasobit® modified aged asphalt binder has the same mixing temperature at 160°C. Figure 2(b) exhibits the magnitude of 6 energy saved as the aged binder content increases.

Figure 2: Effects of aged binder contents under scenario 2 - (a) Mixing energy versus aged binder content (b) Energy difference parameter versus aged binder content



3.5. Effects of Aged Binder Content on CO_2 Emission

Figures 3 and 4 illustrate the effects of various aged binder contents on CO_2 emission for different scenarios and fuel types used in an asphalt plant. It can be seen from the figures that natural gas contributes to the highest reduction of CO_2 emission especially when the aged binder incorporates Sasobit®.

Figure 3: Effects of aged binder content on CO_2 emission for scenario 1

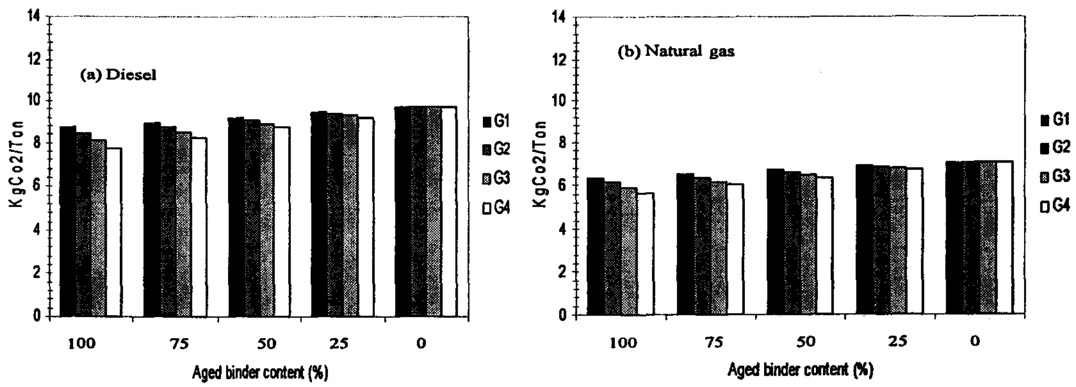
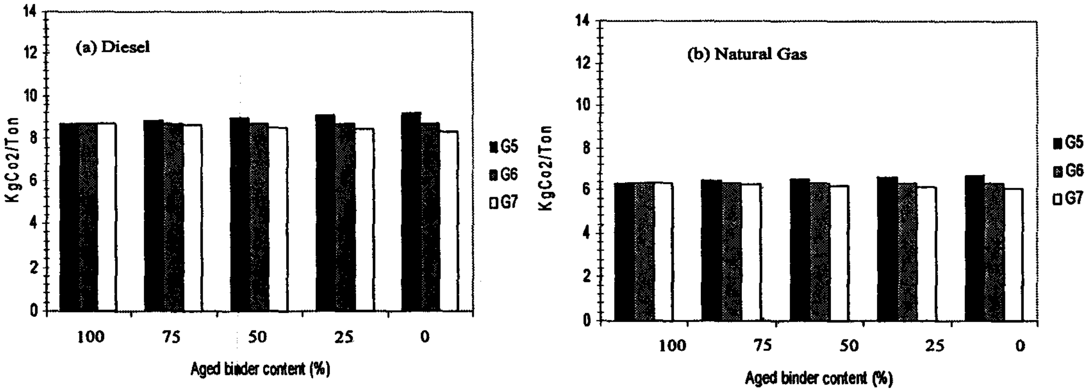


Figure 4: Effects of aged binder content on CO_2 emission for scenario 2



Analysis of the results from these two scenarios has shown that Sasobit® can reduce the required heat energy irrespective of aged binder content. In addition to that, use of Sasobit® helps to conserve the environment and contributes to cleaner technologies. Although aged binders modified by Sasobit® exhibit lower heat energy to raise from ambient to mixing temperatures, it should be noted that these modified binders can be brittle and stiff at low and intermediate temperatures due to synergistic effects of long-term aging and the presence of Sasobit®. To offset the increase of binder stiffness caused by WMA additives and aged binders, Lee et al. (2009) recommended the use of lower performance grade virgin binder.

4. Conclusion

The following conclusions can be drawn from this study;

1. Addition of Sasobit® in the pure and blended asphalt binders has significantly decreased mixing and compaction temperatures.
2. Sasobit® decreases the required heat energy linearly for blended asphalt binder incorporated with aged binder irrespective of aged binder content.
3. In scenario 1, as the percentage of aged binder increases, the required energy increases as well. The EDP decreases linearly as the aged binder content increases.
4. Sasobit® can turn an aged asphalt binder to become as soft as or even softer than an unaged binder as shown in scenario 2 where the energy lines are diverging downwards as the aged binder content increases.
5. The effect of Sasobit® in reducing the heat energy required is more pronounced when the aged binder contained Sasobit®.
6. Sasobit® can significantly decrease the emitted CO_2 as a result of heating up blended asphalt binder incorporated with aged binder during the mixing process.

References

[1] ASTM (2006a). ASTM D 2872 Standard Test Method for Effect of Heat and Air on Moving Film of Asphalt (Rolling Thin-Film Oven Test), West Conshoken, Pa, USA

[2] ASTM (2006b). ASTM D 2872 Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV), West Conshoken, Pa, USA.

[3] Biro S, Gandhi T, Amirkhanian S.J. Determination of zero shear viscosity of warm asphalt binders, Journal of Construction and Building Material 2009;23(5):2080-2086.

[4] Edwards Y, Tasdemir Y, Isacsson U. Rheological effects of commercial waxes and polyphosphoric acid in bitumen 160/220 – High and medium temperature performance, Journal of Construction and Building Material 2007; 21(10):1899-1908.

[5] Gandhi T.S, Akisetty C, Amirkhanian S.J. Laboratory evaluation of warm mix binder aging characteristics, International Journal of Pavement Engineering 2009;10(5):353-59.

[6] Huang, B., Li, G., Vukosavljevic, D., Shu,X., and Egan, B.K., 2005, Laboratory investigation of mixing hot-mix with reclaimed asphalt pavement, Journal of the Transportation Research Board No. 1929, pp 37-45

[7] Kandhal, P.S., and R. Mallick,"Pavement Recycling Guidelines for State and Local Governments", FHWA, U.S. Department of Transportation, 1997

[8] Lee S.J, Amirkhanian S.J, N.W. Park, K.W. Kim, Characterization of warm mix asphalt binders containing artificially long-term aged binders, Journal of Construction and Building Material 2009;23(6):2371-79

[9] Li, X., Marasteanu, M.O., Aliiliams, R.C., and Clyne, T.R.,(2008). Effect of reclaimed asphalt pavement (proposition and type) and Binder Grade on asphalt Mixture. Journal of the Transportation Research Board, No. 2051, Transportation Research Board of the National Academies, Washington D.C., pp 90-97

[10] Mallick R.B, Bradley J.E, Bradbury R.L. Evaluation of Heated Reclaimed Asphalt Pavement Material and Wax-Modified Asphalt for Use in Recycled Hot-Mix Asphalt. Journal of Transportation Research Board, 2007; 1998(14);112-122.

[11] McDaniel, R.S., Soleymani, H.R., Anderson, R.M., Turner, P., Peterson, R.L., Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method. NCHRP Project 9-12, Transportation Research Board, National Research Council, Washington, D.C, 2001

[12] Sasol Wax, 2007, Product data sheet, Division of Sasol chemical industries limited.

[13] The Asphalt Institute-MS1, Mix Design Method, MS-1.The Asphalt Institute, Lexington (KY), USA, 2001

[14] The Asphalt Institute-SP1, Superpave Performance Graded Asphalt, Binder Specifications and Testing, SP-1, The Asphalt Institute, Lexington (KY), USA, 2001

[15] Wasiuddin N.M, Selvamohan S, Zaman M.M, Guegan M.LT.A. Comparative laboratory study of Sasobit® and aspha-min additives in warm mix asphalt. Journal of Transportation Research Board. 2007;10 (1998):82-88.

[16] Widyatmoko. I., (2008), Mechanistic-empirical mixture design for hot mix asphalt pavement recycling. Construction and Building Materials 22, pp 77-87

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Effects of Aging on the Physical, Rheological and Chemical Properties of Virgin Bitumen Incorporating Recovered Reclaimed Asphalt Pavement Binder

Meor Othman Hamzah and Zulkurnain Shahadan

School of Civil Engineering, Engineering Campus, Universiti Sains Malaysia, 14300 Nibong Tebal, Penang, MALAYSIA.

Abstract: This paper focuses on a physico-chemical analysis on virgin bitumen incorporating recovered binder from reclaimed asphalt pavement (RAP) subjected to various aging conditions. The recovered binders extracted from three sources were blended with virgin bitumen in two proportions, namely 15% and 30%, by mass of total bitumen. Penetration, softening point and viscosity values were measured to characterize the physical and rheological properties of the RAP modified binders. The evolution in RAP modified binder chemistry before and after aging process was determined by using the Fourier Transform Infrared Spectroscopy. The penetration and softening point consistently decrease and increase, respectively at each level of oxidation. The penetration index and viscosity aging index increase as the RAP modified binders were further aged. The process of aging has chemically altered the structure of the RAP modified binders. This chemical change has produced a distinct increase in area ratio especially at 1700 cm^{-1} and 1030 cm^{-1} wavelengths which were dominated by C=O and S=O functional groups, respectively. There is a significant correlation between penetration index with viscosity aging index and area ratio of RAP modified binders.

Key words: Asphalt, Aging, FTIR, RAP modified binder, Recovered binder

INTRODUCTION

Nowadays the use of reclaimed asphalt pavement (RAP) as secondary material in the production of asphalt mixes has become a norm and a cost effective method of pavement construction and rehabilitation. Utilizing reclaimed asphalt pavement is found to be very beneficial from the technical, economical, and environmental perspectives. Some of the advantages include reduce waste, preservation of the existing pavement geometrics and conservation of energy and reduction in life-cycle cost. Many laboratory and field studies have shown that asphalt mixtures containing RAP performed similar if not better than conventional asphalt materials in terms of indirect tensile strength, moisture susceptibility, permanent deformation and fatigue (Su *et al.*, 2009; Widyatmoko, 2008).

During mixing, RAP materials are heated and blended at high temperature with hot virgin bitumen and fresh aggregates. It is expected that the highly oxidized RAP binder will melt off from RAP aggregate and intimately blended with the virgin bitumen and fresh aggregates. During this blending process, the virgin bitumen is supposed to rejuvenate the RAP binder such that the resultant binder meets the target viscosity. However, the chemical change that takes place in the RAP binder and virgin binder blends after mixing in-plant and during pavement service life is very much unknown. This is particularly of great concern since the RAP binder is known to be readily oxidized and the mixing process further aged the RAP binder. Under extreme aging conditions, even conventional binder is prone to lose its binding capacity. Subsequently it becomes less adhesive but more cohesive, and make it increasingly brittle (Valcke *et al.*, 2009). The oxidation of binder further contributes to change in the structural and functional grouping that is responsible for chemical and physical aging (Lamontagne *et al.*, 2001). A study by Zhang *et al.*, (2011) found that after the short-term aging, the proportion of bitumen compounds such as asphaltene and resins were increased. Further aging the bitumen by subjecting to Pressure Aging Vessel (PAV), caused the asphaltenes and resins content to continually increase while the saturates content remained constant. Apparently, severe oxidation on bitumen produces more asphaltenes which are present in the micelle form in a colloidal structure of bitumen, directly influencing physical, rheological and chemical properties of the bitumen (Lu X, and U. Isacsson, 2002; Lesueur D., 2009; Le Guern *et al.*, 2010). Fourier Transform Infrared Spectroscopy (FTIR) is an excellent and popular

Corresponding Author: Meor Othman Hamzah, School of Civil Engineering, Engineering Campus, Universiti Sains Malaysia, 14300 Nibong Tebal, Penang, MALAYSIA.
E-mail: cemeor@yahoo.com.

tool to identify chemical evolution in bitumen and able to indicate the severity of oxidation experienced by the bitumen after aging. It has been a major analytical technique to study aging mechanism in asphalt through the characterisation of oxygen-containing functional and hydrocarbon groups. The FTIR can also yield quick qualitative and quantitative results that are highly reproducible. This technique can easily differentiate stretching vibration of carbonyl mode which is largely dominated by asphaltenes compound after the aging process (Toteva *et al.*, 2009).

There appear to be a gap in the literature on studies characterising the physico-chemical of RAP modified binder resulted from blending of virgin and recovered binder from RAP materials subjected to short term and long term aging. Bridging the gap is essential to help understand better the aging effects especially on the evolution of RAP modified binder chemistry. This paper focuses on three RAP binders extracted from cold milled RAP obtained from three major road authorities in Malaysia. The characteristics of RAP modified binders before and after aging are investigated in terms of penetration, softening point, viscosity, and chemical properties.

2. Materials and Test Procedures:

2.1 Materials:

Milling waste of aged and deteriorated pavements from the Malaysian North South Expressway (NSE), Damansara Puchong Expressway (DPE), and Public Work Department (PWD) roads were used in this study. RAP samples from these three sites were sent to IKRAM Sdn. Bhd. for binder extraction to obtain approximately 400 gram of recovered binder (RAP binder) from each RAP source. A conventional virgin binder grade 80/100 (PG64) supplied by PETRONAS, a Malaysian Oil Company was used as the base binder. The virgin binder was blended with RAP binder in quantities of 15% and 30% of recovered RAP binder by mass of total bitumen at 140°C. Table 1 shows the physical and rheological properties of the virgin and RAP binders.

Table 1: Physical and rheological properties of virgin and recovered binders

Binder	Penetration at 25°C (dmm)	Softening point °C	Viscosity at 135°C (Pa.s)	$G^*/\sin\delta$ at 64°C (kPa)
Virgin	90	46.0	0.34	1.14
NSE	14	67.5	3.06	38.0
DPE	13	67.5	4.33	36.1
PWD	11	72.0	2.14	47.8

2.2 Binder Aging Protocol:

The RAP modified binders were subjected to short term and long term aging. The short-term aging, which simulated aging during construction was achieved by using the Rolling Thin Film Oven (RTFO) Test according to ASTM D 2872 (2006) procedures. Binders were aged at 163°C for 85 minutes while 4,000 ml/min of hot air was blown into the rotating bottles lined by the bitumen.

Long-term aging, which simulated field aging in the first 5 to 10 years of pavement service was achieved using the Pressure Aging Vessel (PAV) according to procedures outlined in ASTM D6521 (2006). The RTFO aged binders were placed in the PAV chamber at 100°C and a pressure of 2.1 MPa was applied to the binders for 20 hours.

2.3 Penetration and Ring and Ball Tests:

The penetration test provides a measure of the consistency or hardness of the bitumen. In this test, a needle of specified dimensions was allowed to penetrate a sample of bitumen, under a 100 g load at 25°C temperature for 5 seconds as outlined in ASTM D5 (2006).

In the Ring and Ball test, a standard 3.5 g steel ball was placed onto a sample of bitumen confined in a brass ring that was suspended in a water bath. The water bath temperature was raised at 5°C per minute, the bitumen softened and eventually deformed slowly with the ball moving through the ring. At the moment the bitumen and the steel ball touch a base plate 25 mm below the ring, the temperature was recorded. This temperature was designated as the softening point of the bitumen and represents an equi-viscous temperature. The test was carried out to conformed ASTM D36 (2006).

2.4 Viscosity Test:

A Brookfield Viscometer was employed to measure the viscosities of RAP modified binders according to ASTM D4402 (2006). The test operating speed of the rotational viscometer was set to 20 rpm. The temperature controller of the thermo-chamber was set at 135°C.

2.5 Fourier Transform Infrared Spectroscopy:

A FTIR spectrometer, PerkinElmer model SpectrumOne, was used to determine the functional characteristics of RAP modified binders before and after ageing. All spectra were obtained by 32 scans with 5% iris and 4 cm^{-1} resolution in wavelengths ranging from 4000 to 550 cm^{-1}.

The FTIR spectroscopy allows analyzing functional and structural changes in the fraction of binders due to severe oxidation process by the RTFO and PAV tests. The area of the peak was determined by using the baseline method. The area ratio (AR) for carbonyl and sulfoxide compounds was calculated as in Equations (1) and (2) (Siddiqui and Ali, 1999). The area of particular mode of vibration was measured from valley to valley of the peak by using Spectrum version 5.0.1 software supplied by PerkinElmer™ Instruments.

$$AR_{C=O} = \frac{Area\ of\ the\ carbonyl\ band\ centered\ around\ 1700\ cm^{-1}}{Area\ of\ the\ CH_3\ centered\ around\ 2954\ cm^{-1}} \tag{1}$$

$$AR_{S=O} = \frac{Area\ of\ sulfoxide\ band\ centered\ around\ 1030\ cm^{-1}}{Area\ of\ the\ CH_3\ centered\ around\ 1376\ cm^{-1}} \tag{2}$$

2.7 RAP Modified Binder Designation and Characterising Flow Chart:

A designation system summarised in Table 2 was adopted for easy reference. The RAP modified binder designation consisted of 4 alphanumeric characters in which the first alphanumeric character is an alphabet followed by two numbers or values, and ends with an alphabet. For instance, a designation N15R refers to virgin binder blended with 15% recovered binder from NSE that has been subjected to RTFO test or short term aging. Further detailed explanation of the designation is available in Table 2.

Table 2: RAP modified binder designation

Description	First Alphabet	Number/Value	Last Alphabet
Possible Alphabet/Number	N/D/P	15/30	U/R/P
Denotation	The letter refers to the source of the RAP binder as follows; N – NSE D – DPE P – PWD	The figures reflect percentage of recovered RAP binder blended with virgin bitumen by mass of total bitumen, namely 15% and 30%	The last letter indicates the aging process the sample has been subjected to; U – Unaged R – RTFO P – RTFO+PAV

Figure 1 shows a flow chart to characterize the physical, rheological and chemical properties of the RAP modified binders after subjected various aging conditions.

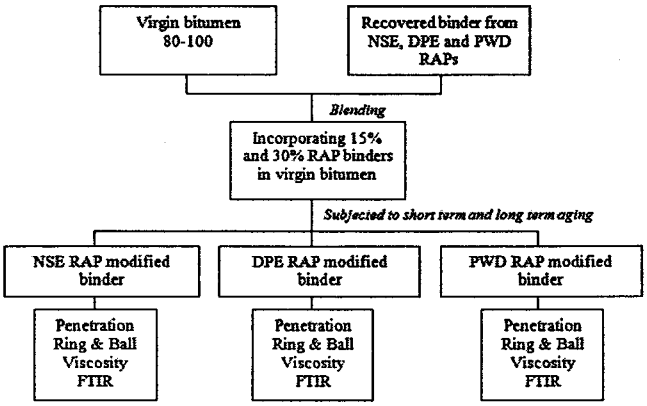


Fig. 1: A flow chart to characterize the physical, rheological and chemical properties of RAP modified binders

RESULT AND DISCUSSION

3.1 Penetration:

Figure 2 shows the penetration values of RAP modified binder after subjected to short term and long term aging. The penetration of RAP modified binder decreases as the RAP modified binders were further aged.

The hardening trend of RAP modified binders is consistent with all RAP binder sources and proportions. For the respective RAP binder proportions, the penetration values of D15 and D30 RAP modified binders after the long term aging are 28 dmm and 24 dmm respectively, and are the lowest.

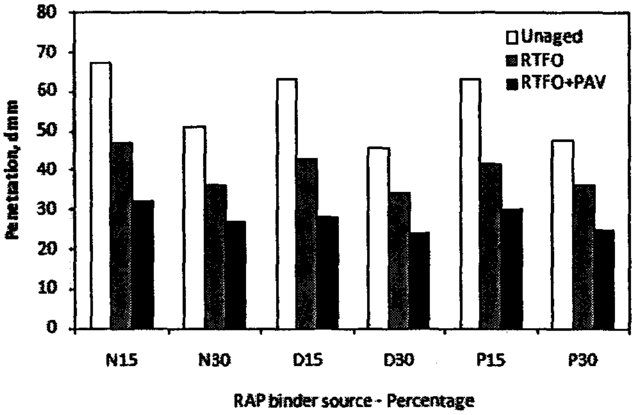


Fig. 2: Penetration of RAP modified binder under different aging conditions

3.2 Softening Point:

The softening point of all RAP binder sources and proportions increase as they undergo aging as shown in Figure 3. For the respective RAP binder proportions, the D15 and D30 RAP modified binders exhibit the highest softening point at 64°C and 67°C, respectively after subjected to long term aging.

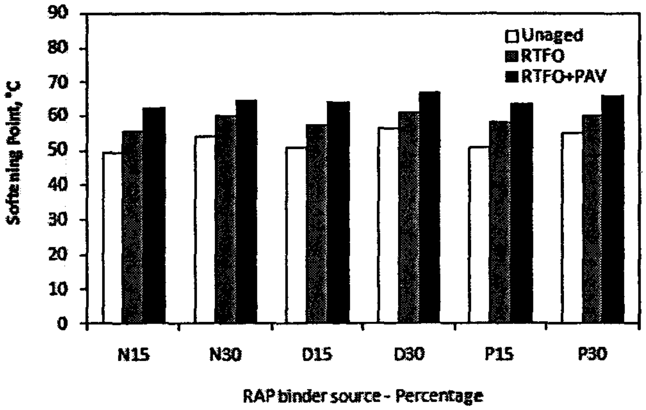


Fig. 3: Softening point of RAP modified binder under different aging conditions

Table 3 shows the percentage decrease and increase of penetration and softening point values, respectively of RAP modified binders after the short term and long term aging. It can be seen that the percentage decrease of penetration from unaged to short term and long term is nearly doubled for all RAP modified binders. Similar trend is also observed for percentage increase of softening point for all RAP binders. It is interesting to note that for RAP modified binder N15P, P15P, D30R, P30R, N30P, D30P and P30P have similar magnitude of percentage decrease within their RAP binder proportion groups and aging conditions. This relates well with similar magnitude of percentage increase in softening point in the groups. The effect of doubling the RAP binder content is to slightly reduce the percentage decrease and increase in penetration and softening point, respectively.

3.3 Viscosity:

Viscosity is a measure of a fluid's resistance to flow. Figure 4 shows consistent increase in RAP modified binder viscosity with percentage of RAP binder as well as level of oxidation increase regardless of RAP binder sources. The N15U and P15U, and N30U and P30U RAP modified binders exhibit similar viscosity values of 0.45 Pa.s and 0.58 Pa.s, respectively. The viscosities of the D15P and D30P samples are the highest which corresponds to the most viscous binders among the RAP modified binders tested. The N15P and P30P RAP modified binders exhibit 61.5% and 60.8% increase respectively and are the highest in their groups. However, P15R and P30R have the lowest viscosity increments at 35.7% and 34.8%, respectively.

Table 3: Percentage change in penetration and softening point of modified RAP binders

RAP Modified Binder	Percentage Decrease in Penetration	Percentage Increase In Softening Point
N15R	29.9	10.8
N15P	52.2	20.2
N30R	29.4	10
N30P	47.1	16.3
D15R	31.7	11.4
D15P	55.6	21.1
D30R	26.1	8.2
D30P	47.8	16.4
P15R	33.3	12.9
P15P	52.4	20.5
P30R	25	8.3
P30P	47.9	16.7

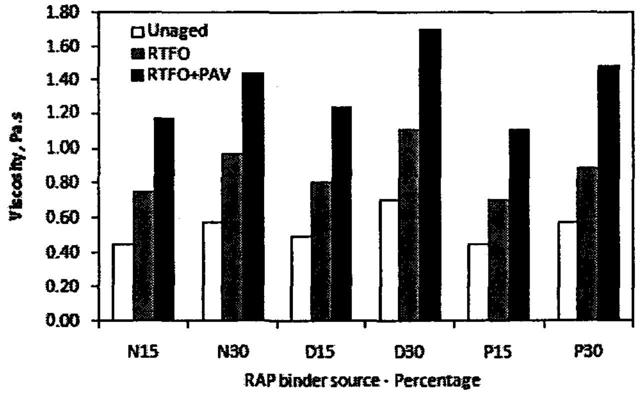


Fig. 4: Viscosity of RAP modified binder under different aging conditions at 135°C

3.4 Penetration Index:

The penetration index (PI) is a measure bitumen susceptibility to temperature and is calculated using Equation 3 (Read and Whiteoak, 2003). The PI for all modified RAP binder lies between +1 and -1 as depicted in Table 4, which is within the PI range for conventional bitumen (Roberts *et al.,* 1991).

$$PI = \frac{1952 - 500 \log pen - 20\, SP}{50 \log pen - SP - 120} \tag{3}$$

where pen = penetration at 25°C
SP = softening point

The table also indicates that as RAP modified binders are further aged, the penetration index is also increased. This means that the RAP modified binders are less temperature susceptible. It can be seen that for the respective RAP binder proportions, P15P and D30P RAP modified binders exhibit higher penetration index at 0.52 and 0.66 respectively, after long term aging.

3.5 Viscosity Aging Index:

The viscosity aging index is defined by Equation (4). Table 4 also shows the viscosity aging index of the RAP modified binders from the three sources after short term and long term aging.

$$VAI = \frac{Viscosity\ of\ aged\ RAP\ modified\ binder}{Viscosity\ of\ unaged\ RAP\ modified\ binder} \tag{4}$$

It can be seen that for the respective RAP binder proportions, N15P and P30P RAP modified binders exhibit higher viscosity aging index at 2.60 and 2.55 respectively, after subjected to long term aging. This high rate of hardening is contributed to the increasing asphaltene compounds in the modified RAP binder due to severe oxidative process

Table 4: Penetration index and viscosity aging index of RAP modified binders

Modified RAP Binder	Penetration Index	Viscosity Aging Index
N15R	-0.06	1.67
N15P	0.38	2.6
N30R	0.26	1.67
N30P	0.48	2.46
D15R	0.05	1.63
D15P	0.47	2.51
D30R	0.33	1.59
D30P	0.66	2.43
P15R	0.21	1.55
P15P	0.52	2.47
P30R	0.26	1.53
P30P	0.58	2.55

3.6 Fourier Transform Infrared Spectroscopy:

During the artificial aging processes, the modified RAP binders were severely oxidized in which aromatization, dehydrogenation and intermolecular and intramolecular hydrogen bonding of polar groups were substantially increased. The chemical functional and structural changes are precisely analyzed through infrared at vibration modes of C=O and S=O. Figure 5 illustrates the infrared spectra of the RAP modified binders for the three RAP sources. The figures clearly show the increasing trend of spectra at carbonyl and sulfoxide groups with increased RAP binder content and aging period as well as aging condition. It can be observed that the absorbance value increases at every level of oxidation at carbonyl and sulfoxide regions.

3.6.1 Carbonyl Groups of Modified RAP Binders:

Carbonyl and/or carboxyl groups were detected by IR spectra with the presence of distinct C=O absorption at 1700 cm^{-1} wavelength. The area of the carbonyl absorption was measured from valley to valley of the peak between 1726 and 1675 cm^{-1} by using Spectrum version 5.0.1 software, which corresponds to the region containing the absorption peaks for carboxylic acid, ketones and anhydrides. Carboxylic acids occur naturally in bitumen while ketones and anhydrides form on oxidative aging. The oxidative aging is well related to the existence of these three functional groups which are an integral part of large asphalt molecules (Siddiqui and Ali, 1999).

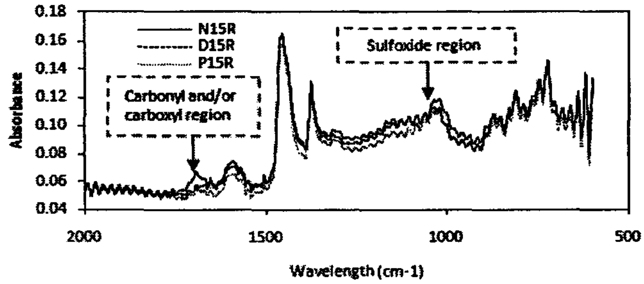
Table 5 shows the area ratio of unaged and aged RAP modified binders. It can be seen that the area ratio of all carbonyl groups increased consistently in both short term aging and long term aging for the three RAP sources. The D15P and P30P RAP modified binders exhibit the highest ratio at 0.124 and 0.175 after long term aging, respectively. The unaged RAP modified binder, N15U and P15U have the lowest C=O area ratio value. On the contrary, the area ratio of N15U and P15U increased by 63.1% and 58.2%, respectively after long term aging, representing the higher percent increase in area ratio among the RAP modified binders tested. This indicates that the evolution of RAP modified binder structures has taken place after extreme aging conditions. During that period, the amount of asphaltene compound has increased due to oxygenation of resin. During the aging process oxygen was being taken up by asphaltene molecular structure. This is similar to the findings of Siddiqui and Ali, (1999) where there was an increase in the percent weight of oxygen in asphaltenes molecules which indicate insertion of substantial amount of oxygen in the asphaltenes after each level of oxidation. Furthermore, the long term aging is attributed to a large quantity of oxygen incorporated in the newly formed oxygen-containing groups such as hydroxyl, carbonyl, and carboxylic groups. The consistent increase in area ratio correlates well to the increase in the level of oxidation which caused a pronounced increase C=O in the carbonyl region.

Table 5: Area ratio of RAP modified binders before and after aging

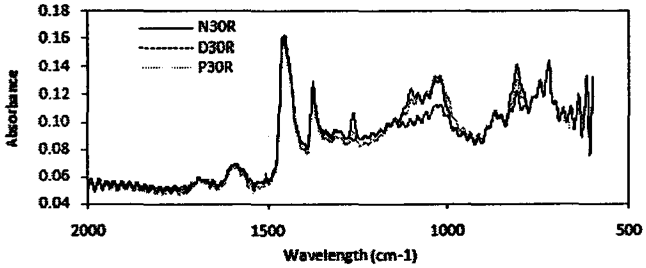
RAP Modified binder	Carbonyl	Sulfoxide
	C=O (1700 cm^{-1})	S=O (1030 cm^{-1})
N15U	0.041	0.51
N15R	0.046	0.54
N15P	0.111	0.55
N30U	0.109	0.54
N30R	0.127	0.543
N30P	0.14	0.56

Table 5: Continue

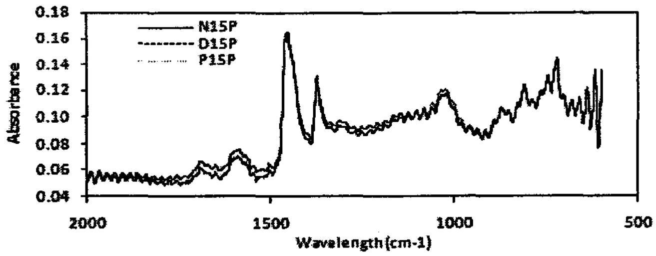
D15U	0.1	0.514
D15R	0.11	0.551
D15P	0.124	0.553
D30U	0.125	0.564
D30R	0.132	0.582
D30P	0.149	0.603
P15U	0.041	0.535
P15R	0.047	0.539
P15P	0.098	0.599
P30U	0.108	0.597
P30R	0.12	0.616
P30P	0.175	0.664



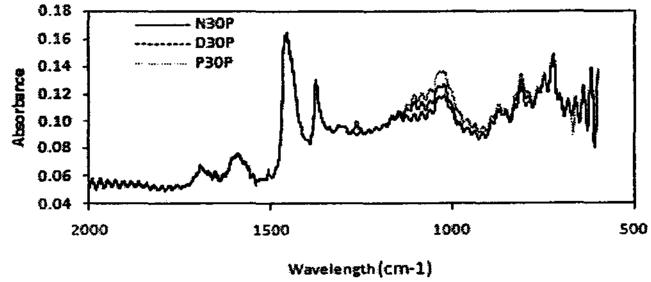
(a) Short Term Aging (15% RAP binder)



(b) Short Term Aging (30% RAP binder)



(c) Long Term Aging (15% RAP binder)



(d) Long Term Aging (30% RAP binder)

Fig. 5: Fourier transform infrared spectroscopy spectrum of RAP modified binders

3.6.2 Sulfoxide Groups of Modified RAP Binders:

Vibration of sulfoxide (S=O), a functional group most easily formed in bitumen upon oxidation of sulphide compound was captured at intense peak 1030 cm^{-1} from the IR spectra. The area of the sulphide absorption covered the wavelength between 1051 and 1027 cm^{-1}.

Table 5 relates similar trend for sulfoxide groups in which the area ratio of all sulfoxide groups increased consistently in both short term aging and long term aging for the three RAP sources. The increase aging time subsequently increase the area ratio in S=O where during the oxidation process, oxygen was absorbed by the sulphide compound of asphaltene molecular structures. This chemical reaction is contributed to further hardening of the RAP modified binder. The P15P and P30P RAP modified binders exhibit the highest S=O ratio at 0.599 and 0.664 at long term aging respectively.

3.7 Correlation Between Penetration Index, Viscosity Aging Index and Area Ratio:

Table 6 shows a high significance and Pearson correlation values between penetration index and viscosity aging index, and between penetration index and area ratio of the RAP modified binders. This is evident by high penetration index well corresponds to high viscosity aging index and area ratio of the RAP modified binders. Even though as viscosity aging index increase, the area ratio of carbonyl groups also increase, however in general there is no significant correlation between the parameters.

Table 6: Coefficient of correlation analysis

Correlation between	Pearson correlation	p-value (2-tailed)	Significant
PI*VAI	0.784	0.003	Yes
PI*AR	0.712	0.009	Yes
VAI*AR	0.501	0.097	No

Notes : PI –Penetration Index, VAI –Viscosity Aging Index, A –Area Ratio

Conclusion:

The penetration and softening point consistently decrease and increase, respectively after severe aging conditions for all RAP modified binders. The penetration index and viscosity aging index increase as the RAP modified binders are further aged. The RAP modified binders for all RAP binder sources and proportions exhibit distinct change in binder chemical evolution after subjected to short term and long term aging. The carbonyl and sulfoxide groups namely C=O and S=O consistently show increase in the area ratio at each level of oxidation. For the respective RAP binder content, the DPE and PWD RAP modified binders incorporating 15% and 30% RAP binders respectively are the most aged binders after long term aging. There is significant correlation between penetration index and viscosity aging index, and between penetration index and area ratio of RAP modified binders.

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REFERENCES

ASTM D2872, 2006. "Standard Test Method for Effect of Heat and Air on Moving Film of Asphalt (Rolling Thin-Film Oven Test)", ASTM International, West Conshohocken, PA,West Conshohocken, PA, USA.

ASTM D6521, 2006. "Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)", ASTM International, West Conshohocken, PA,West Conshohocken, PA, USA.

ASTM D5, 2006. "Standard Test Method for Penetration of Bituminous Materials". ASTM International, West Conshohocken, PA, USA.

ASTM D36, 2006. "Standard Test Method for Softening Point of Bitumen", ASTM International, West Conshohocken, PA, USA.

ASTM D4402, 2006, " Standard Test Method for Viscosity Determination of Asphalt at Elevated Temperatures Using a Rotational Viscometer" ASTM International, West Conshohocken, PA, USA.

Lamontagne, J., P. Dumas, V. Mouillet and J. Kister, 2001. Comparison by Fourier transform infrared(FTIR) spectroscopy of different ageing techniques: application to road bitumen. Fuel., 80: 483-488.

Lesueur, D., 2009. The colloidal structure of bitumen: Consequences on the rheology and on the mechanisms of bitumen modification. Advances in Colloid and Interface Science, 145: 42-82.

Le Guern, M., E. Chailleux, F. Farcas, S. Dreessen and I. Mabille, 2010. Physico-chemical analysis of five hard bitumens: Identification of chemical species and molecular organization before and after artificial aging. Fuel., 89: 3330-3339.

Lu, X and U. Isacsson, 2002. Effect of ageing on bitumen chemistry and rheology. Construction and Building Materials., 16: 15-22.

Read, J., and D. Whiteoak, 2003. The Shell Bitumen Handbook, 5th Edition, Shell Bitumen UK.

Roberts, F.L., P.S. Khandal, E.R. Brown, D.Y. Lee and T.W. Kenneday, 1991. Hot mix asphalt materials, mixture design, and construction. 1st Ed., NAPA Education Foundation, Lanham, Maryland.

Siddiqui, M.N. and M.F. Ali, 1999. Studies on the aging behaviour of the Arabian asphalts. Fuel., 78: 1005-1015.

Su, K., Y. Hachiya and R. Maekawa, 2009. Study on recycled asphalt concrete for use in surface course in airport pavement. Resources, Conservation and Recycling, 54: 37-44.

Toteva, V., A. Georgiev and L. Topalova, 2009. Oxidative desulphurization of light cycle oil monitoring by FTIR spectroscopy. Fuel Processing Technology, 90: 965-970.

Valcke, E., F. Rorif and S. Smets, 2009. Ageing of EUROBITUM bituminised radioactive waste: An ATR-FTIR spectroscopy study. Journal of Nuclear Materials, 39: 175-185.

Widyatmoko, I., 2008. Mechanistic-empirical mixture design for hot mix asphalt pavement recycling. Construction and Building Materials., 22: 77-87.

Zhang, F., J. Yu and J. Han, 2011. Effect of thermal oxidative ageing on dynamic viscosity, TG/DTG, DTA and FTIR of SBS and SBS/sulphur-modified asphalt. Construction and Building Materials, 25: 129-137.

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Dear Dr Hamzah,

I acknowledge with thanks your E-mail dated 01.04.2012 enclosing your paper entitled "Evaluation of the Dynamic Modulus of Asphalt Mixture Incorporating Reclaimed Asphalt Pavement" for publication in *Indian Journal of Engineering & Materials Sciences*. The paper is being processed and a further communication will follow.

With regards,

Yours sincerely,

(R S Beniwal)

Dr Meor Othman Hamzah
School of Civil Engineering
Universiti Sains Malaysia
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Penang
Malaysia

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Professor Dr. Meor Othman Hamzah

Evaluation of the Dynamic Modulus of Asphalt Mixture Incorporating Reclaimed Asphalt Pavement

Zulkurnain Shahadan, Meor Othman Hamzah[1], Ahmad Shukri Yahya and Ali Jamshidi

School of Civil Engineering, Engineering Campus, Universiti Sains Malaysia,
14300 Nibong Tebal, Penang, MALAYSIA.

[1]Corresponding author: cemeor@yahoo.com

ABSTRACT

This paper presents the effects of temperature and loading frequency on the dynamic modulus and phase angle of asphalt mixtures incorporating reclaimed asphalt pavement (RAP) using the Asphalt Mixture Performance Tester. Milling waste from Damansara-Puchong Expressway was incorporated in asphalt mixtures in proportions of 0%, 10%, 20%, 30% and 40%. The asphalt mixtures were tested for dynamic modulus at three temperatures (20, 40, 50°C) and six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz). At constant temperature, the dynamic modulus increased as the loading frequency and RAP content increased. For a given frequency, the dynamic modulus decreased while the phase angle increased as the temperature increased. From statistical analysis, test temperature and frequency have significant effects with high effect size on the measured dynamic modulus and phase angle. The interaction effect of frequency and RAP gave the highest effect size among the interaction effects in the dynamic modulus test. The results also indicated that the highest performance in terms of rutting and fatigue factors can be attained when the frequency of cumulative traffic loading was from 15 to 20 Hz.

Key words: Reclaimed Asphalt Pavement, Asphalt Mixture Performance Tester, Dynamic Modulus, Phase Angle, Interaction Effect, Effect Size

Introduction

Incorporating reclaimed asphalt pavement (RAP) in asphalt mixtures has been a common practice in road construction and rehabilitation especially in North America and Europe. This is in line with sustainable road construction and green development concepts that have been championed for the last two decades. The advantages of utilizing reclaimed asphalt pavement include reduce waste, preservation of the existing pavement geometrics, preservation of natural resources, minimise life-cycle cost, and conservation energy[1]. Recycled asphalt pavement has been proven to perform equally or better than conventional asphalt pavement in the laboratory and field tests. From field tests, recycled asphalt pavement is able to withstand increasing number of vehicles and higher axle loads imposed by different axle configurations and severe climatic conditions. Addition of reclaimed asphalt pavement in asphalt mixture has improved permanent deformation and fatigue distress[2-4]. Currently, dynamic modulus (E*) is the preferred asphalt mixture structural contribution parameter and is one of the most important parameters required in flexible pavement design based on the Mechanistic Empirical Pavement Design Guide (MEPDG)[5]. According to Witczak et al. (2002)[6], the dynamic modulus is a crucial parameter used in evaluating rutting and fatigue cracking distress prediction in the MEPDG. The dynamic modulus represents asphalt mixture stiffness in response to the application of haversine compressive load on a cylindrical sample over several temperatures and loading frequencies. The stiffness of an asphalt mixture reflects its load spreading ability. A master curve is developed to represent the stiffness relationship of asphalt mixture in relation to temperatures and loading frequencies.

There appears to be a gap in the knowledge on performance of asphalt mixtures incorporating different RAP contents at various test temperatures using the Asphalt Mixture Performance Tester (AMPT). Therefore, this paper focuses on the dynamic modulus and phase angle of asphalt mixtures incorporating reclaimed asphalt pavement (RAP) at five RAP proportions, three temperatures and six loading frequencies with in-depth statistical analysis particularly on the interaction effect and effect size. In addition, master curves are developed to evaluate rutting and fatigue factors at different test temperatures for asphalt mixtures incorporating different RAP contents.

Materials and methods

Reclaimed asphalt pavement

Milling waste of aged and deteriorated pavements from the Damansara Puchong Expressway (DPE), Malaysia was used in this study. The gradation of DPE RAP aggregate is shown in Table 1.

Table 1: Gradation of DPE RAP aggregate

Sieve size (mm)	20	14	10	5	3.35	1.18	0.425	0.150	0.075
Percent Passing	99	93.8	87.4	73.2	63.5	38.2	21.7	10.9	6.3

Asphalt binder

A conventional binder penetration grade 80/100 (PG64) supplied by PETRONAS, a Malaysian oil company, was used as the virgin binder. The RAP binder was recovered using the rotavapor method. Table 2 shows the physical and rheological properties of the virgin and DPE RAP recovered binders.

Table 2: Physical and rheological properties of virgin and recovered binders

Binder	Penetration at 25°C (dmm)	Softening point °C	Viscosity at 135°C (Pa.s)	G*/Sinδ at 64°C (kPa)
Virgin	90	46.0	0.34	1.14
LDE	13	67.5	4.33	36.1

Aggregate

The crushed virgin granite aggregate was obtained from a local quarry. The virgin aggregate was washed, dried and sieved based on the Malaysian Public Works Department (PWD) aggregate gradation for asphaltic concrete ACW14 (JKR, 2008)[7] as shown in Table 3.

Table 3: PWD Gradation Limits for Asphaltic Concrete ACW14

Sieve size (mm)	20	14	10	5	3.35	1.18	0.425	0.150	0.075
Percent Passing	100	90-100	67-86	50-62	40-54	18-34	12-24	6-14	4-8

Sample preparation

Samples for dynamic modulus testing were prepared by mixing binder, virgin aggregates and processed reclaimed asphalt pavement at 160°C. An hour prior to the mixing, the processed reclaimed asphalt pavement was preheated in an oven at 135°C. The optimum binder content for each RAP proportion is shown in Table 4. After mixing, the mixtures were poured onto a large flat pan and placed in an oven set at 135°C for 4 hours for short term aging in accordance with AASHTO R30 (AASHTO, 2005a)[8] procedures. The samples were compacted at 150°C in a 150 mm diameter mould to a height of 170 mm by using a gyratory compactor. After compaction, the sample was extruded from the compaction mould, labelled and allowed to cool to room temperature. The compacted samples were cored and trimmed to obtain a standard 150 mm height and 100 mm diameter test sample with targeted air voids 7±0.5%.

Table 4: Optimum binder content

Percentage of RAP	0	10	20	30	40
Optimum Binder Content	4.6%	4.9%	5.2%	5.3%	5.4%

Methodology

Dynamic modulus test

The linear viscosity properties of asphalt mixtures were measured from the dynamic modulus test conducted in accordance with AASHTO TP 62-03(AASHTO, 2005b)[9] procedures using the Asphalt Mixture Performance Tester (AMPT). The AMPT machine, manufactured by IPC Global, was equipped with a temperature chamber and a pressure cell. Compressive load was applied in the form of a continuous haversine wave without rest period. Six frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and three temperatures (20, 40 and 50°C) were selected for the tests. For data measurement, three linear variable differential transducers (LVDTs) were used. The LVDTs were mounted on the sides of the specimen to measure axial deformation. The tests were conducted within the linear viscoelastic stress level where the strain was controlled between 85 to 115 micro-strains. Since the test was nondestructive test, the same specimen was used for a complete test at six frequencies and three temperatures. The tests were performed from the lowest temperature to the highest temperature and from the highest frequency to the lowest frequency.

The dynamic modulus |E*| was calculated using Equation (1)[6].

$$|E^*| = \frac{\sigma_o}{\varepsilon_o} \quad (1)$$

where σ_o = Applied stress amplitude (MPa)
ε_o = Measured strain amplitude

The phase angle can be obtained using Equation (2).

$$\delta = \frac{t_i}{t_p} \quad (2)$$

where t_i = Average time lag between a cycle of stress and strain (s)
t_p = Average time for a stress cycle (s)

Statistical analysis

An analysis of variance (ANOVA) was adopted to analyze main and interaction effects of independent variables on the measured dynamic modulus and phase angle by using the statistical software SPSS. Further analysis was pursued to determine the effect sizes (the magnitude) of the main and interaction effects by using Equations (3) through (7)[10]. This effect size analysis is limited to two main effects (two independent variables) and one interaction effect.

$$\hat{\sigma}_A^2 = \frac{(a-1)(MS_A - MS_R)}{nab} \tag{3}$$

$$\hat{\sigma}_B^2 = \frac{(a-1)(MS_B - MS_R)}{nab} \tag{4}$$

$$\hat{\sigma}_{AxB}^2 = \frac{(a-1)(b-1)(MS_{AxB} - MS_R)}{nab} \tag{5}$$

$$\hat{\sigma}_{total}^2 = \hat{\sigma}_A^2 + \hat{\sigma}_B^2 + \hat{\sigma}_{AxB}^2 + MS_R \tag{6}$$

$$\omega_{effect}^2 = \frac{\hat{\sigma}_{effect}^2}{\hat{\sigma}_{total}^2} \tag{7}$$

where, ω^2 = Effect size
$\hat{\sigma}^2$ = Variance
MS = Mean square
MS_R = Mean square of residual
A = Main effect of the first independent variable
B = Main effect of the second independent variable
AxB = Interaction effect AxB
a = Number of levels of the first independent variable
b = Number of levels of the second independent variable

Result and discussion

Dynamic modulus

Figure 1 shows the effects of RAP content, temperature and loading frequency on the dynamic modulus of asphalt mixtures. At 20°C, the dynamic modulus consistently increases as the RAP content increases. The average percentage increase ranges from 1% to 6% when up to 20% RAP is added. The dynamic modulus increases from 13% to 17% when incorporated with 30% to 40% RAP. When the temperature doubles to 40°C, the dynamic modulus dramatically reduces to 80% and 78% for asphalt mixtures incorporating 10% and 40% RAP, respectively. As the temperature increases from 40°C to 50°C, the dynamic modulus is significantly reduced to 70% and 64% for asphalt mixtures with 10% and 40% RAP, respectively. Similar decreasing trend in dynamic modulus between 71% to 87% and 61% to 68% is observed when loading frequency decreases as temperature increases from 20°C to 40°C and 40°C to 50°C, respectively.

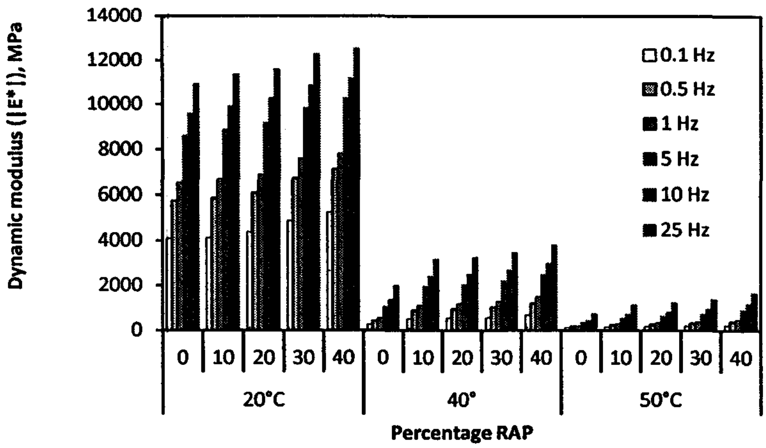


Fig. 1: Dynamic modulus at different RAP content, temperature and frequency

For a given RAP content, the reduction in the dynamic modulus at 40°C is more evident compared to samples tested at 20°C. However, for samples tested at 40°C, the dynamic modulus increases from 47% to 50% and 53% to 59% when 20% and 40% RAP are added, respectively compared to control mixtures. This takes place because RAP materials contain aged asphalt binder where resins turn into asphaltenes which in turn affects the elastic solid behavior of the aged asphalt binder[12-14]. However, the higher amount of RAP incorporated in virgin mixtures implicates an increased amount of fuel requirement and green house gas emission during asphalt production in the mixing plants[14-15].

The dynamic modulus increases from 42% to 56% as the loading frequency increases from 0.1 Hz to 25 Hz. As the test temperature further increases to 50°C, the dynamic modulus reduces significantly to 94% and 92% for asphalt mixtures incorporating 10% and 40% RAP, respectively. For specimens tested at 50°C, in comparison with control mixtures, the dynamic modulus increases from 32% to 40% and 47% to 57% when blended with 20% and 40% RAP, respectively. The increase of the dynamic modulus at that respective RAP content is less than 10% compared to those samples tested at 40°C. However, the dynamic modulus increases from 90% to 96% as the loading frequency increases from 0.1 Hz to 25 Hz. This increase in mixture stiffness is attributed to the viscoelastic property of the aged binder from the RAP. Higher RAP content incorporated in the asphalt mixtures results in increased mixture stiffness that enable the mixture to withstand the detrimental effects of high temperature and low loading frequency.

An analysis of variance (ANOVA) was performed to determine the effects of RAP content, temperature, frequency and interaction of the main effects on the measured dynamic modulus. Table 5 shows the results of ANOVA which indicate that all main effects as well as their interactions have significantly influenced the measured dynamic modulus at 5% significance level. It can be noticed that main effect of temperature has the highest F-ratio value which indicates that an increase in temperature would dramatically affects the dynamic modulus of the asphalt mixtures.

Table 5: ANOVA results on main and interaction effects on dynamic modulus

Source	Sum of Squares	df	Mean Square	F	p-value	Significant
Intercept	2194763907.200	1	2194763907.200	1159029.442	<0.001	Yes
RAP	22284799.244	4	5571199.811	2942.086	<0.001	Yes
Temperature	2059022431.433	2	1029511215.717	543672.969	<0.001	Yes
Frequency	278791124.267	5	55758224.853	29445.274	<0.001	Yes
RAP * Temperature	6156365.289	8	769545.661	406.388	<0.001	Yes
RAP * Frequency	2159036.622	20	107951.831	57.008	<0.001	Yes
Temperature * Frequency	145407902.300	10	14540790.230	7678.823	<0.001	Yes
RAP * Temperature * Frequency	501909.644	40	12547.741	6.626	<0.001	Yes
Error	170426.000	90	1893.622			
Total	4709257902.000	180				
Corrected Total	2514493994.800	179				

Figure 2 shows the interaction plot of temperature and RAP against dynamic modulus at 10 Hz. The plot displays increasing trend of estimated marginal means of the dynamic modulus as RAP content increases. At 40°C, a sharp increase in estimated marginal mean of the dynamic modulus is observed for asphalt mixtures with 10% RAP. The effect of high temperature drastically reduces the stiffness of the asphalt mixtures regardless of RAP content. However, the addition of RAP in asphalt mixtures at a particular temperature has improved mixture stiffness compared to control mixtures.

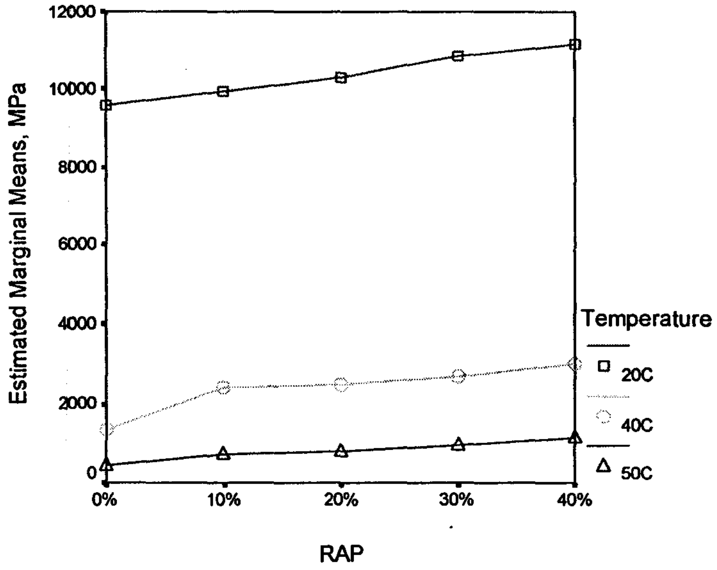


Fig. 2: Interaction plot of temperature and RAP against dynamic modulus at 10 Hz

Phase angle

Figure 3 shows the effects of RAP content, temperature and loading frequency on the phase angle of the asphalt mixtures tested. Generally, at 20°C the phase angle decreases as the RAP content increases with average percentage decreases of 0.5% when up to 20% RAP is added and further decreases to between 8% to 14% when incorporating 20% to 40% RAP, compared to control mixtures. When the temperature doubles to 40°C, the phase angle for asphalt mixture incorporating up to 20% RAP, increases sharply between 12% to 52%, while for asphalt mixtures incorporating 30% to 40% RAP, the phase angle increases to between 23% to 54%. In addition, as the loading frequency increases, the phase angle consistently increases from 12% to 54%. When the test temperature increases from 40°C to 50°C, the phase angle initially reduces from 14% and 8% when incorporating 10% and 40% RAP, respectively. However, as the frequency progressively increases, the phase angle also increases up to 19% at 40% RAP. For specimens tested at 40°C, the phase angle slightly increases at 0.1 Hz for all RAP percentages but decreases up to 25% for the rest of the loading frequencies compared to control samples. At 50°C, in comparison with control mixtures, the phase angle increases up to loading frequency equivalent to 10 Hz. However, at 25Hz frequency, the phase angles slightly decreases up to 9%. This decrease in phase angle can be explained in terms of aggregate interlocking effects.

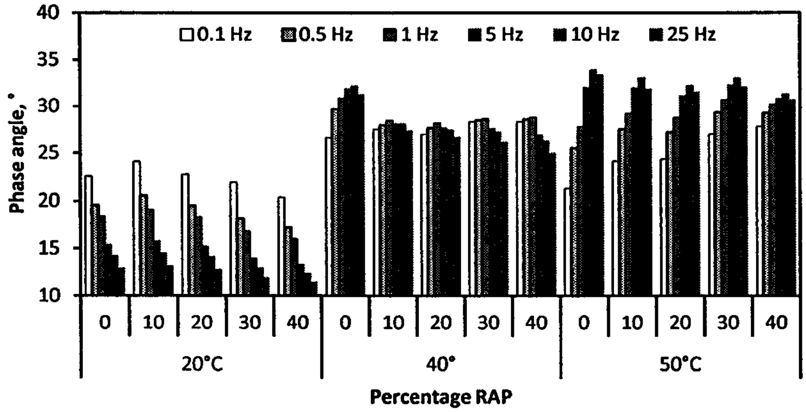


Fig. 3: Phase angle at different RAP content, temperature and frequency

An analysis of variance (ANOVA) is carried out to determine the effects of RAP content, test temperature, frequency and interaction of the main effects on the measured phase angle. Table 6 shows the ANOVA results which indicate that all the main effects as well as their interactions have significantly influence the measured phase angle at 5% significance level. Similar to dynamic modulus, temperature is a dominant main effect since temperature exhibits the highest F-ratio which indicates that temperature increase will significantly affects the phase angle of the asphalt mixtures.

Table 6: ANOVA results on main and interaction effects on phase angle

Source	Sum of Squares	df	Mean Square	F	p-value	Significant
Intercept	111122.244	1	111122.244	2214865.125	<0.001	Yes
RAP	40.843	4	10.211	203.517	<0.001	Yes
Temperature	6129.408	2	3064.704	61085.032	<0.001	Yes
Frequency	37.189	5	7.438	148.250	<0.001	Yes
RAP * Temperature	119.215	8	14.902	297.020	<0.001	Yes
RAP * Frequency	87.996	20	4.400	87.696	<0.001	Yes
Temperature * Frequency	1126.096	10	112.610	2244.511	<0.001	Yes
RAP * Temperature * Frequency	57.246	40	1.431	28.525	<0.001	Yes
Error	4.515	90	.050			
Total	118724.752	180				
Corrected Total	7602.508	179				

Figure 4 shows the interaction plot of temperature and RAP against phase angle at 10 Hz. The plot clearly indicates that generally the estimated marginal means of the phase angle decreases as RAP content increases. Improvement in elasticity property of asphalt mixture incorporating RAP compared to control mixtures is particularly evident at

40°C. The contribution of aged bitumen from reclaimed asphalt pavement has effectively enhanced the mixture viscoelastic characteristics which results in improved stiffness of the asphalt mixtures at high temperature.

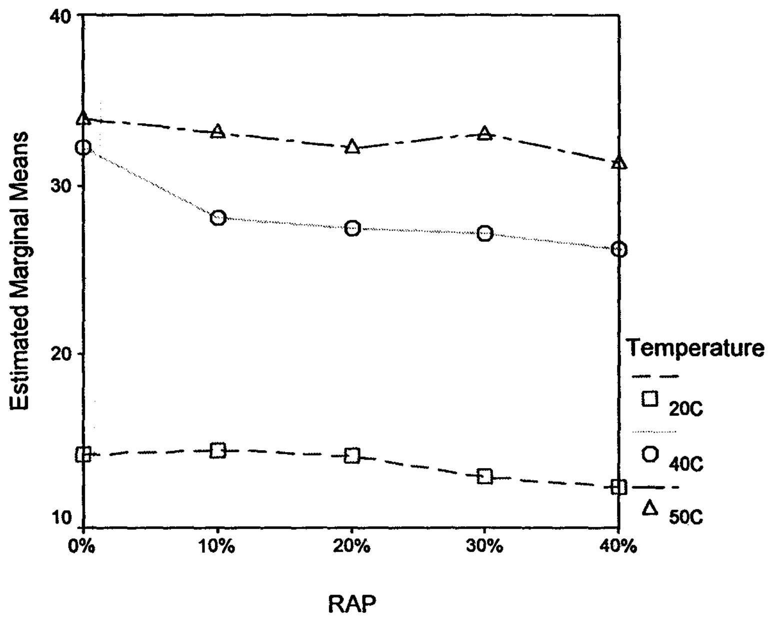


Fig. 4: Interaction plot of temperature and RAP against phase angle at 10 Hz

Figures 5 illustrates the Cole–Cole curve of the relationship between the elastic or storage modulus (E′) and the viscous or loss modulus (E″) of the asphalt mixtures based on results obtained from the AMPT. The Cole–Cole curve is suggested to validate the test data for a master curve at any frequency or temperature[11]. The acceptable data are independent of temperature and frequency and form a single curve. As illustrated in Figure 5, the relationship between E′ and E″ are linear and form a single curve with R2 equal to or greater than 90%, for all test temperatures. This implies that the data from the master curves illustrated in Figures 3, and 1 are valid.

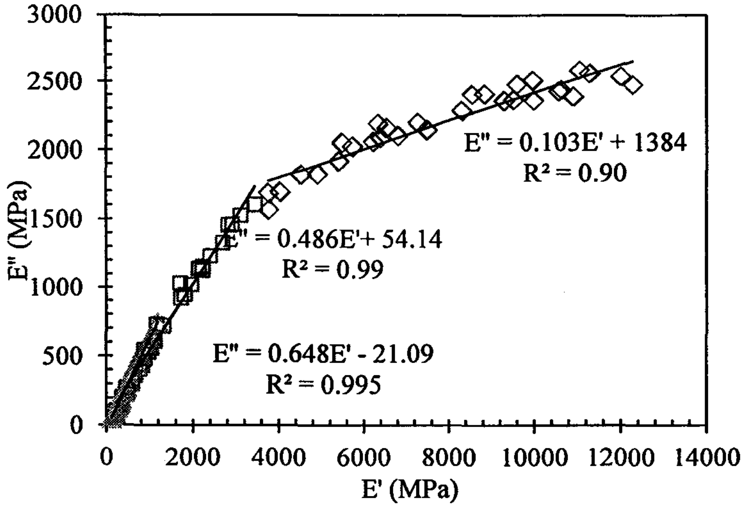


Fig. 5. Cole–Cole curve of the relationship between E′ and E″

Effect size of the main and interaction effects

The effect size of the main and interaction effects on measured dynamic modulus and phase angle are categorized into 3 Groups as shown in Table 7. It can be seen that effect size of temperature is distinctly large for both dynamic modulus and phase angle when temperature becomes one of the main effects as in Groups 1 and 2. In Group 3, frequency has the largest effect size compared to main effect of RAP on dynamic modulus. However, with the same main effects (frequency and RAP), the effect sizes are comparable in the measured phase angle. Even though RAP has significantly contributed to asphalt mixture stiffness at high temperature, the effect size is fairly low compared to temperature and frequency except for measured phase angle where its effect size is 0.24. Similarly, the effect size for the interaction effects is considerably low compared to the main effects except when measuring the phase angle. The interaction effect between frequency and RAP in phase angle has effect size of 0.51, is the highest among the interaction effects (temperature x frequency and temperature x RAP) in the tests. Even though the effect size of the interaction effect analysis is limited to two main effects only, it can be expected that the effect size of the interaction effect of three main effects (RAP x temperature x frequency) is much less than the interaction effect of two main effects based on the lowest F-ratio of the interaction effect of RAP x temperature x frequency compared to other interaction effects.

Table 7: Effect size (ω^2) of the main and interaction effects

Group	Measured parameter	Main effect	Interaction effect

		Temperature	Frequency	Temperature x Frequency
1	Dynamic Modulus	0.83	0.11	0.05
	Phase Angle	0.84	0.005	0.15
		Temperature	RAP	Temperature x RAP
2	Dynamic Modulus	0.99	0.01	0.003
	Phase Angle	0.97	0.006	0.02
		Frequency	RAP	Frequency x RAP
3	Dynamic Modulus	0.92	0.07	0.007
	Phase Angle	0.22	0.24	0.51

Asphalt mixture stiffness

Figure 6 displays the dynamic modulus master curve for mixtures with 5 RAP contents, tested at three test temperatures (20°C, 40°C and 50°C) and six loading frequencies(25, 10, 5, 1, 0.5 and 0.1Hz). The master curve was developed taking 25°C as the reference test temperature. Generally, the mixture stiffness increases as the RAP content increases. These increases in RAP content have significantly increase the dynamic modulus which directly translates into improved load spreading ability of the asphalt mixtures incorporating RAP. Figure 7 shows that the measured dynamic modulus |E*| matches reasonably well with the predicted dynamic modulus |E*| calculated based on Witczak model without any significant bias with regression coefficient equal to 0.983.

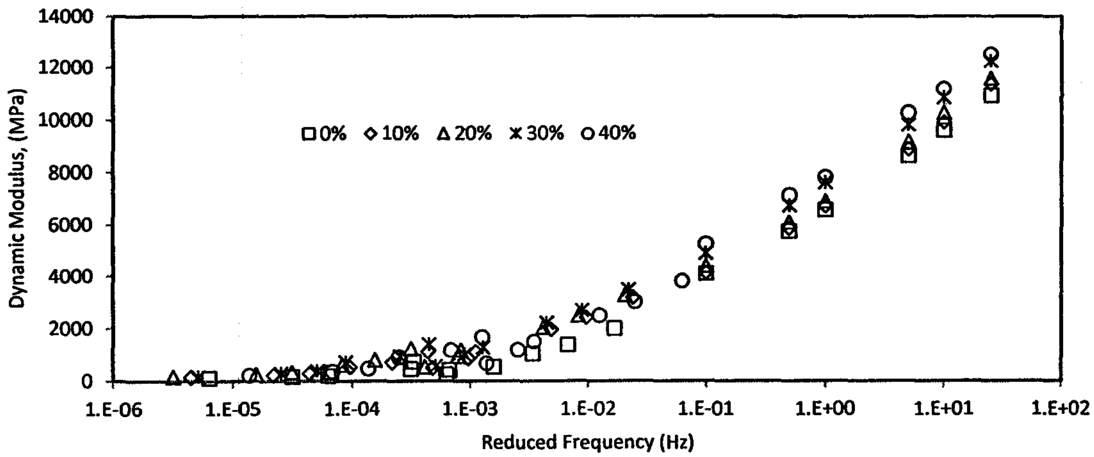


Fig. 6: Master curves at different RAP percentages

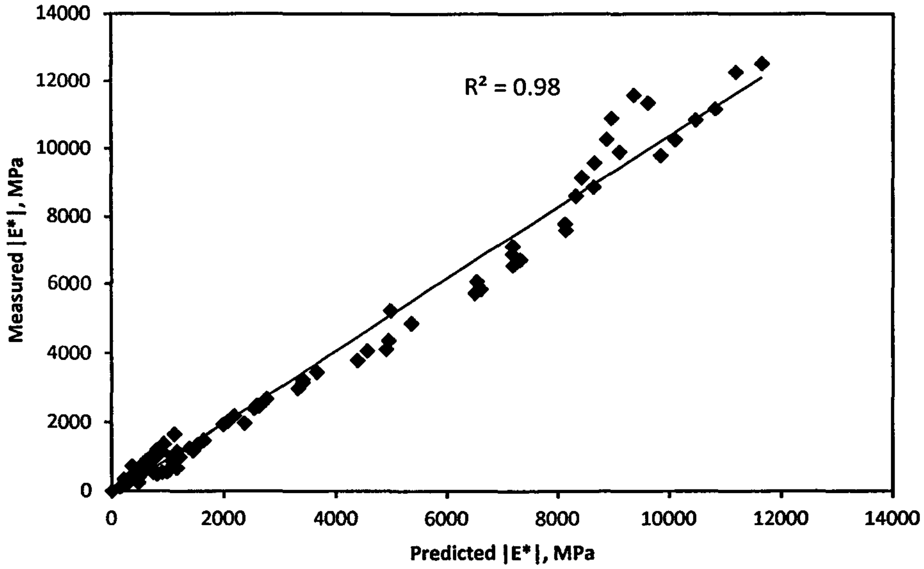
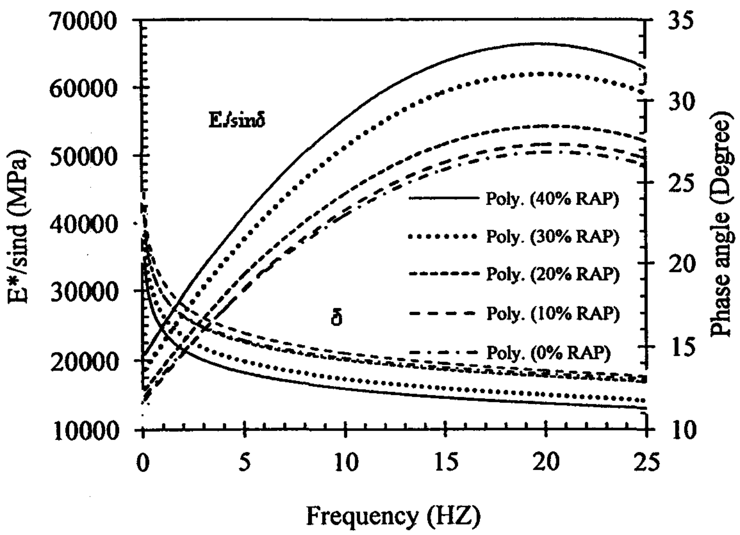


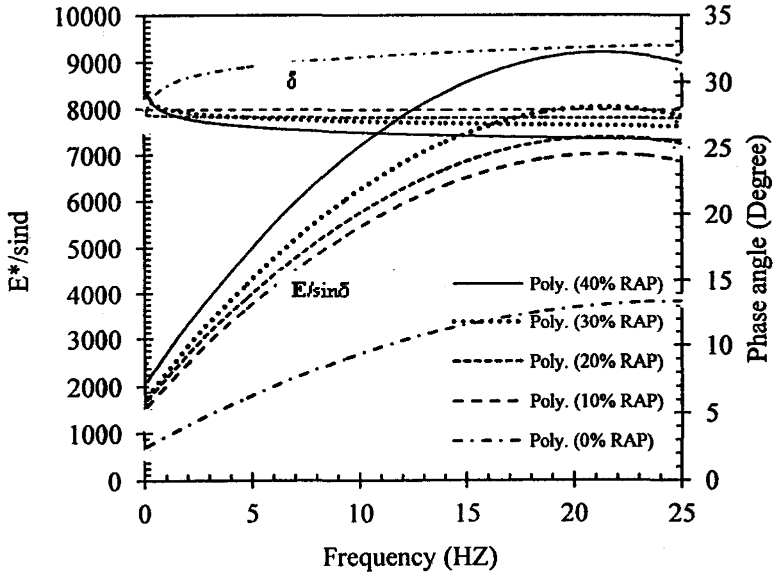
Fig. 7: Relationship between measured |E*| Laboratory and |E*| Predicted

An improved resistance to rutting and fatigue in mixtures incorporating RAP is further illustrated in Figures 8 and 9, respectively. For instance, rutting factor, |E*|/sin δ at 50°C and 10Hz has significantly increased by 37 % and 64%, respectively for mixtures incorporating 10% and 40% RAP. Interestingly, the fatigue parameter ,|E*|.sin δ at 20°C and 10 Hz slightly increased up to 5% at 20% RAP and 1% for mixture incorporating 40% RAP, and which is comparable to the fatigue parameter of control mixtures.

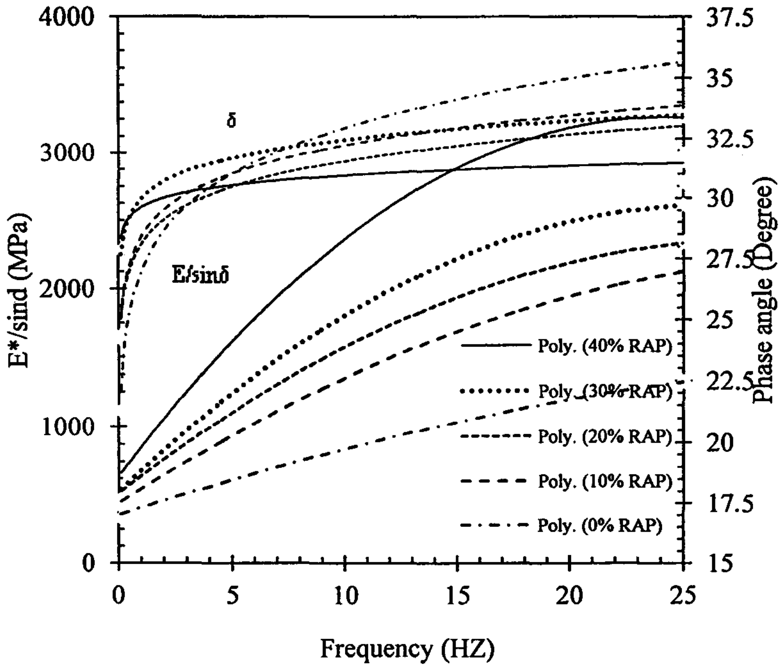
From Figures 8(a) to 8(c), it can be see the maximum rutting factor takes place at approximately 20 HZ at each test temperature, while the maximum fatigue factor is from 15 Hz to 20 Hz as shown in Figures 9(a) to 9(c). Therefore, it is recommended that the frequency of total traffic loading to achieve maximum rutting and fatigue performance should be in the range 15 to 20 Hz for the asphalt mixtures tested using RAP from DPE. Since aged binder in RAP materials supplied from various sources may have different rheological properties, it is expected to affect the engineering properties of asphalt mixtures containing RAP from different sources (Jamshidi et. al., 2012). Therefore, the recommended range of frequency to obtain maximum rutting and fatigue factors can vary for asphalt mixtures constructed using RAP from other sources.


70000
60000
50000
40000
30000
20000
10000
E*/sind (MPa)
35
30
25
20
15
10
Phase angle (Degree)
E/sinδ
δ
Poly. (40% RAP)
Poly. (30% RAP)
Poly. (20% RAP)
Poly. (10% RAP)
Poly. (0% RAP)
0
5
10
15
20
25
Frequency (HZ)

(a) 20°C

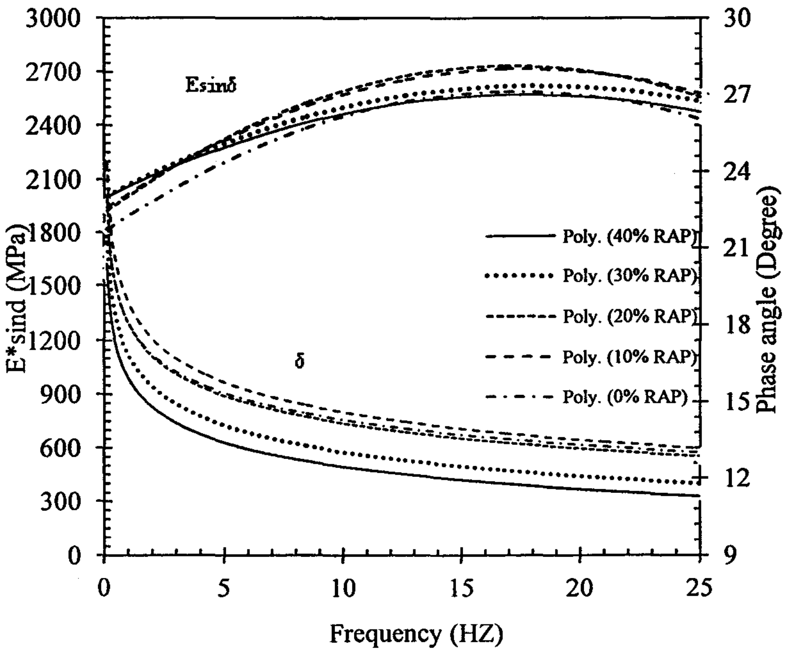

10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
E*/sind
35
30
25
20
15
10
5
0
Phase angle (Degree)
δ
E/sinδ
Poly. (40% RAP)
Poly. (30% RAP)
Poly. (20% RAP)
Poly. (10% RAP)
Poly. (0% RAP)
0
5
10
15
20
25
Frequency (HZ)

(b) 40°C

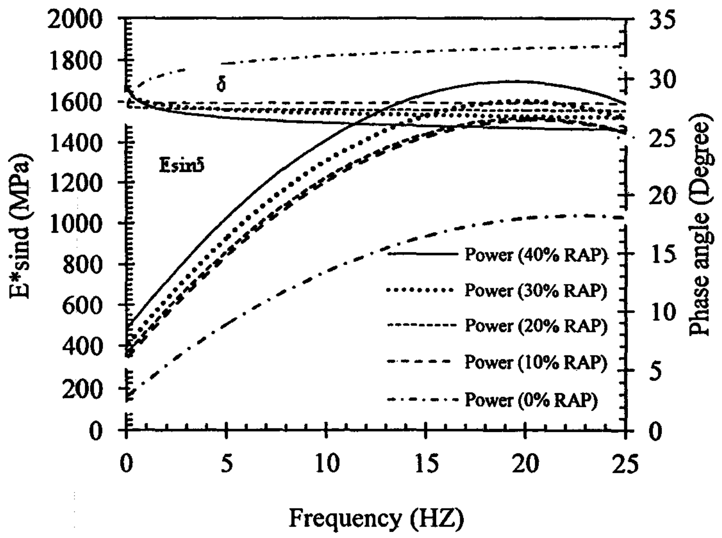


(c) 50°C

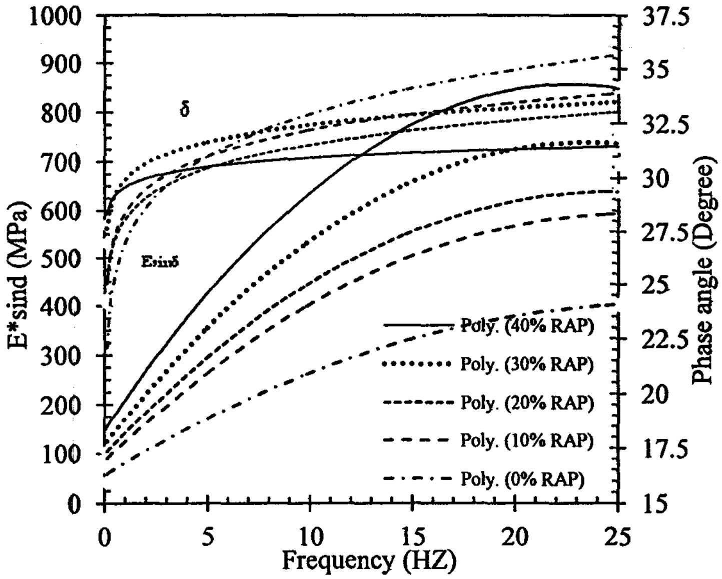
Fig.8 Master curve of rutting factor versus frequency sweep at designated temperatures



(a) 20°C



(b) 40°C



(c) 50°C

Fig. 9 Master curve of fatigue factor versus frequency sweep at designated temperatures

Conclusion

At constant test temperature, the dynamic modulus increases as the loading frequency and RAP content increase. The phase angle decreases as the loading frequency and RAP content increases at 20°C. However, at 40°C and 50°C, the phase angle increases up to loading frequencies 1Hz and 10Hz, respectively beyond which it reduces. At constant frequency, the dynamic modulus decreases as temperature increases, while the phase angle increases as temperature increases. Temperature and frequency have significant effects with high effect size on the measured dynamic modulus and phase angle. The interaction effect of frequency and RAP has the highest effect size among the interaction effects on the dynamic modulus test particularly on the phase angle. Rutting parameter increases as RAP content increases and fatigue parameter is well below the maximum limit and in fact the fatigue parameter further decreases with addition of more than 20% RAP. From the rutting and fatigue versus frequency master curves, the recommended cumulative traffic loading frequency for the asphalt mixtures tested ranges from 15 to 20 Hz.

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References

1 Kandhal P.S & Mallick R, *Pavement Recycling Guidelines for State and Local Governments*, (FHWA, U.S. Department of Transportation),1997.

2 Su K, Hachiya Y, & Maekawa R, *Resources, Conservation and Recycling,* 54(2009) 37-44

3 Aravind K & Das A, *Const and Build Mater,* 21(2007) 928-936

4 Widyatmoko I, *Const and Build Mater* 22(2008) 77-87

5 *Guide for Mechanistic Empirical Design of New and Rehabilitated Pavement Structures*, (NCHRP Project 1-37A), 2004

6 Witczak MW, Kaloush K, Pellinen T, El-Basyouny M & Von Quintus H, *Simple performance test for Superpave mix design*, (Transportation Research Board, National Research Council, Washington, DC Report No. 465), 2002

7 *Standard Specification for Road Works* (SPJ 2008-Section4: Flexible Pavement), Cawangan Jalan, Jabatan Kerja Raya(JKR) Malaysia, Kuala Lumpur), 2008

8 *AASHTO Designation, R30, Mixture Conditioning of Hot Mix Asphalt (HMA)*, (American Association of State Highway and Transportation Officials, Washington, D.C), 2005a

9 *AASHTO Designation TP 62-3*, Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures, (American Association of State Highway and Transportation Officials, Washington, D.C), 2005b

10 Field A, *Discovering Statistics Using SPSS*, (Third Edition, SAGE Publication), 2009

11 Akisetty C.K, Gandhi T. Lee S.J & Amirkhanian S. N. *Canadian J Civil Eng* 37(2010): 763-771

12 Gao M, Xiao B, Liao K, Cong YF & Dai Y, *Petro Scie and Techno* 24(2006) 689-698.

13 Rajan N.K, Selvavathi V, Sairam B & Krishnan J.M, *Petro Scie and Techno* 28(2010) 331-350

14 Hamzah M.O, Jamshidi A & Shahadan Z, *Europ J Sci Res* 42(2010) 15-24

15 Jamshidi A, Hamzah M.O & Shahadan Z, *J Clean Prod* 23(2012) 20-27

Rheological characterization of asphalt binders blended with different contents of aged binders from various sources

Ali Jamshidi[1], Meor Othman Hamzah[1*], Zulkurnain Shahadan[1], Zhaoping You[2]

[1]School of Civil Engineering, Engineering Campus, Universiti Sains Malaysia,

14300 Nibong Tebal, Penang, MALAYSIA.

[2] Department of Civil and Environmental Engineering, Michigan Technological University, 1400 Townsend Drive, Houghton, MI, USA

*Corresponding author: Prof. Meor Othman Hamzah

Tel.: +604599 6210; fax: +6045941009

Email: cemeor@yahoo.com

Abstract

This paper studies the rheological properties of unaged and aged asphalts blended with different amounts of binder recovered from reclaimed asphalt pavement (RAP) from various sources. In order to characterize the effects of RAP source and content, three rheological based parameters were used, namely non-dimensional viscosity index($\nabla\eta_{RAP}$), relative Superpave™ rutting factor gradient (RSRG) and relative Superpave™ fatigue factor gradient (RSFG). Activation energy of asphalt binder blends was also studied. The results indicated that changes in the rheological properties of asphalt blends in terms of $\nabla\eta_{RAP}$, RSRG, RSFG and AE significantly depended on RAP source and content.

Keywords: Reclaimed Asphalt Pavement; Asphalt Rheology; Aging; Activation Energy.

1. Introduction

Asphalt mixture is one of the largest materials used to construct pavements in transportation infrastructures in the United States, that more than 90% of surfaced roads were constructed using asphalt concrete (Roberts et al., 2002, Druta et al., 2009, Kim, 2009). In this regard, around 500 million tons asphalt mixtures are produced to pave new roads and maintenance and rehabilitation of constructed roads(Keches and LeBlank, 2007). When asphalt pavement's life is over, the road surfacing is milled, creating a milling waste material known as Reclaimed Asphalt Pavement (RAP). These waste materials are very valuable since those are a rich source of asphalt and aggregate materials. The importance of RAP material increases significantly in recent decades because it reduces aggregate extraction in quarries; therefore, it is consistent to efforts made to preserve non-renewable source of construction materials for future generations. In addition, the price of asphalt mixture production increases in recent years (Hassan, 2009). One explanation can be that the oil refineries are very keen high-value-added products rather than asphalt binder, which was once regarded as waste material from "the bottom of the barrel" (Jamshidi et al., 2013). More important reason is that the price of crude oil as main source of asphalt binder increase since asphalt pavement technology has already come to the end of cheap crude oil era (Jamshidi, 2012). Therefore, use of RAP can be substantially harmony with concept of sustainable pavements. For instance, use of RAP in construction of pavement base and sub-base layers could reduce global warming potentials (20%), amount of energy consumption (16%), life-cycle costs (21%) and, hazardous waste generation (11%) (Lee et al., 2010).

It should be considered that when virgin asphalt binder in the mixture is blended with aged asphalt binder in the RAP, the resultant asphalt blend is stiffer than the virgin binder but no stiffer than the aged binder of RAP. Meanwhile the RAP may contain different aggregate type, binder type and content as well as various additives, such as anti-stripping agent and asphalt modifier. These variables can affect the rheological properties of the asphalt binder blends and engineering properties of mixture. For example, Jamshidi et al., (2012) showed that construction temperatures of mixtures incorporating RAP from various sources can be different, and hence, fuel requirement and greenhouse gas emission in asphalt mixing plants.

Therefore, in order to take the advantageous of RAP in pavement construction, it is necessary to study effects of RAP source on the rheological properties of the asphalt binder blends before evaluating the engineering properties of mixtures containing RAP. Currently, detailed information on characterization of rheological properties of unaged and aged asphalt binders blended with different contents of recovered binders from various RAP sources are not available. This context attempts to fill up this gap of knowledge. To accomplish this aim, the parameters are used to compare and characterize the effects of RAP source and content at high and intermediate temperatures using Superpave™ asphalt binder tests. The effects of RAP source and content on energy activation of unaged and aged asphalt binders were also evaluated at high and intermediate temperature ranges.

2. Material and Methods

2.1. Materials

2.1.1 Asphalt Binder

A virgin or control asphalt binder grade PG64 was used as the base binder. The binder was supplied by PETRONAS, a Malaysian national oil company. Table 1 presents rheological characteristics of the base binder used.

Table 1: Rheological properties of virgin PG64 binder

Aging State	Test Properties	Value
Unaged binder (original state)	Viscosity at 135°C (Pa.s)	0.465
	G*/sin δ at 64C° (kPa)	1.23
	Failure temperature (°C)	66.4
	Penetration (0.1 mm)	88
	Softening point(°C)	46
RTFO aged residue	G*/sin δ (kPa) at 64°C	2.68
	Failure temperature (°C)	66
RTFO+ PAV aged residue	G*sin δ at 25°C (kPa)	2958.75

2.2.2. Recovered Asphalt Binder from RAP

The study focuses on RAP milled from three roads and highways in Malaysia namely, the Damansara Puchong Expressway (DPE), North-South Expressway (NSE) and Public Works Department (PWD) road. Asphalt binder recovery and extraction from RAP materials were carried out via the Rotovapor method by IKRAM Sdn Bhd. Approximately 400 g of recovered binder was obtained from each RAP source and Table 2 SHOWS their rheological characteristics.

Table 2: Rheological properties of recovered RAP binder

Source	Penetration test at 25°C (dmm)	Softening point (°C)	Viscosity at 135°C (Pa.s)
NSE	14	67.5	3.06
DPE	13	67.5	4.33
PWD	11	72	2.14

2.2. Methods

2.2.1 Sample Preparation and Designation

The recovered binder from RAP was directly blended with the virgin binder at 140°C in proportions of 15% and 30% by mass of asphalt binder. A designation was adopted to simplify identification of the unaged and aged asphalt blends as indicated in Table 3. For example, binder sample D15S denotes an asphalt binder blended with 15% recovered binder from source DPE subjected to short term aging. Figure 1 shows flowchart of experimental design used in this study.

Table 3: Designation of asphalt binder blends

Source	Recovered binder content (%)	Aging state	Asphalt blends designation
NSE	15	Unaged	N15U
	30		N30U
	15	STA[1]	N15S
	30		N30S
	15	LTA[2]	N15L
	30		N30L
DPE	15	Unaged	D15U
	30		D30U
	15	STA[1]	D15S
	30		D30S
	15	LTA[2]	D15L
	30		D30L
PWD	15	Unaged	P15U
	30		P30U
	15	STA[1]	P15S
	30		P30S
	15	LTA[2]	P15L
	30		P30L

[1]: Short term aging; [2]: Long term aging

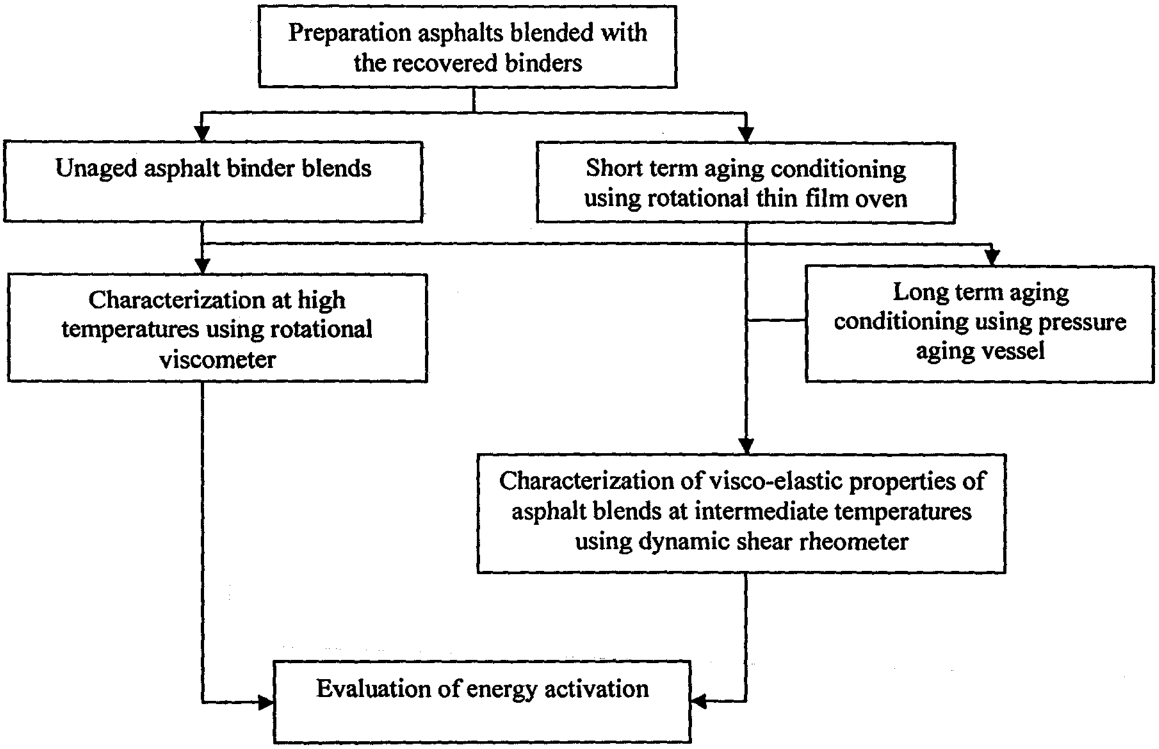


Figure 1: Flowchart of experimental procedures

2.2.2 Binder Aging Protocol

The control (without the recovered binder) and the asphalt blends were synthetically aged using the Rolling Thin Film Test (RTFO) and Pressure Aging Vessel (PAV) according to ASTM D 2872, (ASTM, 2006) and ASTM D 6521, (ASTM, 2006), respectively to simulate short term and long term aging.

2.2.4 Characterization at High Temperature

The effects of recovered binder source and contents on the rheological characteristics of the asphalt blends at high temperatures were evaluated in terms of viscosity. A Brookfield rotational Viscometer (RV) was used to measure the viscosities of all resultant asphalt binder blends according to the Asphalt Institute procedure (SP-1, 2003). The sweep temperature in the RV test was commenced at 125°C to 165°C at 10°C increments for unaged and short term aging conditions.

2.2.5 Characterization at Intermediate Temperature

The rheological properties, complex shear modulus (G*) and phase angle (δ), of the virgin and the asphalt blends were measured using the Dynamic Shear Rheometer (DSR) at intermediate temperatures. Temperature sweeps were applied from 52°C to 82°C at 6°C increments for the unaged and the short-term-aged samples, while Temperature sweep was from 13°C to 31°C at 3°C for the long-term-aged samples according to Superpave™ recommendations (SP-1, 2003).

2.2.6. Evaluation of Activation Energy

When a fluid flows, the layers of the fluid molecules slide over each other, while intermolecular forces resist the motion and cause resistance to flow (Haider et al., 2011). Therefore, in order to flow in the fluids, energy is required that should be higher than the intermolecular forces, called activation energy (AE). The higher AE indicates higher energy is required to flow. The Equation (1), called Arrhenius equation, was used to model the viscosity-temperature dependency of asphalt binders in this study.

$$\nu = Ae^{\frac{E_f}{RT}} \qquad (1)$$

Where v is viscosity (Pa.s), A is regression constant, E_f activation energy for flow(kJ/mol), T is temperature(°K) and R is universal gas constant(8.34 J/mol/°K).

3. Result and Discussion

3.1. Characterization at High Temperatures

3.1.1. Effects of RAP on Viscosity

Viscosity of a given asphalt binder depends on binder type, test temperature and aging conditions. Table 4 presents viscosity of unaged and short-term-aged asphalt binder blends at each test temperature. As can be seen in Table 4, asphalt binder blended with same recovered binder content from different sources show different viscosities at a given test temperature. For example, viscosity of N15U is 0.75 Pa.s, while that of D15U is 0.83 Pa.s at the same test temperature.

As expected, viscosity of the asphalt blends is higher than control asphalt sample at each aging condition and testing temperature. Table 4 also shows the asphalt blends containing recovered binder from of source DPE, which is highlighted in grey color, have the maximum viscosity for each recovered binder content, testing temperature and aging condition. Furthermore, the asphalt blends containing recovered binder from source NSE (N15U and N30U) have the same viscosity as the blends containing recovered binder from source PWD (P15U and P30U) at unaged condition for each recovered binder content and testing temperature. However, the viscosities of these blends are different for short term aging state.

Table 4: Viscosity of the asphalt binder blends

Temperature(°C)	Control	Viscosity (Pa.s)				
		N15U	N30U	[redacted]	P15U	P30U
125	0.54	0.75	1.00	[redacted]	0.75	0.99
135	0.34	0.45	0.58	[redacted]	0.45	0.58
145	0.22	0.29	0.36	[redacted]	0.29	0.35
155	0.15	0.19	0.23	[redacted]	0.19	0.23
165	0.10	0.13	0.16	[redacted]	0.13	0.16
		N15S	N30S	[redacted]	P15S	P30S
125	1.00	1.35	1.78	[redacted]	1.23	1.59
135	0.58	0.75	0.97	[redacted]	0.70	0.89
145	0.36	0.46	0.58	[redacted]	0.43	0.54
155	0.23	0.29	0.37	[redacted]	0.26	0.34
165	0.16	0.20	0.24	[redacted]	0.18	0.23

To investigate the effects of one unit percentage of recovered binder content (1%) from each RAP source on the increase in binder viscosity at each test temperature, a non-dimensional viscosity index at high temperature is used, as defined in Equations (2) and (3).

$$\eta_{RAP} = \left(\frac{\nu}{\nu_v}\right) \tag{2}$$

$$\nabla\eta_{RAP} = \left[\frac{\partial\eta_{RAP}}{\partial RAP}\right] = \left[\frac{\Delta\eta_{RAP}}{\Delta RAP}\right] = \left[\frac{\eta_{RAP} - \eta_v}{RAP}\right] \tag{3}$$

where η_{RAP} is the relative viscosity of asphalt blend, RAP is the recovered binder content, ν is the binder viscosity and ν_v is the binder viscosity at initial, virgin or control condition, while $\nabla\eta_{RAP}$ is the non-dimensional viscosity index.

Table 5 presents the ANOVA results for $\nabla\eta_{RAP}$ of the binder blends samples. The data indicates that aging states, recovered binder content and recovered binder source have significant effects on the $\nabla\eta_{RAP}$ values. It implies that $\nabla\eta_{RAP}$ can be a viable parameter to study the rheology of binder incorporating different amounts of recovered binder from various sources depending on aging conditions and test temperatures.

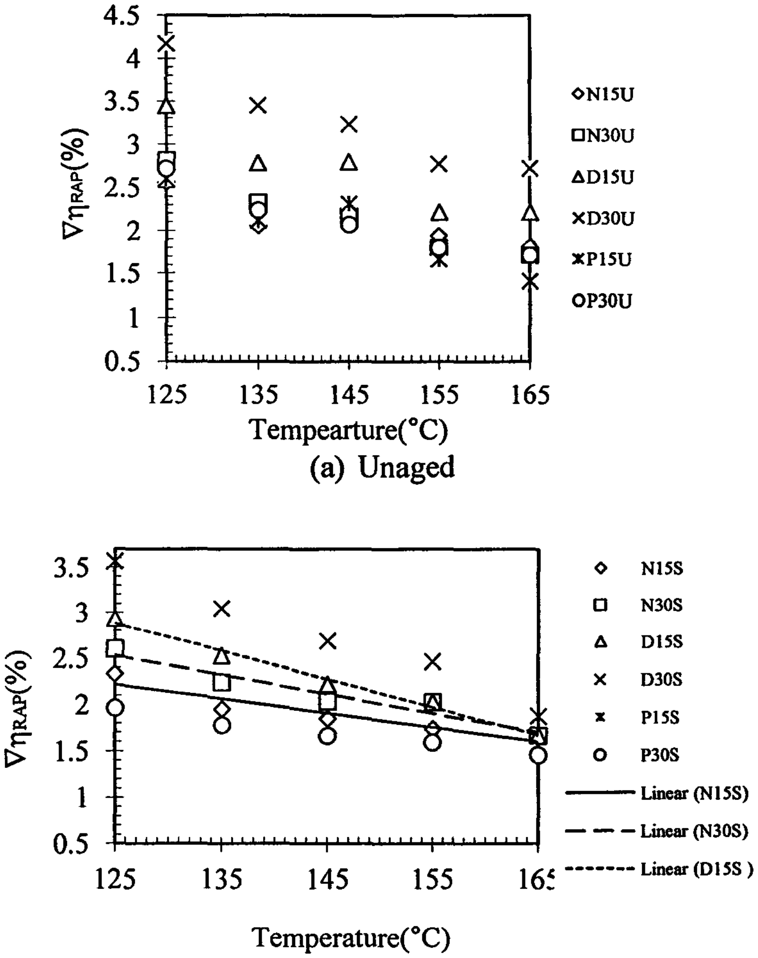
Table 5: ANOVA results of $\nabla\eta_{RAP}$

Source	DF	Seq SS	Adj SS	Adj MS	F	P	Significant
C1	2	38.15	38.15	19.08	374.36	<0.001	Yes
C2	4	26.30	26.30	6.57	129.01	<0.001	Yes
C3	1	8.79	8.79	8.79	172.48	<0.001	Yes
C4	2	18.28	18.28	9.14	179.34	<0.001	Yes
C1*C2	8	2.84	2.84	0.35	6.97	<0.001	Yes
C1*C3	2	2.25	2.25	1.12	22.08	<0.001	Yes
C1*C4	4	3.16	3.16	0.79	15.45	<0.001	Yes
C2*C4	8	1.82	1.82	0.22	4.48	<0.001	Yes
C1*C3*C4	4	1.39	1.39	0.34	6.83	<0.001	Yes
Error	180	180	9.17	0.051			
Total	269	114.33					

S=0.22, R-Sq=91.98%, R-Sq(adj)=88.01%

C1: Recovered Source; C2: Test Temperature; C3: Recovered binder Contents; C4: Aging

Figures 2(a) and (b) show the $\nabla\eta_{RAP}$ trends at the sweep temperatures for unaged and short termaging conditions, respectively. The $\nabla\eta_{RAP}$ values for N15U, D15U, and P15U are 2.56%, 3.44%, and 2.60% at 125°C, respectively as indicated in Figure (2a). It indicates that incorporation of 1% recovered binder increases the relative viscosity by 2.56%, 3.44%, and2.60% depending on the recovered sources at 125°C.



(a) Unaged

(b) Short-term-aged

Figure 2: Relationship between non-dimensional viscosity index and temperature of asphalt blends

Although 1% recovered binder increases the relative binder viscosity by 2.56% at unaged state for N15U sample at 125°C, 1% RAP binder content increased the relative binder viscosity by 2.8% at the unaged state, at 125°C for N30U binder. It implies that the performance of 1% recovered binder in various proportion (15% and 30%) can be different even for the same recovered binder source and at the same temperature in terms of $\nabla\eta_{RAP}$. This phenomenon is also observed for other asphalt blends at different aging states and test temperatures. Set Equation 3 presents $\nabla\eta_{RAP}$ trends as functions of temperature for unaged condition based on different recovered binder sources and contents. The performance of 1% recovered binder is found to be varied at different temperatures. For example; $\nabla\eta_{RAP}$ for N15U is equivalent to 2.56% and 1.94% at 125°C and 155°C, respectively. This reduction in $\nabla\eta_{RAP}$ by 24% when temperature increases from 125°C to 155°C implies the extent of temperature effects on decreasing the relative viscosity of the asphalt blends samples. This phenomenon can also be seen at different aging states. The $\nabla\eta_{RAP}$ values at different temperatures can exhibit their own rheological trend over the sweep temperatures in set Equations (4) and (5) for unaged and short term aging conditions, respectively. Set Equation (5) indicates that the trend of $\nabla\eta_{RAP}$ versus temperature is linear regardless of the recovered binder source and content.

$$(\nabla\eta_{RAP})_{Un} = f(T,RAP) = \begin{cases} -0.0161T + 4.457 & \text{for N15U} \quad R^2 = 77\% \\ -0.0273T + 6.121 & \text{for N30U} \quad R^2 = 95\% \\ -0.0302T + 7.078 & \text{for D15U} \quad R^2 = 88\% \\ -0.0354T + 8.461 & \text{for D30U} \quad R^2 = 91\% \\ -0.0284T + 6.114 & \text{for P15U} \quad R^2 = 90\% \\ -0.0244T + 5.652 & \text{for P30U} \quad R^2 = 94\% \end{cases} \quad 125°C \le T \le 165°C \quad (4)$$

The effects of recovered binder source can be observed from the slope of the linear equations and the Y-intercept. For instance, the gradient of the equation for D15U is -0.0302, which is 1.87 and is 1.06 times higher than the gradients for N15U and P15U samples, respectively. The value of $\nabla\eta_{RAP}$ after subjected to short term aging decreases irrespective of test temperature and recovered binder source but the linear relationships between $\nabla\eta_{RAP}$ and temperature remain unchanged. From Figure (2b), the $\nabla\eta_{RAP}$ for N15S, D15S and N30S converge to 1.66 as the temperature increases to 165°C. It implies that at 165°C, the performance of these asphalt blends in terms of $\nabla\eta_{RAP}$, are similar.

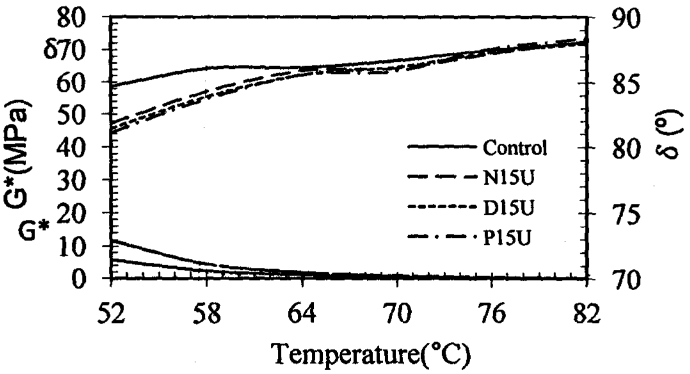
Similar to Set Equations 4, Set Equations 5 presents a linear reduction trends irrespective of the recoved binder source and content. It also indicates that the short term aging process has no effects on the trends of $\nabla\eta_{RAP}$.

$$(\nabla\eta_{RAP})_{Sta} = f(T, RAP) = \begin{cases} -0.0155T + 4.154 & \text{for} \quad \text{N15S} \quad R^2 = 87\% \\ -0.0208T + 5.129 & \text{for} \quad \text{N30S} \quad R^2 = 92\% \\ -0.0303T + 6.674 & \text{for} \quad \text{15DS} \quad R^2 = 98\% \quad 125°C \le T \le 165°C \\ -0.0397T + 8.477 & \text{for} \quad \text{30DS} \quad R^2 = 96\% \\ -0.0121T + 3.439 & \text{for} \quad \text{P15S and P30S} \quad R^2 = 98\% \end{cases} \quad (5)$$

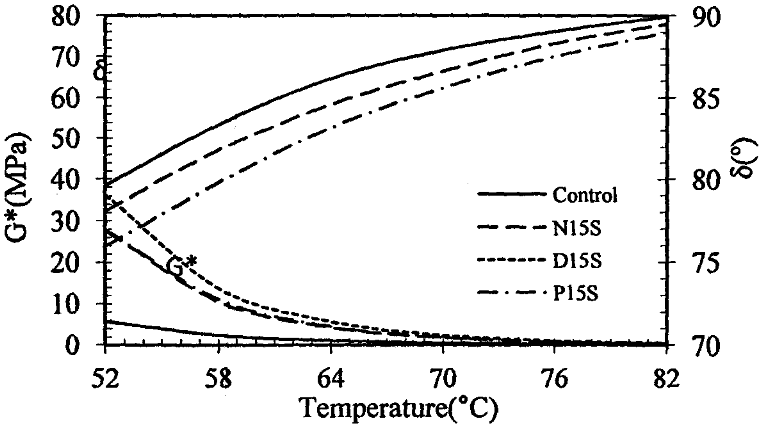
3.2. Characterization at Intermediate Temperatures

3.2.1. Visco-Elastic Properties

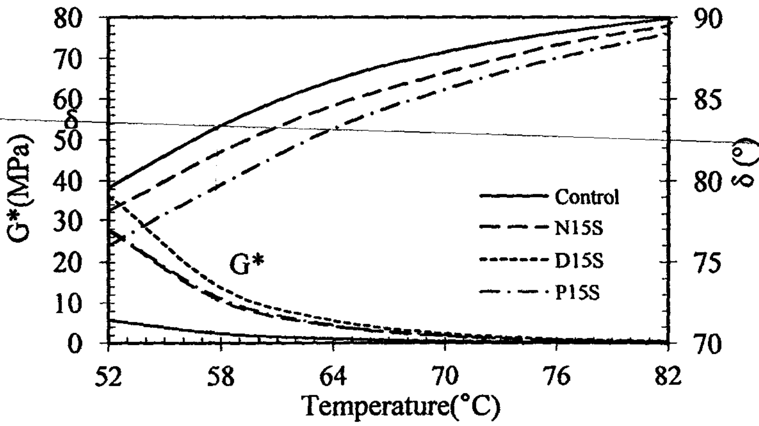
The effects of the recovered binder sources and contents on the complex shear modulus (G*) and phase angle (δ) of the virgin and asphalt blends at various temperatures are illustrated in Figures 3 and 4. The binders that were subjected to unaged and short term aging conditions exhibit similar trends. The values of G* and δ exhibit polynomial and power trends, respectively with R^2 more than 0.90 at every aging state. It implies that the recovered binder sources and contents have no effects on the observed trends of G* and δ at the sweep temperatures.



(a) Unaged

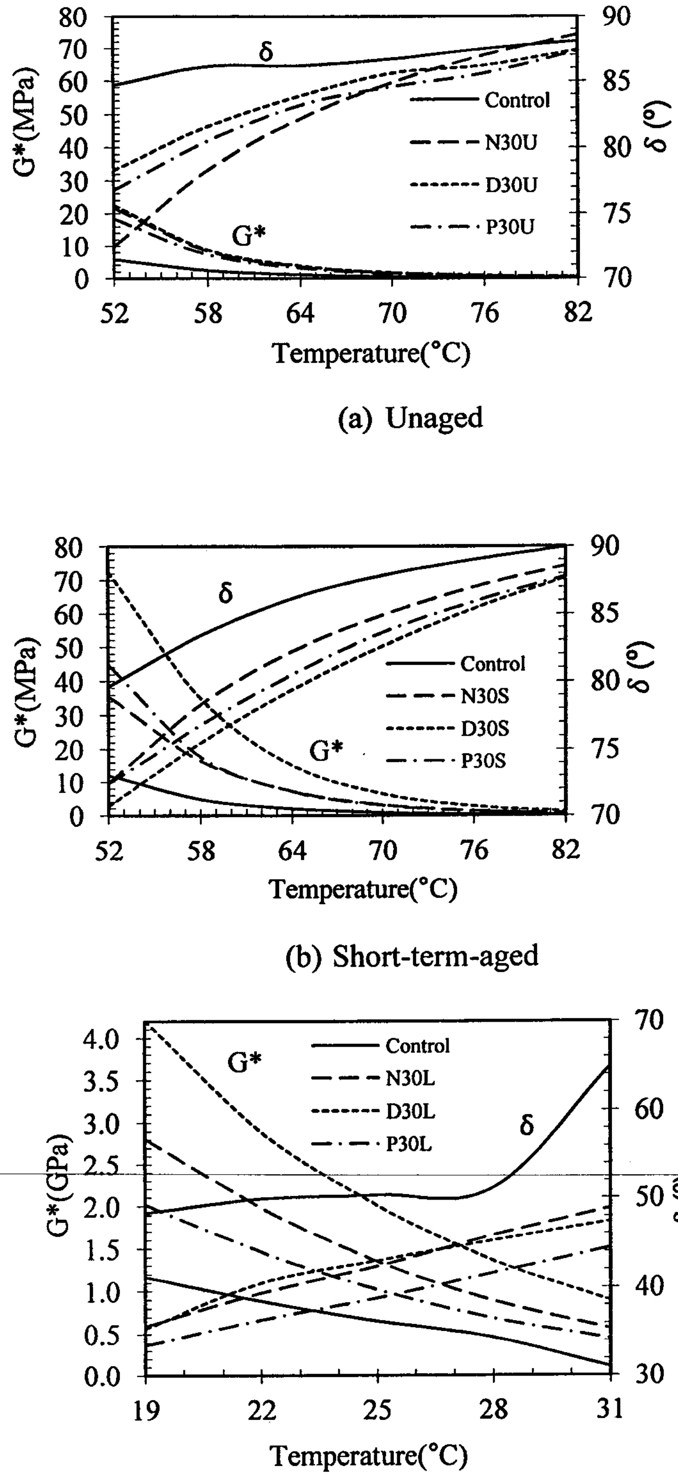


(b) Short-term-aged



(c) Long-term-aged

Figure 3: Relationship between complex shear modulus, and phase angle-temperature dependency of the asphalt blends containing 15% recovered binder.



(a) Unaged

(b) Short-term-aged

(c) Long-term-aged

Figure 4: Relationship between complex shear modulus, and phase angle-temperature dependency of the asphalt blends containing 30% recovered binder.

The phase angle (δ) of the control sample subjected long term aging increases abruptly beyond 28°C. Equation (6) presents the rheological trend of G* for virgin sample subjected to long term aging.

$$G^* = 0.043T^3 - 3.057T^2 + 71.30T - 500.53 \qquad R^2 = 0.99 \qquad 52 \leq T \leq 82 \qquad (6)$$

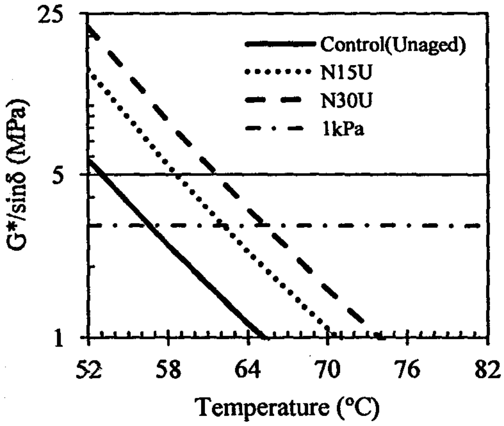
The significant increase in δ at the sweep temperature indicates the presence of a directional change in gradient of the G* to the temperature relationship or $\left(\frac{\partial\left(\frac{\partial G^*}{\partial T}\right)}{\partial T}\right)$ and also known as the inflection point. From Equation (7), the inflection point takes place at 23.7°C. However, such trend is not observed in the asphalt blends.

$$\frac{d^2G^*}{dT^2} = 0 \quad \Rightarrow \quad T = 23.70^{\circ}C \qquad (7)$$

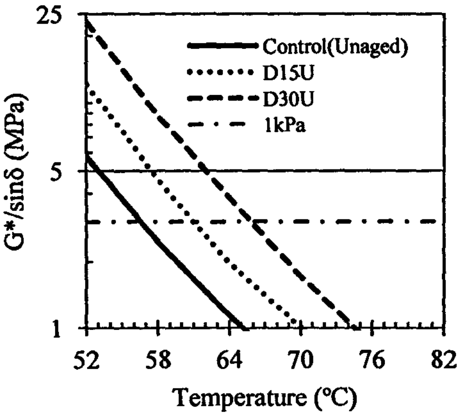
3.2.2. Rutting parameter

According to Superpave™, rutting is controlled by limiting the G*/sin δ to a value greater than 1 kPa and 2.20 kPa for unaged and short term aged binders, respectively (SP-1, 2003). The increase in G*/sin δ means improvement in rutting resistance of an asphalt mixtures under operational conditions of heavy traffic loadings and at high service temperatures (Sherwood et al. 1998; Stuart and Izzo, 1995). Therefore, a higher value is more desirable from the viewpoint of resistance to fatigue failure.

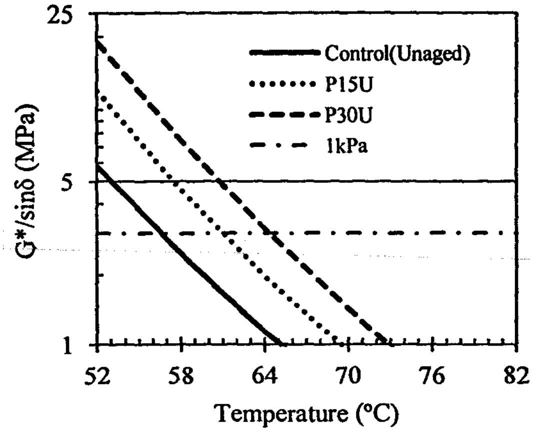
Figures 5 and Figure 6 show the effects of recovered binder on G*/sin δ of the asphalt blends for unaged and short term aging conditions. As indicated in Figures 5 and 6, it can be clearly seen that adding recovered binder, irrespective of the source, improve $G^*/\sin\delta$ (above dashed-pointed straight line in Figures 5 and 6) of asphalt binders at unaged and short term aging conditions, respectively. The range of 1 kPa and 2.2 kPa for unaged and short term aging conditions are highlighted by dash-pointed straight line in Figures 5 and 6.



(a) NSE source

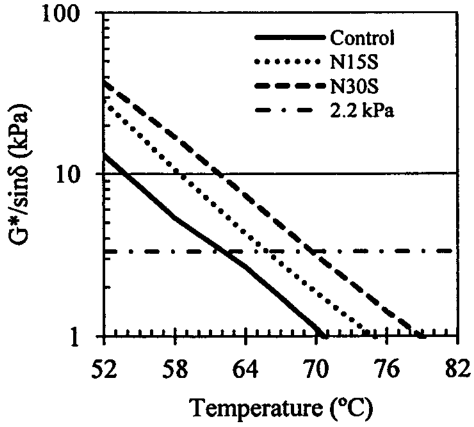


(b) DPE source

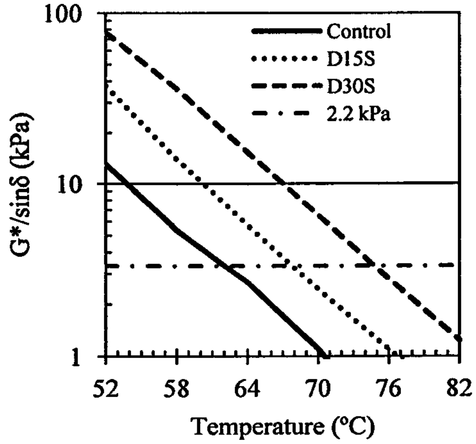


(d) PWD source

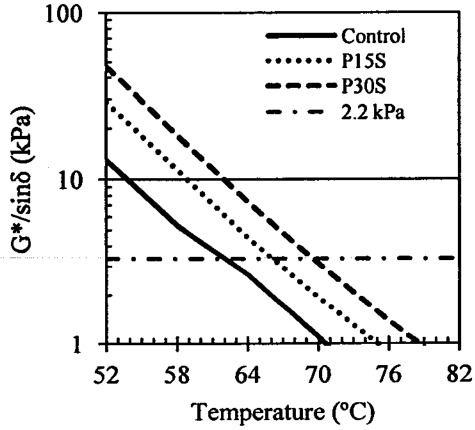
Figure 5: G*/sinδ for the unaged asphalt blends



(a) NSE source



(b) DPE source



(b) PWD source

Figure 6: G*/sinδ for the short-term-aged asphalt blends

To evaluate the increase in rutting parameter to a unit of recovered binder content (1%) at each test temperature and characterize the trend of each recovered binder source, the relative Superpave™ rutting factor gradient (RSRG) is calculated using Equations (8) and (9).

$$RSR=\frac{[G^*/Sin\delta]_{RAP}}{[G^*/Sin\delta]_v} \tag{8}$$

$$RSRG=\frac{\partial RSR}{\partial RAP}=\left[\frac{\Delta RSR}{\Delta RAP}\right]=\left[\frac{RSR_{RAP}-RSR_v}{RAP}\right] \tag{9}$$

where RSRG is the relative Superpave™ rutting factor; $[G^*/Sin\delta]_{RAP}$ is the Superpave™ rutting factor for the asphalt blends; $[G^*/Sin\delta]_v$ is for the virgin samples. Table 6 presents the ANOVA results for RSRG of all asphalt blend samples tested. Figure 7 illustrates Relationship between RSRG and temperature of the asphalt blends.

Table 6: ANOVA results of RSRG

Source	DF	Seq SS	Adj SS	Adj MS	F	P	Significant
C1	2	787.55	787.55	393.77	2249.65	<0.001	Yes
C2	5	145.185	145.185	29.037	165.89	<0.001	Yes
C3	1	91.08	91.08	91.08	6296.54	<0.001	Yes
C4	1	1102.14	1102.14	1102.14	6296.54	<0.001	Yes
C1*C2	10	6.822	6.822	0.682	3.9	<0.001	Yes
C1*C3	2	287.67	287.67	143.835	821.73	<0.001	Yes
C1*C4	2	769.932	769.932	384.966	2199.31	<0.001	Yes
C2*C3	5	18.58	18.58	3.716	21.23	<0.001	Yes
C2*C4	5	65.88	65.88	13.176	75.27	<0.001	Yes
C3*C4	1	40.922	40.922	40.922	233.79	<0.001	Yes
C1*C2*C3	10	12.081	12.081	1.208	6.9	<0.001	Yes
C1*C2*C4	10	13.268	13.268	1.327	7.58	<0.001	Yes
C1*C3*C4	2	77.44	77.44	38.72	221.21	<0.001	Yes
C2*C3*C4	5	12.446	12.446	2.489	14.22	<0.001	Yes
C1*C2*C3*C4	10	10.868	10.868	1.087	6.21	<0.001	Yes
Error	144	25.20	25.20	0.175			
Total	215	3467.01					

S=0.41, R-Sq=99.27%, R-Sq(adj)=98.91%

C1: Recovered binder source; C2: Test Temperature; C3: Recovered binder Contents; C4: Aging

Although the RSRG values are not similar for different sources at unaged conditions, all samples RSRG values decrease linearly as temperature increase. However, D30U is an exception whose average RSRG remains a constant at 6.3 and independent of temperature. Therefore, two trends are observed for RSRG at unaged condition as indicated in Set Equation (10).

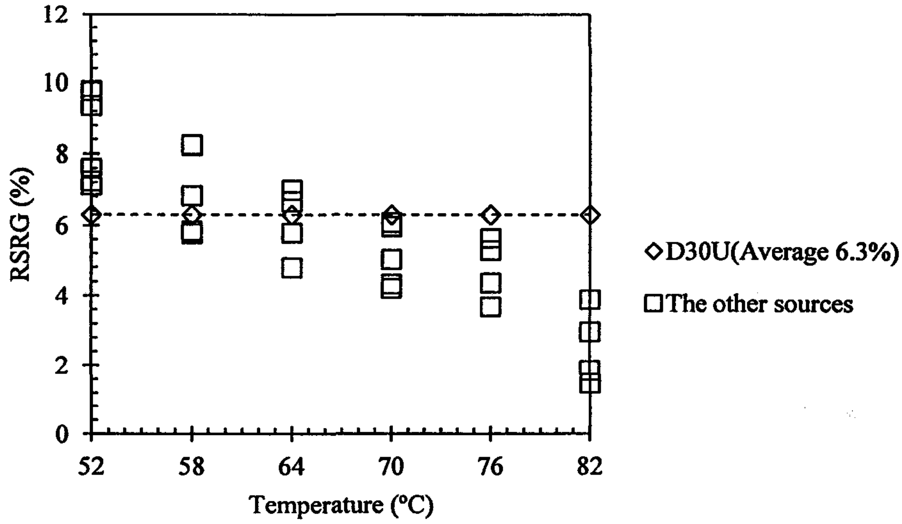


Figure 7: Relationship between RSRG and temperature of the asphalt binder blends

$$\text{RSRG} = f(T, \text{RAP}) = \begin{cases} 6.3\% & \text{for D30U} \\ -0.17T + 17.03 & \text{for the other sources} \end{cases} \qquad 52 \le T \le 82 \qquad \text{For unaged (10)}$$

where T is temperature.

There is no obvious trend for asphalt blends subjected to short term aging. However, it can be observed that the amorphous pattern is similar for all the asphalt blend samples. The RSRG can vary for the asphalt blends containing different recovered binder quantities from the same source at a given aging state. For instance, the RSRG equals to 9.77% for N15U at 52°C, whereas the corresponding value is 9.36% for N30U. Hence, the differences in the amount of RSRGs are not significant. However, 1% recovered binder could have different effects on increasing the relative Superpave™ rutting factor from the same source irrespective of the aging state. The same phenomenon can be seen in the binder characterization at high temperatures in terms of $\nabla\eta_{RAP}$.

3.2.4. Failure temperature

The failure temperature corresponds to the temperature when G*/sin δ is less than 1 kPa and 2.2 kPa for unaged and short term aged asphalt binder, respectively (SP-1, 2003). Figure 8 shows the high failure temperature of unaged (Ft_U) and short term aged asphalt blends (Ft_S). Equation (11) presents a linear trend of the high failure temperatures for unaged and short term aged asphalt blends.

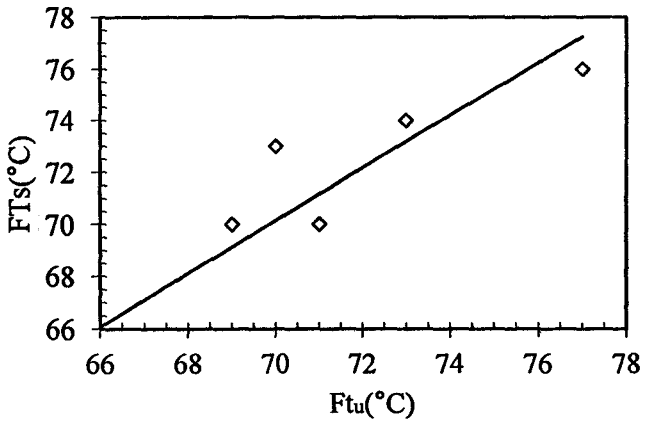


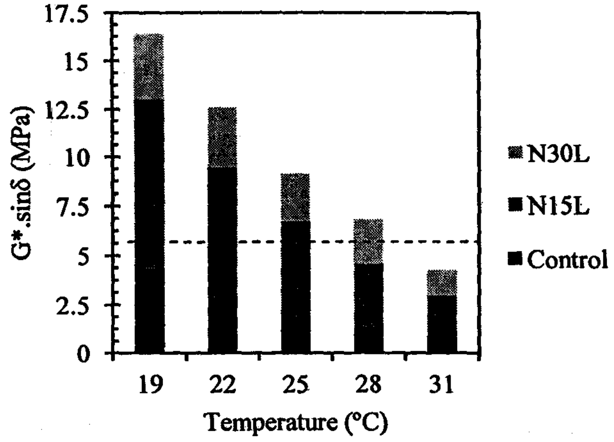
Figure 8: Relationship between the high failure temperature of unaged and short-term-aged

$$FT_S = 1.01FT_U - 0.85 \qquad R^2 = 0.81 \qquad (11)$$

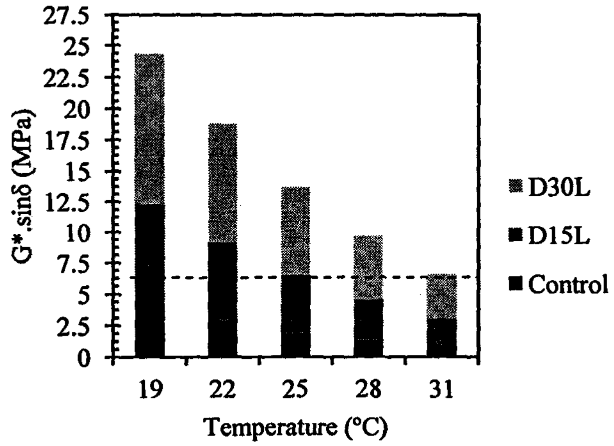
where FT_S and FT_U are the high failure temperatures at short term aged and unaged conditions, respectively. The results obtained are consistent with the findings of other researchers who studied other binder additives such as Sasobit® and crumb rubber (Akisetty et al., 2009).

3.2.5. Fatigue Parameter

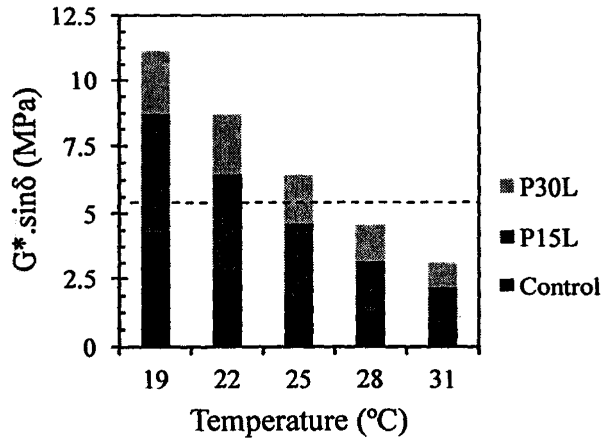
Incorporating recovered binder increases its $G^*/\sin\delta$, making it stiffer and subsequently subjecting the asphalt pavement to fatigue failure especially at low temperature. According to Superpave™ recommendations (SP-1, 2003), the value of $G^*\sin\delta$ should be less than 5 MPa or the area located below the dashed horizontal line shown in Figures 9.



(a) Source NSE



(c) Source DPE



(e) Source PWD

Figure 9 : G*.sinδ for the asphalt blends

A lower value is more desirable from the viewpoint of resistance to fatigue cracking. As anticipated, G*sin δ increases as the recovered binder content increases, irrespective of source. However, amount of G*sin δ values differ according to recovered binder source. For instance, Figures (7a) and (7c) show that N30L and P30L fulfill the Superpave™ requirements for fatigue parameter at 31°C, while the asphalt blend D30L can not satisfy at the same testing temperature. In order to characterize the effect of recovered binder in fatigue parameter to a unit of recovered binder content (1%) at each test temperature, the relative Superpave™ fatigue factor gradient (RSFG) is calculated using Equations (12) and (13) based on Equations (8) and (9).

$$\mathrm{RSF}=\frac{[G*\mathrm{Sin}\delta]_{\mathrm{RAP}}}{[G*\mathrm{Sin}\delta]_{\mathrm{v}}} \tag{12}$$

$$\mathrm{RSFG}=\frac{\partial \mathrm{RSF}}{\partial \mathrm{RAP}}=\left[\frac{\Delta \mathrm{RSF}}{\Delta \mathrm{RSP}}\right]=\left[\frac{\mathrm{RSF}_{\mathrm{RAP}}-\mathrm{RSF}_{\mathrm{v}}}{\mathrm{RAP}}\right] \tag{13}$$

where RSFG is the relative Superpave™ rutting factor; $[G*.\sin\delta]_{\mathrm{RAP}}$ is the Superpave™ rutting factor for the recovered binder; $[G*.\sin\delta]_{\mathrm{v}}$ is for the virgin samples. Table 7 presents the ANOVA results for RSFG of all asphalt blends tested.

Figure 10 shows relationship between RSFG and temperature of the asphalt blends. It can be seen that RSFG increases linearly as temperature increases for each recovered binder content and source. Based on the observed results, Set Equation (14) of a multiple function is valid for RSFG of asphalt blends. Like Set Equations (4) and (5), changes in slope and the Y-intercept can be considered as an effect of aging on the RSFG trends.

Table 7: ANOVA results of RSFG

Source	DF	Seq SS	Adj SS	Adj MS	F	P	Significant
C1	2	787.55	787.55	393.77	2249.65	<0.001	Yes
C2	5	145.185	145.185	29.037	165.89	<0.001	Yes
C3	1	91.08	91.08	91.08	6296.54	<0.001	Yes
C1*C2	10	6.822	6.822	0.682	3.9	<0.001	Yes
C1*C3	2	287.67	287.67	143.835	821.73	<0.001	Yes
C2*C3	5	18.58	18.58	3.716	21.23	<0.001	Yes
C1*C2*C3	10	12.081	12.081	1.208	6.9	<0.001	Yes
Error	144	25.20	25.20	0.175			
Total	215	3467.01					

S=0.41, R-Sq=99.27%, R-Sq(adj)=98.91%

C1: Recovered binder source; C2: Test Temperature; C3: Recovered binder Contents;

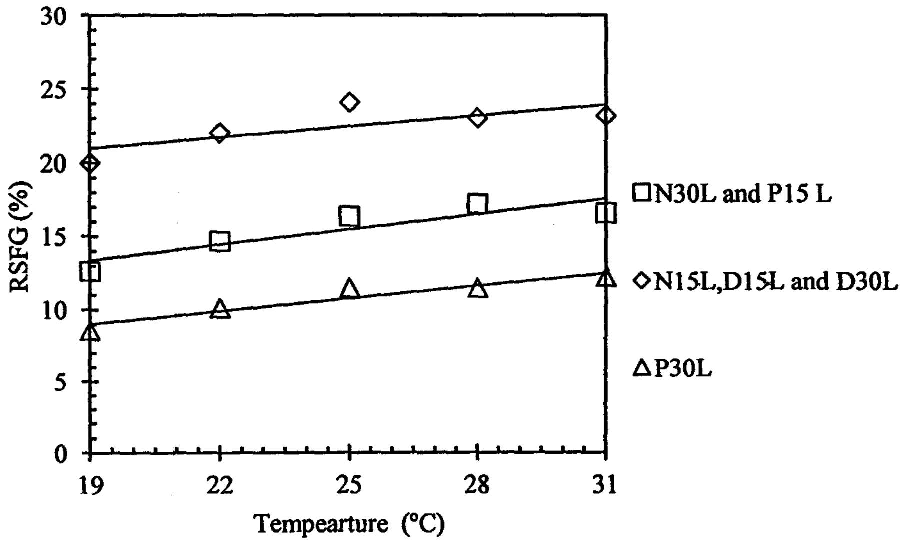


Figure 10: Relationship between RSFG and temperature of the asphalt binder blends

$$\mathrm{RSFG}=\begin{cases} 0.24T + 16.37 \quad R^2 = 0.55 \quad \text{for N15L, D15L and D30L} \\ 0.35T + 6.74 \quad R^2 = 0.79 \quad \text{for N30L and P15L} \\ 0.29T + 3.55 \quad R^2 = 0.90 \quad \text{for P30L} \end{cases} \qquad 19 \le T \le 31 \qquad (14)$$

The results indicate that the increase in amount of G*.sinδ per 1% recovered binder depends on the source and temperatures tested, likewise RSRG. For example, value of RSFG is almost 22% at 22°C for N15L and D15P, while the corresponding value for P15L is 15% at the same test temperature. It means that the increase in amount of G*.sinδ by 1% recovered binder added is 22% for N15L and D15L, while it amount of the increase is 15% for P15L at 22°C, this 7% difference can be considered as effect of various reference per 1% recovered binder in terms of G*.sinδ.

3.2.6. Energy Activation

3.2.6.1. Activation energy based on viscosity

Tables 8 and 9 present the energy activation computed using viscosity for the unaged and short-term-aged asphalt blends, respectively.

Table 8: Energy activation for the asphalt blends

Unaged			Short-term-aged		
Asphalt binder	Energy(kj/mol)	Change (%)	Asphalt binder	Energy (kj/mol)	Change (%)
Control	60.76	-	Control	66.63	-
N15U	63.36	4.26	N15S	69.24	3.93
N30U	66.63	9.64	N30S	72.15	8.29
D15U	64.66	6.41	D15S	71.54	7.37
[redacted]					
P15U	63.75	4.91	P15S	70.15	5.29
P30U	66.34	9.18	P30S	70.10	5.20

From Tables 8, it can be seen that the asphalt blends show higher energy activation in comparison with control binder for each aging state. It takes place because the recovered binder is aged binder, containing a lot of Asphaltene, making the stiffer asphalt binder. Consequently, higher energy activation is required to capture intermolecular forces for the asphalt binder blends. For example, the energy activation for N15U is 63.36 kj/mol, while it is 66.63 for N30U, as indicated in Table 8.

Although incorporating recovered asphalt binder in the asphalt binder increase the energy activation, amount of increase in the energy activation depends on the recovered binder source. For instance, Table 9 presents that percent of change in the energy activation for D15U is 6.41%, while it is 4.91 % for P15U. Table 6 also shows that the maximum energy activation as well as maximum change percent in energy activation is for D30U and D30S samples. The N15U and N15S asphalt blends show the minimum energy activation.

Aging is another factor influence on the energy activation, as shown in Table 8, the energy activation of short-term-aged asphalt blends increases compared to unaged asphalt blends. Amount of changing in the energy activation is show in Table 9.

Table 9: Difference in activation energy values and change percentage due to short-term-aging

Asphalt binder	Difference in Energy(kj/mol)	Change (%)
Control	5.86	+9.64
N15	5.88	+9.29
N30	5.52	+8.29
D15	6.87	+10.63
D30	8.62	+12.76
P15	6.40	+10.04
P30	3.74	+5.65

Interaction available Asphaltane in the recovered binder and new produced Asphaltane after short term aging increases intermolecular resistance forces to flow. That is why the energy activations of all the short-term-aged asphalt blends are higher than the unaged blends. However amount increase in the energy activation depends on recovered binder source, as presented in Table 9.

3.2.6.2. Activation energy based on G*/sin δ

Table 11 shows the AE values based on G*/sin δ. It can be seen that AE increases as recovered binder is added to the asphalt binder. However, amount of increase in AE depends on recovered binder source and content.

Table 10: Energy activation for the asphalt blends

Unaged			Short-term-aged		
Asphalt binder	Energy(kj/mol)	Change (%)	Asphalt binder	Energy (kj/mol)	Change (%)
Control	111873	-	Control	127353	-
N15U	127279	13.77	N15S	135476	6.37
N30U	128692	15.03	N30S	129681	1.82
D15U	126372	12.96	D15S	136183	6.93
D30U	127902	14.32	D30S	133148	4.55
P15U	125641	12.30	P15S	135676	6.53
P30U	128584	14.93	P30S	137281	7.76

Table 10 also presents that the maximum AE values for each aging state is P30U and P30S, as highlighted by grey color, while Table 8 shows the maximum AE value is for D30U and D30S. It implies that the maximum amount of AE values depends on rheological parameter selected to study AE. Furthermore, the temperature sweep of DSR to find G*/sin δ is different from that of RV test.

As similar as AE values based on RV results, amount of AE increases after aging because of higher stiffness in the short-term-aged asphalt blends as indicated in Table 10. From the AE values in Table 10, it can be seen that amount of increase in AE values also depend on RAP source. Table 11 also shows that the minimum change in AE is for N30 that is less than 1%. It means that N30 asphalt binder is not susceptible to short term aging in terms of AE.

Table 11: Difference in activation energy values and change percentage due to short-term-aging

Asphalt binder	Difference in Energy(kj/mol)	Change (%)
Control	5.86	+13.83
N15	5.88	+6.44
N30	5.52	+0.76
D15	6.87	+7.76
D30	8.62	+4.10
P15	6.40	+7.98
P30	3.74	+6.72

3.2.6.3. Activation energy based on G*.sin δ

Table 12 shows AE values based on G*.sin δ., the AE values based G*.sin δ decrease. It can be seen that the AE values decrease by adding the recovered binder, while AE values of the asphalt blends computed using viscosity and G*/sin δ increase. The reason behind such decreasing trend is that AE in Table 12 are computed based on G*.sin δ. Therefore, the lower AE values can mean that less energy activation is required to happen flow in terms of fatigue cracking as recovered binder is added in asphalt binder.

Table 12: Energy activation for the asphalt blends

Asphalt binder	AE (kj/mol)	Difference in AE (kj/mol)	Change (%)
Control	98903.34	-	-
N15	90556.09	-8347.26	-8.43981
N30	81377.43	-17525.9	-17.7202
D15	86340.89	-12562.5	-12.7017
D30	80770.51	-18132.8	-18.3339
P15	86058.21	-12845.1	-12.9876
P30	78251.37	-20652	-20.881

From Table 12, the N15L has the maximum AE values based on G*.sin δ, while D30U and P30U showed the maximum AE values in terms of viscosity and G*/sin δ, respectively.This can be attributed to the difference in DSR and RV test nature. It means that RV is designed to measure asphalt binder shear resistance to flow, while the DSR measure the visco-elastic properties. Furthermore, the asphalt binder conditions for RV and sweep temperature to evaluate G*/sin δ are unaged and short-term-aged, while the condition to evaluate G*.sin δ is long-term aging. In addition, the temperatures tested of G*.sin δ are lower than those of G*.sin δ and viscosity.

4. Conclusion

At high temperatures, the rheological properties of virgin asphalt binder incorporating recovered binder was quantified in terms of $\nabla\eta_{RAP}$. The increase in relative viscosity per 1% recovered binder content depended on RAP source and proportion at a particular temperature. At temperatures ranging from 125°C to 165°C, $\nabla\eta_{RAP}$ reduces linearly with temperature for each aging condition irrespective of the binder sample blended. However, sample N15L was an exception.

At intermediate temperatures, the rheological properties of asphalts blended with recovered binder were characterized in terms of the relative Superpave™ rutting factor gradient (RSRG) and the relative Superpave™ fatigue factor gradient (RSFG). From the results, the increase in relative Superpave™ rutting and fatigue factors per 1% recovered binder are dependent on source and content at each test temperature. However, the RSRG of D30U is independent of temperature. The results also showed that RSFG increases linearly as temperature increases for each recovered binder source and content. The results observed for RSFG at the sweep temperature showed that N30L and P15L showed the similar trend. Furthermore, D15L and also showed the same trend as N15L and D30L, indicating increase in relative Superpave™

rutting and fatigue factors per 1% RAP can be the same for asphalt binders incorporating different amount of RAP from various sources.

Activation energy was also studied for all the asphalt blends. The results indicated that mount of AE can be various based on recovered binder source and content. The evaluation of Activation energy also showed that adding the recovered changed activation energy for each recovered binder source and content. The values of changed can be positive and negative, depending on the asphalt binder rheological property selected to compute AE. As viscosity and G*/sin δ were chosen to find AE, amount of AE and change percentages were positive by adding the recovered binder. While, if G*.sin δ is chosen, AE values of the asphalt blends would be negative negative.

5. Suggestions for further research

The suggestions for further research on this subject include:

- Since Amirkhanian and Williams (1993) found that mixtures prepared using moisture-damaged RAP material increase indirect tensile strength (ITS) and resilient modulus under wet and dry conditioning, evaluation of engineering properties of asphalt mixtures containing moisture-damaged RAP materials from various sources can be an interesting issue.
- Low temperature properties of asphalt blends containing recovered binder from different sources are also recommended to study.

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References

Akisetty, C. K., Lee, S. J. and Amirkhanian, S. N. (2009) High Temperature Properties of Rubberized Binders Containing Warm Asphalt Additives. Construction and Building Materials, 23(1), 565-573.

Amirkhanian, S. N. and Williams, B. (1993) Recyclability of Moisture Damaged Flexible Pavements. Journal of Materials in Civil Engineering, 5(4), 510-530.

ASTM D 2872: Standard Test Method for Effect of Heat and Air on Moving Film of Asphalt (Rolling Thin-Film Oven Test). West Conshohocken, PA, USA; 2006.

ASTM D6521: Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV). West Conshohocken, PA, USA; 2006.

Druta, C., Wang, L. and Zhu, T.(2009).Laboratory Investigation of Reclaimed Asphalt Pavement Mixed with Pure Binder Using X-Ray CT Scanner,9th International Conference of Chinese Transportation Professionals (ICCTP),Harbin, China

Haider, S. W., Mirze, M. W., Thottempudi, A. K., Bari, J. and Baladi, G. Y.(2011).Characterizing Temperature Susceptibility of Asphalt Binders Using Activation Energy Flow,Transportation and Development Institute Congress 2011: Integrated Transportation and Development for a Better Tomorrow,Chicago, Illinois, USA.

Hassan, M.(2009).Life-Cycle Assessment of Warm-Mix Asphalt: An Environmental and Economic Perspective, Proceedings of 88th Transportation Research Board Annual Meeting, Washington D.C, USA

Jamshidi, A.(2012) Rheological Properties of Asphalt Binders, Performance and Sustainability of Warm Mixtures Asphalt Incorporating Sasobit®. PhD thesis, Universiti Sains Malaysia(USM).

Jamshidi, A., Hamzah, M. O. and Shahadan, Z. (2012) Selection of Reclaimed Asphalt Pavement Sources and Contents for Asphalt Mix Production based on Asphalt Binder Rheological Properties, Fuel requirements and Greenhouse Gas Emissions. Journal of Cleaner Production, 23(1), 20-27.

Jamshidi, A., Hamzah, M. O. and You, Z. (2013) Performance of Warm Mix Asphalt containing Sasobit®: State-of-the-art. Construction and Building Materials, 38(530-553).

Keches, C. and LeBlank, A.(2007)Reducing Greenhouse Gas Emissions from Asphalt Materials.Worcester Polytechnic Institute, USA.

Kim, R. Y. (2009) Modeling of Asphalt Concrete, McGraw-Hill.

Lee, J. C., Edil, T. B., Tinjum, J. M. and Benson, C. H. (2010) Quantitative Assessment of Environmental and Economic Benefits of Recycled Materials in Highway Construction. Transportation Research Record: Journal of the Transportation Research Board, 2158(1), 138-142.

Roberts, F. L., Mohammad, L. N. and Wang, L. B. (2002) History of Hot Mix Asphalt Mixture Design in the United States. Journal of Materials in Civil Engineering, 14(4), 279-293.

Sherwood J.A, Thomas N.L, Qi X. Correlation of Superpave G*/sin δ with rutting Test Results from Accelerated Loading Facility. Transportation Res Rec: J Transportation Res Board. 1998; 1630: 53–61.

SP-1 (2003) Superpave Performance Graded Asphalt Binder Specifications and Testing, Asphalt Institute, KY, USA.

Stuart K.D and Izzo R.P. Correlation of Superpave G*/sin δ with rutting susceptibility from laboratory mixture tests. Transportation Research Board, National Research Council, Washington D.C. USA; 1995.

Proceedings of the First Makassar International Conference on Civil Engineering (MICCE2010), March 9-10, 2010, ISBN 978-602-95227-0-9

EFFECTS OF TEMPERATURE AND BINDER TYPE ON THE RESILIENT MODULUS PROPERTIES OF POROUS ASPHALT SUBJECTED TO AGEING

Che Wan C.N [1], Hamzah M.O [2] and Ramadhansyah P.J [3]

ABSTRACT: The objective of this paper is to evaluate the effects of temperature and binder type on the resilient modulus properties of porous asphalt subjected to ageing. Specimens were subjected to short term and long term ageing according to AASHTO R30-02 (AASHTO, 2002) procedure. A conventional 60/70 base bitumen and Styrene-Butadiene-Styrene (SBS) modified bitumen were used as binders for the mixes. The mixing temperatures adopted were 140°C and 180°C respectively for the base and SBS modified bitumen. The short-term ageing involved heating loose mix for four hours at 135°C and long-term ageing by exposure of compacted samples to 85°C for 120 hours in a forced-draft oven. As a result, the resilient modulus of short term and long term aged specimens was found to be higher than un-aged specimen. Conversely, the resilient modulus decreases when the test temperature increases

Keywords: Resilient Modulus, Temperature, Ageing test, SBS and 60/70 binder.

INTRODUCTION

Porous asphalt is a type of open-graded mix and has been use as a wearing course since the 1950s. Use of porous asphalt wearing courses can potentially inhibit ponding water on the road surface during wet weather (Che Wan et. al, 2009). Porous asphalt mix is designed to be overlaying an impervious course and has air voids exceeding 20%. Its application helps to enhance traffic safety especially during rainy days, reduce aquaplaning potential and improved good skid resistance properties at higher speed. In addition it also has good resistance to permanent deformation and is not subjected to rutting. Lately in the developed world, porous asphalt is applied to reduce traffic noise.

The resilient modulus test can be used to evaluate the mixture temperature susceptibility and to compare mix performance. Though it was once believed that stiffer pavements had greater resistance to permanent deformation, it has been concluded that resilient modulus at low temperatures is somewhat related to cracking, as stiffer mixes (higher MR) at low temperatures tend to crack earlier than more flexible mixtures (lower MR). (Michel, Bruce, 2002).

Ageing is a phenomenon that occurs in roads in which up to seven decades of research has carried out to better understand factors that contribute to short and long term ageing. The short-term component occurs during the construction phase, while the mix is hot. This is probably caused by volatilization although there may be some steric hardening and oxidation involved and volatilization of oxidation products. The long-term component occurs while the mixture is in service. A time period of about 10 years is a reasonable period of interest for this component. This long-term ageing is predominantly caused by oxidation, although there may be some steric hardening involved and volatilization of oxidation products. Actinic light may serve to accelerate the ageing at the surface of a pavement.

MATERIALS AND METHODS

Binder and Aggregate

The binder used in this study was a conventional penetration grade 60/70 and SBS modified binder supplied by Shell Malaysia. The conventional binder was a typical grade used on Malaysian roads. Granite aggregates supplied by Kuad Quarry Sdn. Bhd. were used throughout this investigation. To arrive at a final blend in mixture proportioning, the aggregates were washed, dried and sieved into their respective size range.

Gradation

Gradation test was done to isolate aggregates from the stockpile to the designated sizes that would be used in mix design. In this investigation, aggregate gradation adopted met the JKR specifications for porous asphalt as shown in Figure 1 was used.

[1] Lecturer, Polytechnic Tuanku Sultanah Bahiyah, 09000 Kulim, MALAYSIA
[2] Professor, Universiti Sains Malaysia, 14300 Nibong Tebal, MALAYSIA
[3] PhD Student, Universiti Sains Malaysia, 14300 Nibong Tebal, MALAYSIA

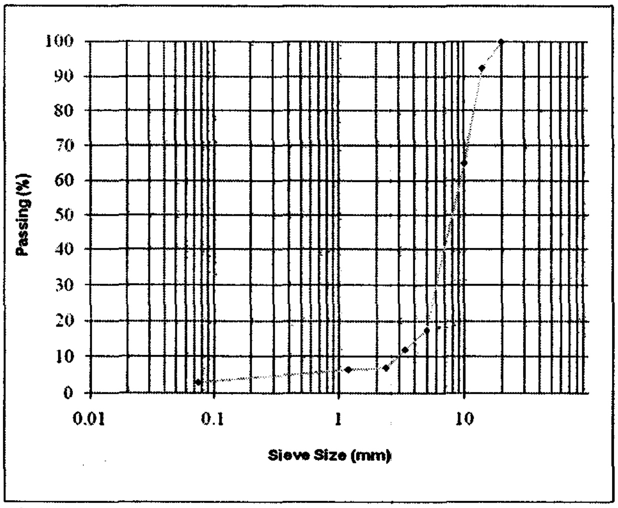

Fig. 1: Porous Asphalt Aggregate Gradations Used in This Study

Porous Mix Preparation

Ordinary Portland cement and hydrated lime were used as filler in this investigation. The conventional 60/70 penetration grade bitumen was used as binder for mix design. The aggregates were first mixed into batches according to the designated gradations and weight. These batches were then heated in an oven at the designated mixing temperature for at least 4 hours before the mixing process. For this study, all porous mixes were mixed at 180°C. The heated aggregate batches were then mixed with designated amount of bitumen. The mixes were compacted with 50 blows on each side with the standard Marshall hammer to avoid disintegration of aggregates. After compaction, the specimens were removed from the molds and allowed to cool down.

Resilient Modulus Test

A five pulse indirect tensile modulus test conforming to ASTM D 4123 test method (ASTM, 2005) was conducted using the Universal Testing Machine, MATTA. Each specimen was tested at 10°C and 25°C after a 4-hour-conditioning period. In the test, a pulsed diametral loading force was applied to a specimen and the resulting total recoverable diametral strain was then measured. Strains in the same axes were not measured. A Poisson's ratio 0.4 was adopted for the material. With a fixed level of applied peak force, the test sequence consisted of the application of 150 conditioning pulses followed by 5 pulses where data acquisition took place. The conditioning pulses ensured that the loading platens were seated onto the specimen for consistent results. For controlled temperature testing, the specimen's skin and core temperatures were estimated by transducers inserted in a dummy specimen and located near the specimen under test. The test procedure was repeated by orientating the specimen at 90 degree.

Aging of Bituminous Mixtures

The short-term (STA) conditioning for the mixture mechanical property testing procedure was applicable to laboratory-prepared, loose mix only. The method used in this test program was conducted according to AASHTO R30-02 (AASHTO, 2002). The method consisted of curing mix samples in a forced-draft oven at 135°C for four hours. After curing, the samples were brought to the required compaction temperature and compacted using a Marshall hammer.

The long term (LTA) mixture conditioning procedure can be applied to laboratory-prepared samples following short term aging, to plant-mixed HMA, or to compact roadway samples when needed to simulate long term aging effects. The procedure was carried out on compacted specimens after they have been short-term aged. The specimens were placed in a forced-draft oven, pre-heated to 85°C, and left for five days. After the aging period, the oven was turned off and left to cool to room temperature. The specimens were then removed from the oven and tested no less than 24 hours later.

RESULTS AND DISCUSSION

Resilient Modulus at Varying Binder Contents

The resilient modulus is simply the modulus of elasticity when the asphalt sample is loaded within its elastic range where the deformation is fully recoverable. It is defined as the ratio of the applied stress to the recoverable strain when a dynamic load is applied. The graphical illustration of the resilient modulus for 60P and SBS mixes versus binder content relationship of porous asphalt (PA) at 10°C and 25°C are presented in Figure 2 and 3. Both mixes show similar trend when their respective resilient modulus is related to the bitumen content.

The resilient modulus of PA increases until a maximum at 4.2% bitumen content, then decreases as the bitumen content increases. It is also observed that the resilient modulus of SBS mix is higher than those of 60P mixes, with the maximum resilient modulus of SBS mixes 37.60% (10°C) and 20.80% (25°C) higher than the maximum resilient modulus of 60P. The higher resilient modulus indicate less flexibility under loading. Based on the Figure, the resilient modulus for all mixes specimens reduces by approximately 58.65% to 67.40% when the test temperature increases from 10°C to 25°C.

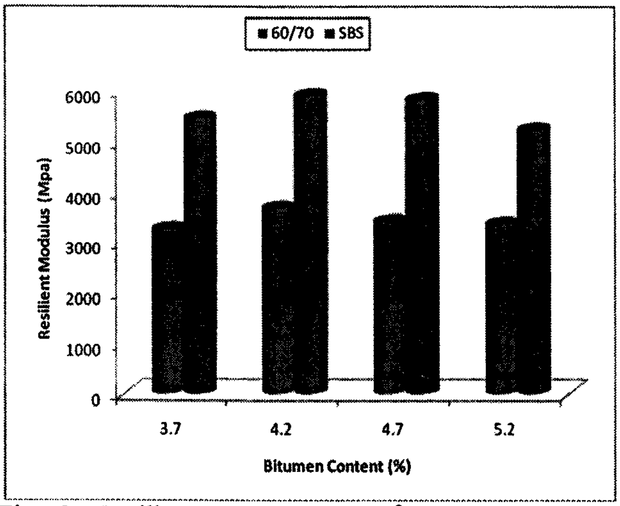

Fig. 2: Resilient Modulus at 10°C versus Bitumen Content (Un-aged)

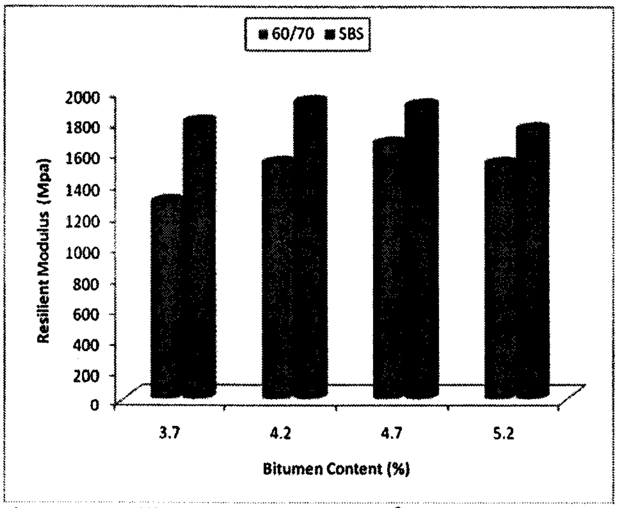

Fig. 3: Resilient Modulus at 25°C versus Bitumen Content (Un-aged)

Effects of Short-term Ageing on Resilient Modulus of Porous Mixtures

The curves representing the relationship between resilient modulus and bitumen content of un-aged and aged specimens are illustrated in Figure 4 and summarized in Table 1. The points plotted are the average of two readings. In general, the resilient modulus of the all mix types increases after every ageing. For instance, 60P mix, at 4.2% bitumen content and 10°C test temperature, the resilient modulus of PA increases by almost 33.70% after short-term ageing. At 25°C, SBS mix, the resilient modulus of specimens with 4.2% bitumen content increase by 7.70% when subjected to STA. Ageing process causes oxidation and increases the hardening rate of the bitumen, thus resulting in the increasing resilient modulus.

As concluded that, the resilient modolus of PA increases after every ageing session. The effecf of increased stiffness contributed towards longer service lifespan of pavements by increasing the rutting resistance. Based on the results shown in Table 1, the resilient modulus decreases drastically with the increase in temperature. When tested at 10°C, the resilient modulus of 60P mix equal 3242 Mpa, at 25°C test temperature, the corresponding value is 1276 Mpa which represent a 60.64% decrease.

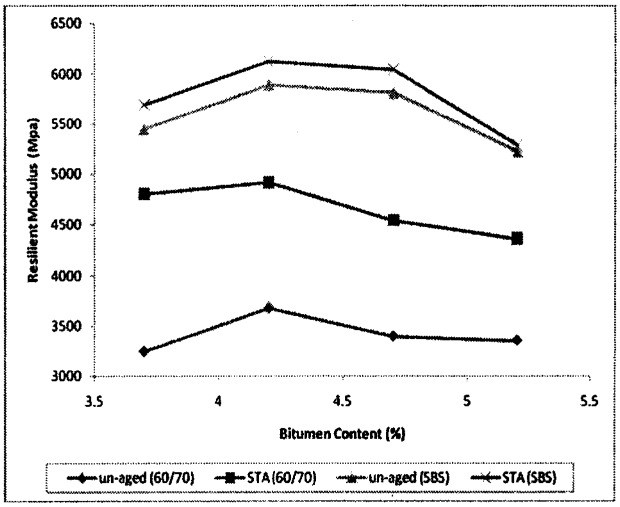

Fig. 4: Resilient Modulus at 25°C versus Bitumen Content (Un-aged)

Table 1: Resilient Modulus Test Results Subjected to STA

Temperatures	Bitumen Content (%)	Type of Binder			
		60/70		SBS	
		Un-aged	STA	Un-aged	STA
10°C	3.7	3242	4802	5442	5687
	4.2	3676	4915	5887	6125
	4.7	3398	4543	5808	6043
	5.2	3353	4359	5222	5282
25°C	3.7	1276	1553	1788	1969
	4.2	1520	1652	1919	2067
	4.7	1647	1696	1896	2062
	5.2	1516	1601	1742	1797

Effects of Long-term Ageing on Resilient Modulus of Porous Mixtures

The results of the resilient modulus test for long-term ageing at varying bitumen content and temperatures are tabulated in Table 2. Regardless of bitumen type or test temperatures, all LTA mixes exhibit increased resilient modulus values compared to un-aged mixes. The general trend showing the resilient modulus decreases with the increase in temperature and the resilient modulus of LTA specimens are higher than un-aged specimens; are similar to STA specimens tested under the two temperature regimes. Thus, the effect of using SBS modified binder is to cause an increase in resilient modulus. Given the same test temperature, the SBS mixtures generally exhibit a higher increase in resilient modulus when tested at a particular temperature.

Table 2: Resilient Modulus Test Results Subjected to LTA

Temperatures	Bitumen Content (%)	Type of Binder			
		60/70		SBS	
		Un-aged	LTA	Un-aged	LTA
10°C	3.7	3242	4839	5442	6028
	4.2	3676	5735	5887	6447
	4.7	3398	5306	5808	6284
	5.2	3353	4539	5222	5380
25°C	3.7	1276	1705	1788	2067
	4.2	1520	1833	1919	2317
	4.7	1647	1881	1896	2245
	5.2	1516	1594	1742	1996

CONCLUSIONS

1. The resilient modulus of porous asphalt increases until a maximum at 4.2% bitumen content, then decreases as the bitumen content increases
2. There is a general trend for the resilient modulus to decrease when the test temperature increases.
3. Polymer modified bitumen, which has been used for STA and LTA shows encouraging results, high resilient modulus values at higher temperatures as compared to the conventional mix.

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REFERENCES

ASTM D4123-82 (2005). Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures. Annual Book of ASTM Standards, West Conshohocken, Philadelphia, U.S.A.

AASHTO R30-02. (2002). Mixture Conditioning of Hot Mix Asphalt. Standard Specification for Transportation Materials and Methods of Sampling and Testing.R30-02 Part 1B. American Association of State Highway and Transportation Officials.

Che Wan C.N, Hamzah M.O, Ramadhansyah P.J (2009). Effects of Binder Type on Resistance to Abration Loss of Porous Asphalt Subjected to Ageing. Proceedings of National Postgraduate Conference 2009. Universiti Teknologi Petronas, March 2009.

Michael S., Bruce A. C., and Andrew D., (2002). Investigation of Recycled Martial Asphalt Pavement (RAP) Mixtures. Minnesota Department of Transportation, USA.

Rheological Properties of Recovered Binder from Aged Asphalt Pavements

Meor Othman Hamzah, Zulkurnain Shahadan and Joewono Prasetijo

School of Civil Engineering

Universiti Sains Malaysia

Corresponding author: cemeor@eng.usm.my

Abstract

In line with sustainable development, efforts have been geared towards recycling and reusing asphalt materials that has reached the end of its design life. The effort is compounded with declining quality aggregate sources and fluctuation in petroleum cost and availability in the past few years. Utilizing reclaimed asphalt pavement (RAP) or milling waste in asphalt mixes has more advantages than just recycling, which includes the preservation of existing road profile and the environment, conservation of asphalt binder and aggregate resources, conservation of energy and reduction in life-cycle cost. In Malaysia, mill and pave rehabilitation technique is most commonly used to remove old and distressed asphalt pavements and resurface with a new layer. This is a good opportunity to reclaim and reuse the milled asphalt pavement rather than mere dumping elsewhere because it still has valuable properties and substantial strength to be incorporated in new mixes. The objective of this paper is to conduct laboratory study on a reclaimed asphalt pavement materials and performance of recycled mix. This paper presents the characteristics of the reclaimed asphalt pavement materials and results from the experiments indicated that asphalt mixtures containing RAP performed similar if not better than conventional asphalt materials.

Keywords: Reclaimed Asphalt Pavement, Milling Waste, Rheological Properties, Indirect Tensile Strength

1. Introduction

Recycling of reclaimed asphalt pavement (RAP) has been practiced in the United States since 1915 and continue to increase appreciably in the 1970s with the Arab oil embargo causing inflation of construction cost due to limited oil supplies[1]. Today, the United States and several European countries have been using reclaimed asphalt pavement from 80% up to 100% in an effort to promote recycling in sustainable road construction[2]. In Malaysia, recycling was started in 1993 with the arrival of first recycling machine and also the first Asian country to use cold in-place recycling[3]. However, the resulting recycled pavement exhibited poor durability. As technology developed, road concessionaires and JKR are adopting mill and pave technique as rehabilitation works on most of all surface distress pavements. In this method, the old asphalt pavement and distressed pavement are milled and replaced by new HMA. Recycling of existing pavement material on this rehabilitation works will definitely preserved the environment by reducing the usage of new aggregate and binders for new asphaltic mix which in turn reduced costs of construction. Furthermore, the benefits of using recycled hot mix asphalt include reduced waste, preservation of the existing pavement geometrics and conservation of energy and reduction in life-cycle cost. In terms of performance, studies have shown that addition of RAP in new HMA have no significant effects on pavement performance compared to virgin mix at certain percentage level of RAP [4]. It has also been found that the inclusion of RAP in HMA increases the mixture stiffness which improved rutting performance of HMA and moisture resistance [5,6].

Every year, hundreds of kilometres of asphaltic pavement is milled up in this country and wasted even though it still has substantial engineering value and economically it can save cost of construction or rehabilitation pavement surface if it is recycled. However, it is important to ensure that the RAP materials are compatible with the virgin materials and that the final blend meets all the mix and binder requirements.

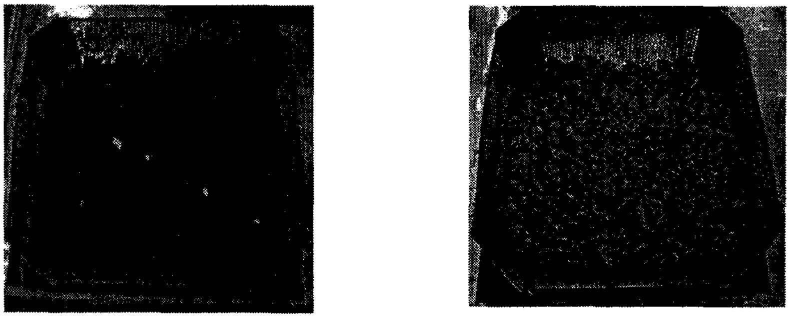
In this study, three RAP samples from different road authorities were sampled and the characteristics of each RAP material were investigated in terms of binder content, softening point, penetration, aggregate gradation and the rheological characteristics of the reclaimed RAP binder. The reclaimed RAP binders were blended with virgin binder in order to determine how RAP binder concentration affected the stiffness. Asphalt concrete mixtures containing RAP were evaluated for strength by using Indirect Tensile Test (ITS) and the results were compared to virgin materials mix.

2. Materials and test procedures

The study focus on RAP removed by cold-milling from Lebuh Raya Damansara Puchong (LDP), Projek Lebuh Raya Utara-Selatan (PLUS) and JKR (Federal Road) roads which were experiencing crack distressed pavements after 4 - 5 years in service. Approximately 500 kilograms of RAP materials were obtained from each site and kept in the barrels. The RAP samples consist of both ACW20 and ACW14 which were normally used by the Lembaga Lebuhraya Malaysia and Jabatan Kerja Raya respectively. Virgin binder grade 80/100 (PG64) was used as a base binder in this study.

2.1 RAP binder content by ignition oven method

The AASHTO TP53-95 (AASHTO, 2002) procedure was used to determine asphalt binder content and extraction of aggregates from RAP. Samples were carefully selected by quartering in a large tray in order to get a best representative of the samples and to avoid any segregation. Approximately 1.5 to 2.0 kg of RAP sample was placed in the special basket. In this test procedure, the oven chamber was preheated to 538°C then sample of RAP was put in the chamber and heated up to approximately 600°C where the asphalt cement binder was removed from the sample by ignition and burning. The ignition method produces a clean aggregate sample that can be used to determine a mixture's aggregate gradation. Figure 1 shows RAP sample material before and after the burning process.



(a) Before (b) After

Figure 1. RAP sample before and after ignition oven test

2.2 RAP aggregate gradation

Sieve analysis of fine and coarse aggregates of RAP aggregates was carried out by using AASHTO T27-88 (AASHTO, 2002). Accordingly, the individual particle size fractions of the aggregates was separated using a range of sieve sizes and the relative percentages of each size fraction determined.

2.3 RAP binder physical properties

Penetration and softening point tests were carried out using standard procedure based on AASHTO T49-97 and AASHTO T53-96 (AASHTO, 2002) respectively.

2.4 RAP binder rheological properties

RAP samples from the three sites were sent to IKRAM Sdn Bhd for binder extraction to obtain approximately 400 gram recovered RAP binder from each RAP source. The rheological properties and SUPERPAVE performance grade of the recovered RAP binders were determined by using Dynamic Shear Rheometer based on AASHTO TP5-93 (AASHTO, 2002) procedure.

2.5 RAP mixture performance

Marshall samples from control and three RAP sources containing 4 levels percentage of RAP (10, 20, 30 and 40%) were subjected to Indirect Tensile Strength Test (ASTM D-6931) to determine its tensile strength. The load was applied diametrically at a deformation rate 50.8 mm/min at 25°C.

3. Result and discussion

3.1 RAP binder content

Figure 2 shows the average asphalt content obtained via the ignition oven method. It can be seen that RAP from LDP samples exhibit the highest average asphalt content at 4.57 % while PLUS RAP has the lowest average asphalt content at 4.37%. RAP binder from JKR is within the optimum asphalt binder content ranging from 4 to 6% for asphalt wearing course.

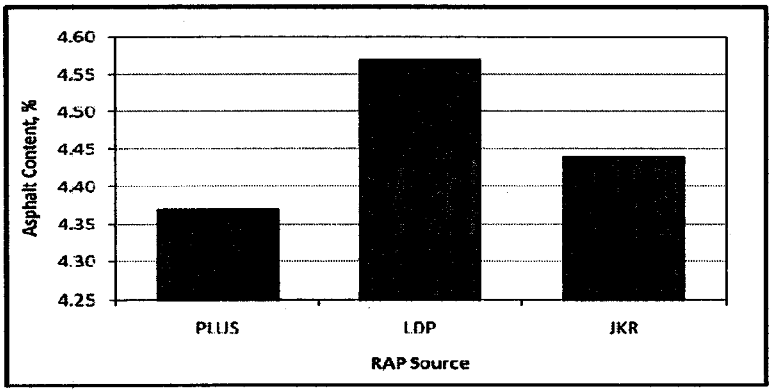


Figure 2. Average asphalt content of reclaimed asphalt pavement

3.2 RAP aggregate gradation

The by-product of the ignition test was then used to determine aggregate gradation by sieving the RAP aggregates through selected sieve sizes. Figures 3 and 4 present gradation curves of RAP aggregates plotted based on Lembaga Lebuh Raya and Jabatan Kerja Raya specification limits respectively. It can be seen that some portions of the aggregate gradations from the three RAP sources are either on the borderline or outside the accepted specification limits. This happened mainly due to disintegration and degrading of RAP aggregates during milling process which created more fines compared to virgin mixes. Large fraction of coarse RAP aggregates can be expected to decrease and subsequently contribute to increasing percentage of retained finer RAP aggregates. Based on 5 mm sieve analysis as summarized in Table 1, it is found that the average percent retained on 5 mm sieve for PLUS and LDP are about 36% and 27% respectively which is about 23% to 43% lower than percentage retained of the virgin aggregates. However, the percentage retained of JKR RAP aggregate at 5 mm sieve is reduced by 34% to approximately 25% retained after the milling process.

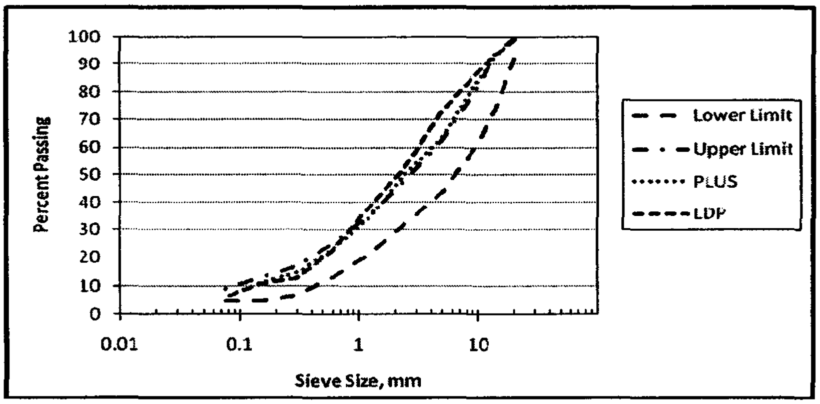


Figure 3. Aggregate gradations of PLUS and LDP RAP samples (LLM-ACWC20)

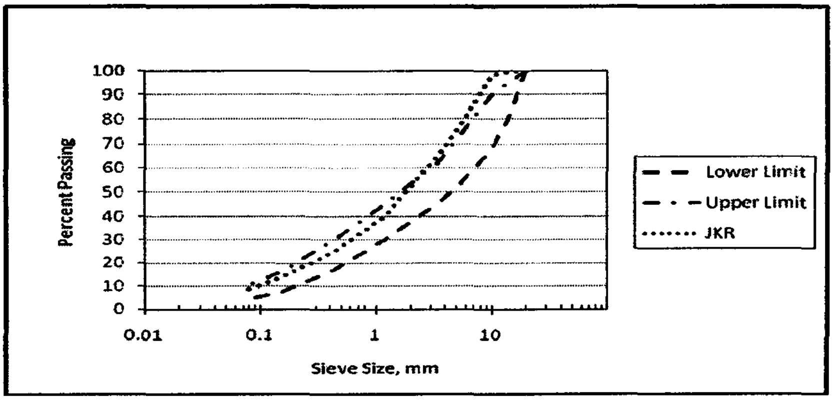


Figure 4. Aggregate gradations of JKR RAP samples (JKR-ACWC14)

Table 1. Percent retained and passing 5 mm sieve of all RAP aggregates

RAP Source	% retain 5 mm sieve	% passing 5 mm sieve
PLUS	36	64
LDP	27	73
JKR	25	65

3.3 RAP binder physical properties

Recovered RAP binders and unaged original 80/100 binder were tested for penetration and softening point. Table 2 shows the result of penetration test of recovered RAP binders which is about 85% lower compared to virgin binder. The low penetration values are expected since aged binder hardens over time due to oxidation while pavements are in service. All three RAP binders exhibit almost similar penetration depth because the RAPs were milled from pavements that had been in service for about 4 to 5 years. The softening point of recovered JKR RAP binder is the highest which complement with its lowest penetration value among the recovered RAP binders. Recovered RAP binders from PLUS and LDP exhibit similar softening point.

Table 2 Penetration and softening point of original and RAP binders

Test	Virgin Binder	RAP Binder		
		PLUS	LDP	JKR
Penetration, dmm	88	14	13	11
Softening Point, °C	46	67.5	67.5	72

3.4 RAP binder rheological properties

The recovered binders were further tested for rheological property for unblended and blended of unaged recovered RAP binders. Table 3 shows the average value of G*/sin δ for unblended RAP binders at high temperature test. It can be seen that G*/sin δ value is decreased as the temperature is increased. From Figure 5, the binder extracted from JKR RAP has the highest stiffness at all temperatures. This is in parallel with this RAP binder having the lowest penetration and highest softening values. Both recovered binder from PLUS and LDP RAP exhibit similar stiffness. From this result, the high temperature performance grade can be estimated at a PG88 for all recovered RAP binders.

Table 3. DSR test results of recovered RAP binders

Temperature °C	Average G*/sin δ, kPa		
	PLUS	LDP	JKR
52	254.0	171.1	253.6
58	98.0	78.0	109.8
64	38.0	36.1	47.8
70	16.1	16.4	20.7
76	6.88	7.39	8.99
82	3.13	3.43	4.13
88	1.56	1.68	2.00

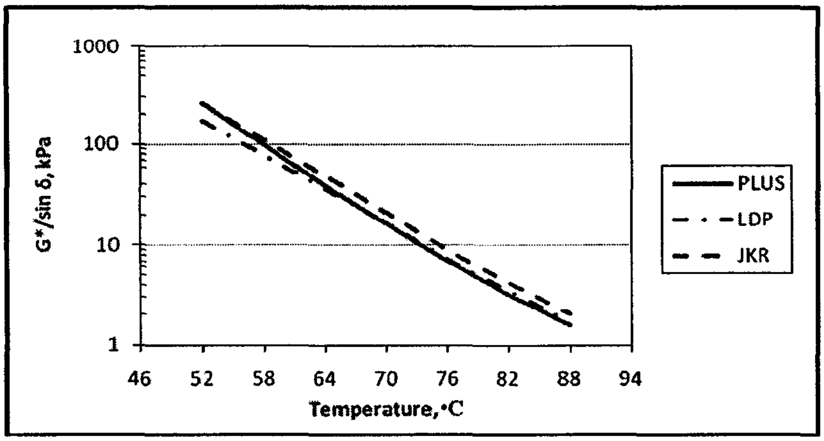


Figure 5. Stiffness of recovered RAP binders

Table 4 summarizes the results of DSR testing on the unaged blended binders. These results show that doubling the RAP binder concentration from 15% to 30% resulted in increasing the stiffness from 46 to 91 percent. PLUS RAP blend has the highest stiffness effect after doubling the concentration. The temperature – stiffness curves in Figure 5 shows LDP and JKR blends are identical while in Figure 6, LDP and PLUS blends curves almost overlap with each other which suggests that both RAP blends exhibit similar stiffness property. The stiffness – temperature relationships of the blended binders can be used to grade the unaged binder high temperature performance of each blend by determining the maximum temperature that satisfy the G*/sin δ ≥ 1.0 kPa requirement. For all RAP binder types, the addition of 15% of RAP binder increases the stiffness of the binder blend one grade higher to PG70 than that of the base virgin binder (PG64). As the RAP binder concentration doubles to 30%, the stiffness of the blend increase by two grades to become PG76 compared to base virgin binder.

Table 4. Results of DSR testing of unaged blended binders

Temp., °C	G*/sin δ, kPa								
	PLUS			LDP			JKR		
	% RAP Binder		% increased	% RAP Binder		% increased	% RAP Binder		% increased
	15	30		15	30		15	30	
52	14.3	22.1	54	12.1	23.1	91	12.0	19.0	59
58	5.46	8.47	55	4.55	8.69	91	4.58	7.44	63
64	2.28	3.53	55	1.96	3.81	95	1.96	3.12	59
70	1.09	1.59	46	0.93	1.71	83	0.94	1.43	52
76	0.52	0.78	50	0.45	0.84	88	0.45	0.67	50
82	0.26	0.39	51	0.22	0.41	83	0.23	0.34	49

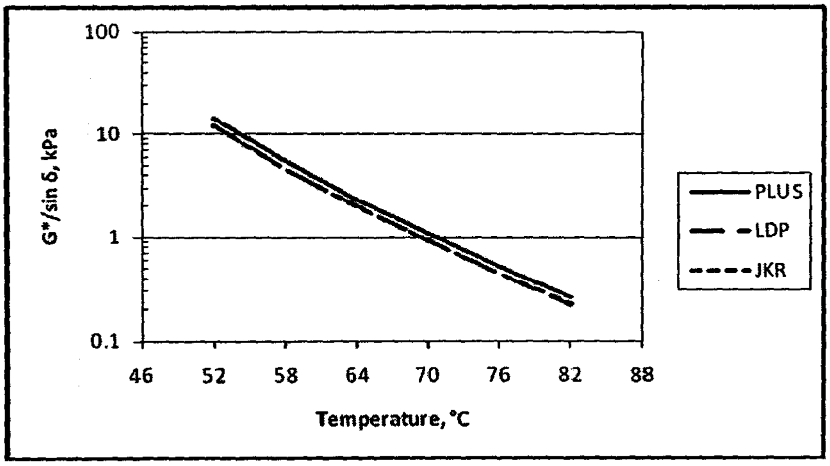


Figure 6. DSR test results for blended unaged binder with 15% recovered RAP binder

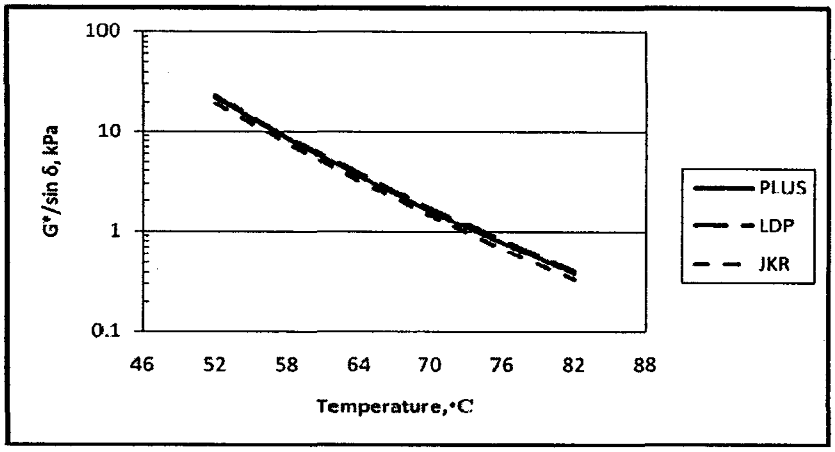
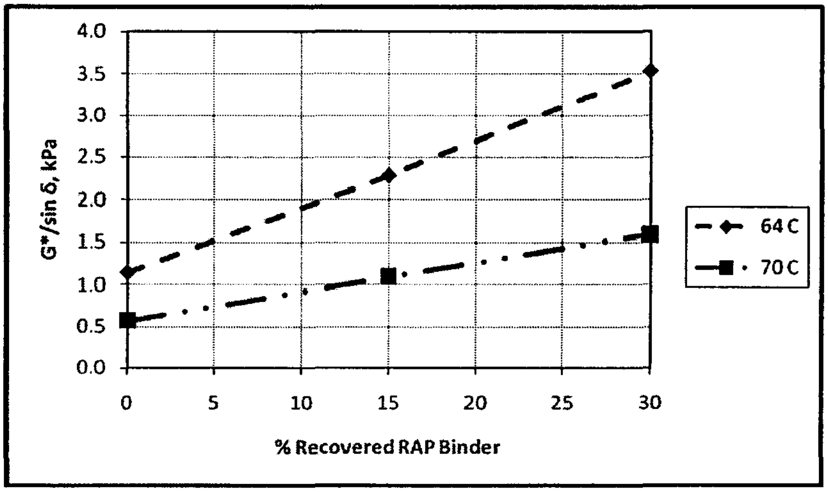
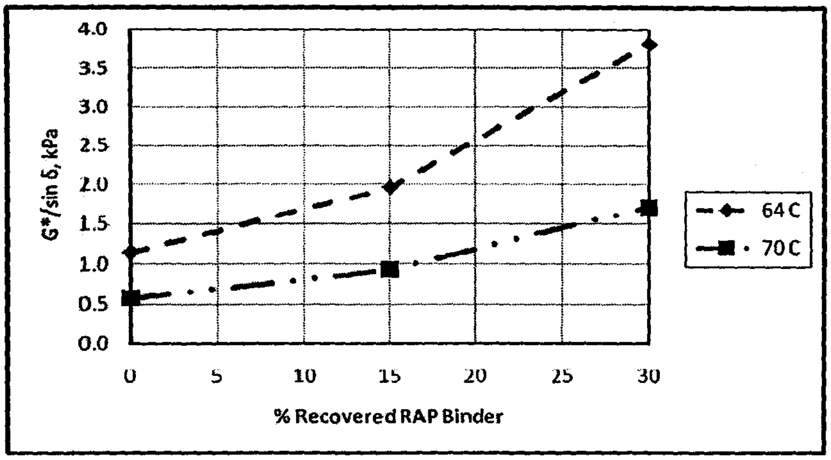


Figure 7. DSR test results for blended unaged binder incorporating 30% recovered RAP binder

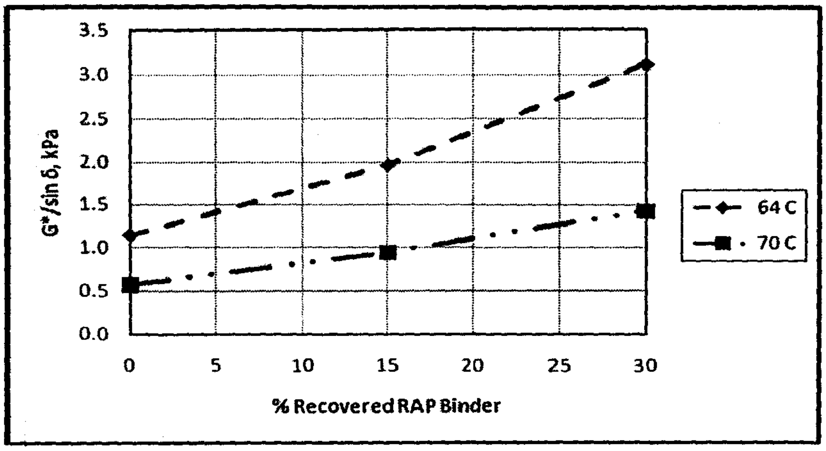
Figure 8 shows the relationship between G*/sin δ versus percent recovered RAP binder for unaged RAP blend. Clearly, G*/sin δ is higher at lower temperature and increases with the addition of RAP binder. The RAP binder produces stiffer blends. The rate of increase is high at lower temperatures and at higher percent of RAP as shown in Table 5. There is a noticeable increase in G*/sin δ at 64°C and 70°C, immediately after RAP binders were added to the virgin binder. At 64 °C, the PLUS RAP and LDP RAP blends exhibit the highest rate of increase in stiffness at 15% and 30% of RAP binder respectively.



(a) PLUS RAP binder



(b) LDP RAP binder



(c) JKR RAP binder

Figure 8. G*/sin δ for unaged RAP blends

Table 5. Rate of increase in G*/sin δ for unaged blends

RAP Source	% RAP	Rate of Increase in G*/sin δ (kPa/%RAP)	
		64 °C	70 °C
PLUS	15	0.076	0.035
	30	0.080	0.034
LDP	15	0.055	0.036
	30	0.089	0.038
JKR	15	0.055	0.025
	30	0.066	0.029

3.5 RAP mixture performance

The potential of thermal cracking of RAP mixes was determined by evaluating the mix fracture strength characteristic. From Figure 8, it can be seen that there is a trend of increasing in tensile strength as higher percentages of RAP are incorporated in the mixes. This result is expected as higher percentages of RAP significantly increase mix stiffness. The tensile strength for mixes containing 10% RAP is comparable with control sample for all RAP sources. Mixes incorporating 40% RAP exhibit the highest strength for all RAP sources where the tensile strength increase range from 27% to 30% compared to control mixes.

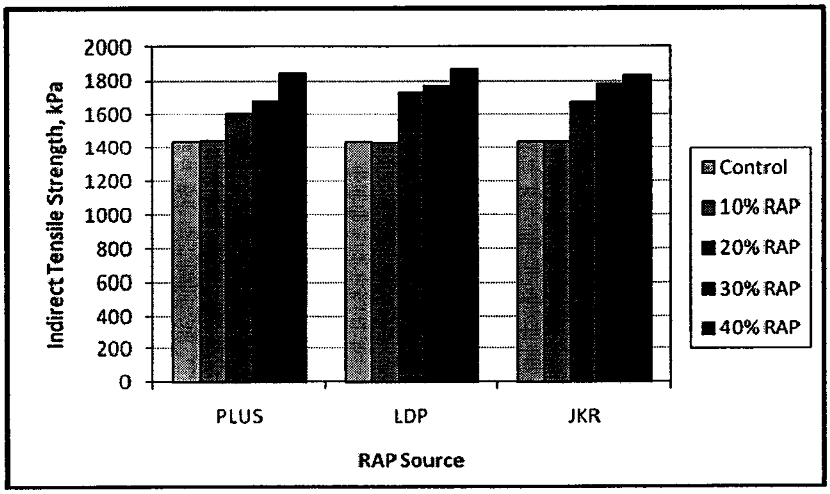


Figure 8. Indirect tensile strength of RAP mixtures

A *t*-Test statistical analysis on each RAP source do not indicate significant differences between the control sample and mixes containing RAP up to 30% as shown in Table 6. In other words, adding RAP up to 30% do not affect the strength of mixes. However, mixes with 40% RAP demonstrates significant difference in strength for all RAP sources.

Table 6 Paired t-Test comparisons of strengths of mixtures with different RAP contents

RAP Source	% RAP	*p*-value	Conclusion
PLUS	10	0.833129	NSD
	20	0.165541	NSD
	30	0.171739	NSD
	40	0.043409	SD
LDP	10	0.923969	NSD
	20	0.245950	NSD
	30	0.065820	NSD
	40	0.041602	SD
JKR	10	0.988426	NSD
	20	0.211271	NSD
	30	0.149638	NSD
	40	0.033305	SD

NSD= Non Significant Difference SD= Significant Difference

4. Conclusion

The original coarse aggregate fraction of RAP aggregates has become finer than virgin aggregate primarily due to disintegration during the milling process and also previous handling and placement of the materials. The virgin-RAP binder blend shows that the stiffness of the binder blend increases with RAP content. The addition of RAP samples in virgin mixes produces stiffer and higher strength mixes. The effect of RAP in mixes is more pronounced when more than 20 % RAP was added. All three RAP sources (PLUS, LDP and JKR) exhibit almost similar physical and rheological properties. The results also showed that recycled mixes when added to virgin mixes exhibit equivalent if not higher indirect tensile strength compared to virgin mixes but depends on the percentage addition of RAP.

References

[1] Kandhal, P.S., and R. Mallick,"Pavement Recycling Guidelines for State and Local Governments", FHWA, U.S. Department of Transportation, 1997

[2] K, Holtz, and T.T. Eighmy, "Scanning European Advances in the Use of Recycled Materials in Highway Construction", Public Roads, Vol. 64, No. 1, http://www.tfhrc.gov/pubrds/julaug00/recycscan.htm. Accessed January 17, 2009.

[3] A.J.N. Lewis,"Developments in Road Pavement Recycling in the Far East", Proceedings of the 8th Conference on Asphalt Pavements for Southern Africa, 2004

[4] K. Aravind, and A. Das, "Pavement Design With Central Plant Hot Mixed Recycled Asphalt Mixes", Construction and Building Materials, 2007, pp 928 – 936

[5] F. Xiao, S. Amirkhanian, and H. Juang, "Rutting Resistance of Rubberised Asphalt Concrete Pavement Containing Reclaimed Asphalt Pavement Mixture", Journal of Materials in Civil Engineering, Vol. 1 No. 6, 2007, pp 475 – 483

[6] I. Widyatmoko, " Mechanistic-Empirical Mixture Design for Hot Mix Asphalt Pavement Recycling", Construction and Building Materials, 2008, pp 77 -87

SUSTAINABLE ASPHALT – THE WAY FORWARD FOR THE MALAYSIAN ASPHALT INDUSTRY

M. O. Hamzah, Z. Shahadan, M. R. M. Hasan, A. Jamshidi
School of Civil Engineering, Universiti Sains Malaysia Engineering Campus,
14300 NIBONG TEBAL, PULAU PINANG,
MALAYSIA

Abstract

Bitumen, from which asphalt mix is produced, is a derivative of crude oil. The need for sustainable asphalt is imperative in view of the escalating price of bitumen, the ability of the oil industry to crack bitumen into higher value added products and the quest for more concrete roads as an alternative to asphalt pavements. Consequently, research and practices on asphalt should focus on sustainability to ensure the well-being and survival of the industry. This paper presents some asphalt research being conducted at the Universiti Sains Malaysia (USM) to address some sustainable related issues. A lot of energy input is required to produce asphalt in the mixing plant. The use of warm mix asphalt with Sasobit® enabled production temperature reduction up to 15°C without compromising on mix properties. At the end of its design life, pavement surfacings are either overlaid or milled for recycling. Use of reclaimed asphalt pavement can save utilisation of virgin binder. Natural asphalt such as asbuton is significantly cheaper than conventional bitumen. Preliminary research results on asphalt mixes incorporating 10% asbuton indicated that only 2.2% of conventional bitumen was needed to produce mix properties that conformed to the JKR specifications. Waste materials such as steel slag can serve as aggregate substitute in asphalt mixes. Being an engineered material, both dense and porous mixes incorporating steel slag were found to exhibit better properties compared to conventional mixes incorporating granite aggregates. Sustainable asphalt also includes the development of quiet pavements. The best candidate material for silent road is the double layer porous asphalt, able to reduce traffic noise at source by 6 dB(A). Extensive characterization of porous asphalt mixes has been conducted at the Universiti Sains Malaysia. Despite being blessed with enormous aggregate and oil reserves, emphasis must be made to ensure sustainability of the asphalt industry. Sustainability is a key issue in the developed world. The European asphalt industry has carried out extensive research on sustainable development focussing on the cost reduction and conservation of the environment and many have been implemented in the field.

Keywords: Sustainable Asphalt, Warm Mix Asphalt, Asphalt Recycling, Waste Material, Natural Asphalt, Porous Asphalt

1.0 INTRODUCTION

In 1987, the United Nations passed a resolution on sustainable development based on a report of the World Commission on Environment and Development which was concerned about the rapid deterioration of the human environment and natural resources and its adverse effects on the economic and social developments (United Nations, 1987). In the report, Brundtland et. al. (1987) defined sustainable development as *a development that meets the needs of the* *present without compromising the ability of the future generations to meet their own needs.* The resolution and concept were well understood and accepted by the world community. Since then, every effort and strategy have been made by government agencies, non-government organizations, private sectors and individuals to support, promote, achieve and maintain sustainable development. In harmony with this and coupled with the Universiti Sains Malaysia (USM) vision *on Transforming Higher Education for a Sustainable Tomorrow*, the School of Civil Engineering USM has formed the Sustainable Asphalt Research Group (SARG). The group has geared its research efforts on sustainable asphalt focusing on warm mix asphalt, asphalt recycling, use of waste materials, utilisation of natural asphalt and porous asphalt. The outcome of these research undertakings will hopefully provides impetus for sustainable road construction practice which is important for the well-being of the nation's asphalt industries.

Road construction demands consumption of a significant amount of non-renewable virgin materials. Fast depleting, declining quality of aggregate sources, escalating price of bitumen and lobby by the concrete industries for more concrete roads; have motivated the asphalt industries to seek other alternatives to reduce production cost and use of virgin materials. As shown in Table 1, the price of bitumen in Malaysia in 2008 was doubled to what it was in the year 2003 due to world economic uncertainties and controlled production of crude oil by some oil producing countries. According to the simple error-correction economic adjustment model propounded by Attanasi (2008), bitumen prices are found to be significantly more volatile than light crude prices. This bitumen price hike has somewhat adversely affected the economics of road construction and rehabilitation developments. The asphalt industry will not be sustainable if business is to go on as usual.

Table 1: Average Bitumen and Crude Oil Prices in Malaysia

Year	2003	2004	2005	2006	2007	2008
Bitumen (RM/tonne)[1]	816.67	730.83	742.50	1,176.67	1,247.50	1,616.65
Crude Oil (US$/barrel)[2]	30.09	40.93	57.92	70.02	77.67	105.11

(Source: [1]Malaysia Country Report, 2008; 2009; [2]World Oil Prices, 2010)

Currently, awareness and stringent environmental regulations are actively being implemented to prevent the worsening effects of climate change. In line with cost effective and environmental friendly road construction, it is necessary to decrease the total cost of pavement construction by using new energy conservation technologies. One of the technologies that is able to significantly reduce construction temperatures, hence cost of asphalt mixes as compared to traditional hot mix asphalt (HMA), is warm mix asphalt (WMA). One type of WMA additive is a synthetic wax named Sasobit®. Use of Sasobit® enables asphalt producers to attain the desired viscosity at lower temperatures. This implies less energy input, reduction of visible and invisible emissions from burning fuels and fumes, and the provision of a safe environment for crews working at asphalt mixing plant and at pavement construction sites (Hurley and Prowell, 2006). Laboratory research has shown that Sasobit® increases the aging index, Superpave™ rutting factor and Zero Shear Viscosity (Wasiuddin et. al., 2008; Biro et. al., 2009) of bitumen. Other experimental investigations revealed that addition of Sasobit® has reduced rutting and increased dynamic modulus of asphalt mixes (Mohammad et. al., 2008).

In the year 1992, 80% (73 million tons) of the total asphalt surface material removed during widening and resurfacing project in the USA was recycled and saving more than $300 million annually (USDOT,1992). In Europe, the Netherlands and Denmark have used up to 100% reclaimed asphalt pavement (Holtz and Eighmy, 2000). Utilizing reclaimed asphalt pavement (RAP) as alternative material in asphalt mixes has more advantages than just recycling and economic benefits associated with it, but further includes the preservation of existing road profile and the environment, conservation of asphalt binder and aggregate resources, conservation of energy and reduction in life-cycle cost (USDOT, 1997). RAP also decreases the amount of waste produced and helps to resolve the disposal problems of highway construction material waste. Many studies on RAP materials incorporated in asphalt mixes with up to 50% RAP content have shown that the performance of recycled asphalt pavement on ravelling, rutting, moisture susceptibility, fatigue resistance and three point bending tests performing as well as a pavement that was made up from totally virgin materials (Su et. al., 2009; Widyatmoko, 2008). From the environmental viewpoint, Huang et al., (2009) investigated the environmental assessment of the energy input and carbon footprint when natural aggregates were partially replaced with waste glass, incinerator bottom ash and RAP and compared the results to the pavement of the same size and function but made up of entirely virgin aggregates. Life cycle assessment model was used and the results showed that incorporating these recycled materials have reduced the amount of energy required and carbon dioxide emission compared to virgin material especially when RAP was incorporated in the production of the recycled asphalt pavement.

A naturally occurring asphalt known as asbuton is a rock asphalt found deposited in abundant quantity in Buton Island, Indonesia. Asbuton has been used as alternative binder in full scale trial in producing hot-mix asbuton (160°C) and warm-mix asbuton (120°C) in 2005 and 2008 respectively, for constructing road base and wearing courses in Indonesia. Based on the Research and Development Centre for Road and Bridge (RDCRB) findings on core samples extracted from warm-mix asbuton pavement, the aggregate gradation, volumetric and Marshall properties have complied to the RDCRB specifications and the pavement is performing well (Iriansyah, 2009). Encouraging results were obtained from studies conducted by Subagio et al, (2003; 2005) who used asbuton in asphalt mixes as fine aggregate and filler. The optimum binder content for asphalt mixes with asbuton as filler was lower than the HMA control mixes using fly ash. Inclusion of asbuton in the asphalt mixes has given the stiffening effect to the samples. This is evident from higher values obtained from the Marshall test, Marshall immersion test, resilient modulus test and better resistance to permanent deformation. Furthermore, use of asbuton as filler resulted in an increased initial flexural stiffness, decreasing the rate of crack propagation and higher number of cycles for crack propagation in the fatigue test.

Application of waste materials is one of the major concerns for sustainable asphalt. Steel slag, a waste product from the steel making industry, is one of the industrial wastes reported to exhibit great potential to replace naturally occurring aggregates in asphalt mixes. According to Holtz and Eighmy (2000), recycled materials in some countries were engineered into construction materials due to their close loop recycling policy where recycled and waste materials were continually reused without wastage. Steel slag was reused 100% as alternative materials in road construction in Sweden and Denmark, while 92% in Germany. In Japan, 99% of steel slag was employed by national agencies such as the Ministry of Land, Infrastructure and Transport and by local governments and other users, and it had gained both high acclaim and certification (NSA, 2006). Liz Hunt and Boyle (2000) and TFHRC (2000) mentioned that the addition of steel slag may increase the stability of the mix, improve the live expectancy and have good wear resistance, skid resistance and heat retention. Steel slag aggregate also exhibited a hydrophobic nature which enhances its superior adhesion with bitumen compared to the siliceous granite aggregates (See and Hamzah, 2002). Another waste material which is widely incorporated in asphalt mixes is crumb rubber. Research on crumb rubber mix has been ongoing for decades. Crumb rubber is a recycled rubber obtained by mechanical shearing or grinding of scrap tires into small particles. The characteristics of crumb rubber depend on the rubber type, asphalt composition, size of rubber crumbs as well as time and temperature of reaction. These factors have considerable effects on pavement performance (Raad et al., 2001; Kim et al., 2001). Palit et al., (2004) conducted a study to determine the effects of 0.6 mm crumb rubber particle size on the performance of crumb rubber modified asphalt mixes. Crumb rubber modified mixes exhibited improved fatigue and permanent deformation characteristics.

Road construction has also adversely affects the environment in terms of noise pollution, urban temperature, and storm water run-off. To mitigate these effects, porous asphalt (PA) was developed as an alternative material for road surfacings. This type of wearing course material has received acceptance globally to reduce traffic noise, promote safe driving conditions, clear visibility, improve skid resistance, and reduce hydroplaning on the road surface. PA has also been used for other applications such as parking lot area and noise attenuation (Alvarez et. al., 2006). Additionally, use of PA results in cleaner storm water run-off (Roseen et. al., 2007). It was reported that 99% of suspended solids, 38% phosphorus, 96% zinc and 99% petroleum hydrocarbons from diesels were found filtrated through PA from storm water. However, its service life is somewhat limited due to poor durability compared to conventional mix. According to Huber (2000), many countries adopted modified binder and polymer additives to improve the performance of PA with less susceptible to binder drainage during production and after construction. However, PA incorporating conventional binder 70/100 has been successfully used on Dutch highways with service life extending up to 16 years on the fast lanes and currently 90% of Dutch highways have been paved with the material, primarily for the noise reduction.

2.0 OBJECTIVE

The main objective of this paper is to highlight sustainable asphalt research conducted at the School of Civil Engineering, Universiti Sains Malaysia. It concludes with some sustainable asphalt initiatives and implementation in some European countries, particularly in the Netherlands.

3.0 SUSTAINABLE ASPHALT RESEARCH INITIATIVES

3.1 Warm Mix Asphalt

Laboratory investigation was conducted using asphalt binder 80/100 penetration grade blended with 0%, 1%, 2%, 3% and 4% Sasobit®. Unaged and long-term aged Sasobit® modified asphalt binders were tested for viscosity using a Brookfield rotational viscometer to determine the desired mixing and compaction temperatures. The asphalt mixes were subjected to indirect tensile strength and resilient modulus tests. Table 2 shows the required mixing and compaction temperatures of all binders tested based on the Asphalt Institute criteria (Asphalt Institute, 2007).

Table 2: Reduction in Mixing and Compaction Temperatures of Asphalt Mixes Incorporating Sasobit®

Aging State	Sasobit® content (%)	0	1	2	3	4
Unaged asphalt	Mixing temperature	160	155	152	150	145
	Compaction	150	145	142	140	137
Long term aged (RTFO+PAV)	Mixing temperature	175	168	165	160	154
	Compaction	158	153	151	150	149

Incorporation of Sasobit® in the asphalt binder has decreased the required mixing and compaction temperatures of unaged and aged asphalt binders. For unaged binder without Sasobit® (pure asphalt binder), the mixing and compaction temperatures are 160°C and 150°C respectively. For Sasobit® modified binder; the required mixing temperature can go down to as low as 145°C. The mixing temperature of aged asphalt binder sample modified by 3% Sasobit® is equal to the mixing temperature of unaged pure asphalt binder, while 4% Sasobit® makes the aged asphalt binder softer than the unaged pure asphalt binder. The economic advantage of Sasobit® on pavement construction projects is evaluated in terms of fuel saving to heat up aggregate to the mixing temperature. The required energy to heat up the asphalt binder is ignored because the quantum is negligible when compared to energy consumption required to heat up the aggregates. Calculations were made based on the assumptions that the price of heating oil for industrial usage is RM32.56 per mmBTU (IEA, 2010). The required energy and cost savings for different Sasobit® contents are shown in Table 3 based on heating 100,000 tonnes of granite aggregate to the mixing temperature. The Table also postulates the implications of temperature reduction to 135°C.

Table 3: Cost and Energy Savings for Mixes Incorporating Sasobit®

Mixing Temp (°C)	Sasobit® (%)	ITS (MPa)	ITS Percentage Difference	Mr (MPa)	Mr Percentage Difference	Required Energy (mmBTU)	Price (RM)	Energy Savings Cost (%)
160	0	1.25		6135		11,381.4	370,578.00	
150	1.5	1.32	(+) 5.6	6623	(+) 8.0			
	3	1.46	(+)16.8	6818	(+)11.1	10,451	340,284.00	8.2
145	1.5	1.17	(-) 6.4	5817	(-) 5.2			
	3	1.29	(+) 3.2	6477	(+) 5.5	9,983	325,046.00	12.3
135	1.5	1.12	(-)10.4	4813	(-)21.5			
	3	1.15	(-) 8.0	5361	(-)12.6	9,093	296,063.00	20
(+) = Increment; (-) = Reduction								

It can be seen that by decreasing the mixing temperature to 135°C, the energy cost required to heat up the aggregate decreases by 20%. However, the resilient modulus (Mr) and indirect tensile strength (ITS) also decrease. Samples with 3% Sasobit® and mixed at 145°C exhibit higher resilient modulus and ITS when compared with the control samples, while the energy savings amount to 12.3%. However, slower aging of the Sasobit® modified binder due to anti-aging properties of Sasobit® causes a reduction in the indirect tensile strength and resilient modulus of samples prepared at low mixing and compaction temperatures.

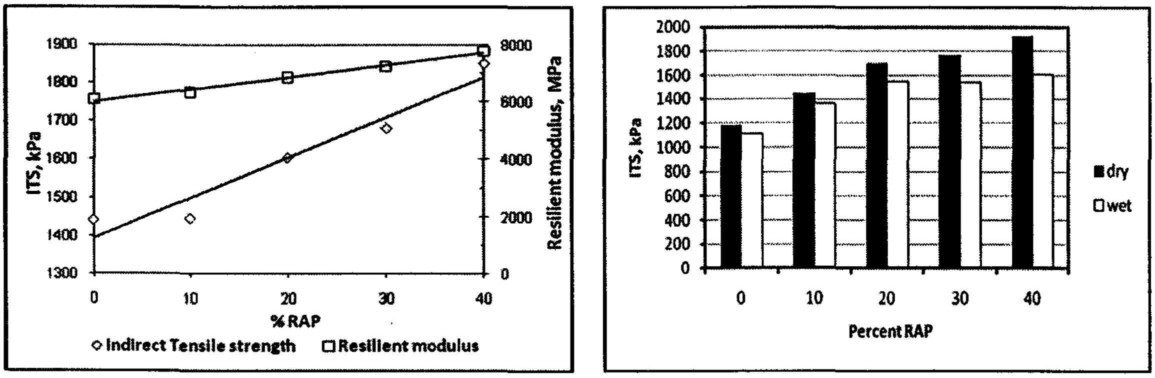
3.2 Recycled Asphalt Pavement

RAP milled from Projek Lebuh Raya Utara-Selatan (PLUS) highway was studied at the USM. A mix design of RAP mixes with four different RAP contents (10%, 20%, 30% and 40%) was carried out to make the gradation for each RAP mix as close as possible to the control mix. The total asphalt binder content determined in the design of the virgin material mixes was used in the mixes containing RAP. The amount of virgin asphalt added to the RAP mixes was reduced to account for the asphalt binder contributed by the RAP material.

Gradation of the RAP samples was based on JKR mix type AC14 (JKR, 2008). The virgin aggregate and RAP were heated separately at 160°C and 135°C, respectively. The RAP was heated to soften it and to ensure intimate blending between the virgin and RAP materials during mixing. However, the RAP was heated for a shorter period to avoid further hardening of the RAP binder. Both virgin and RAP materials were mixed at 150°C. The mixing and compaction temperatures were adopted based on the viscosity test results on virgin and recovered RAP binders blend. The RAP samples were subjected to indirect tensile strength (ITS), resilient modulus and moisture susceptibility tests. From Figure 1(a), the ITS increases with RAP content. This result is expected as higher percentages of RAP increase the mix stiffness. The ITS for mixes containing 10% RAP is comparable with control sample. Mixes incorporating 40% RAP exhibit the highest ITS, increasing to 22% compared to samples without RAP. Figure 1(a) also shows that the resilient modulus test result exhibit similar trend as the ITS test results, increasing by 22% with addition of 40% RAP in the mix.

Specimens incorporating various percentages of RAP from PLUS highway were fabricated and compacted to 7±1% air voids to evaluate its moisture sensitivity using the freeze/thaw cycle procedure. Figure 1(b) shows that the addition of RAP in the mix increases the ITS of both dry and wet specimens compared to control specimen. The percentage increase in ITS is up to 39% and 31% for dry and wet specimens respectively. However, the ITS of dry specimens is always higher than the ITS of wet specimens. The difference in ITS values between dry and wet specimens increases as the percentage of RAP in the mix increases, the highest being equal to 16% at 40% RAP content. Generally, the Tensile Strength Ratios (TSR) of all mixes are greater than 0.80 even though the ratio slightly decreases as the RAP content increases. The TSR obtained indicates that the recycled asphalt specimens is resistant to moisture induced damage. Approximate saving of 19% in virgin binder can be made if 40% dry RAP is incorporated in the recycled asphalt mix. In general, the recycled asphalt mixes perform better than the control mix. Therefore, reclaimed asphalt pavement can be a potential material to partially replace virgin materials in producing asphaltic concrete.

An extensive study on binder properties of recovered and blended binders is currently being carried out at the USM to further investigate their rheological and chemical properties via the dynamic shear rheometer (DSR), fourier transform infrared, differential scanning calorimetry and X-ray diffraction tests. The DSR characterizes the visco-elastic behaviour of binders, while the chemical tests detect the chemical changes in asphalt structure after oxidation due to the formation of new chemical elements. Chemically, aging leads to a decrease in the aromatic content and a subsequent increase in resin content together with higher asphaltenes content which hastens binder hardening.



(a) ITS and Resilient Modulus Test Results (b) Moisture Sensitivity Test Results

Figure 1: Performance of Mixes Incorporating RAP

3.3 Natural Asphalt (Asbuton)

A preliminary study on asbuton as alternative material for binder was carried out by mixing 10% asbuton in asphalt mix with four different 80/100 asphalt binder contents (1%, 2%, 3% and 4%) and the samples were tested for Marshall stability and conformity to the JKR specifications. The results showed that at 4.6% optimum binder content of control sample only requires 2.2% of virgin conventional 80/100 bitumen when blended with 10% asbuton to achieve similar stability, flow and stiffness as the control samples while all mix properties conform to JKR specifications. This translates into a percentage saving of binder content exceeding 50% when asbuton is incorporated in the asphalt mix.

3.4 Industrial Waste Materials

Due to favourable impact on the economic and asphalt performance, studies were conducted at the USM on steel slag and crumb rubber as alternative and additive materials respectively in asphalt mix. Yi (2008) ascertained the superiority of 20 mm and 10 mm steel slag size with respect to flakiness index (4%), abrasion loss (9.80%) and polished stone value (56.6), while no evidence of stripping was observed with steel slag aggregate. Laboratory studies were conducted on mixes incorporating 100% granite aggregate, 100% steel slag and another with an equal proportion of steel slag and granite aggregates. The results indicated the superiority of mixes incorporating steel slag while their mix properties conformed to the JKR specifications for AC 14 (JKR, 2008). From moisture sensitivity tests, mixes incorporating steel slag exhibit TSR approximately 1.0 confirming its excellent resistance to moisture induced damage. The subject of steel slag expansion, resulting in pavement heaves and disintegration, shall be a subject of future investigation.

A laboratory investigation on asphalt mixes incorporating crumb rubber additives at varying concentrations was carried out by Mohamed (2007). A novelty of the research was that the crumb rubber based additive can be blended via the dry process and made possible by incorporating activators. Anti-oxidants were also included in the crumb rubber additives to retard the hardening process. Samples blended with the crumb rubber based additives were oven aged in a specially fabricated oven fitted with ultraviolet lamp for short term and long term aging and tested for resilient modulus, indirect tensile strength, creep and fatigue. The results showed that the Marshall stability of crumb rubber modified mixes increased by 35%. Generally, the resilient modulus, indirect tensile strength and creep stiffness values of the crumb rubber modified mixes were found to be higher than the control mix. Furthermore, the long term ageing has a lesser adverse effect on crumb rubber modified mixes. For example, the resilient modulus values of control mixes has shown an increase of 64% after long term aged, while crumb rubber modified mixes indicated an increase of 7% to 40%. Mixes incorporating a mere 1% crumb rubber additives were found to exhibit longer fatigue life compared to the control mixes.

3.5 Porous Asphalt

For many years, the USM has carried out advanced research on PA and has developed it into a niche research area. The research endeavour has already extended into the realm of the two layer PA, one of the best candidate materials for quiet pavements. Various aggregate gradings have been developed based on various parameters aimed at a trade-off between mix performance and permeability. Various successful PA gradations globally were adapted to suit Malaysian quarry practice. A number of PA aggregate gradations have been developed from the theory of aggregate packing proposed by Cabrera and Hamzah (1996). According to the packing theory, when two aggregates of different sizes were blended, there is an optimum proportion of the two that leads to minimum air voids and that this air void is always less than the air voids of the individual fractions. The next finer fraction was then added incrementally until the coarse aggregate matrix combinations that results in minimum porosity, was attained, beyond which the fines were successively added so as not to bulk the mix but achieving a target air voids. Suitable gradations were selected based on performance parameters which typically involved a trade-off between mix stability or resistance to abrasion loss and permeability. PA made of steel slag, and those incorporating novel additives such as the Korean Drain Asphalt Modified Additive (DAMA) and Sasobit® have been evaluated.

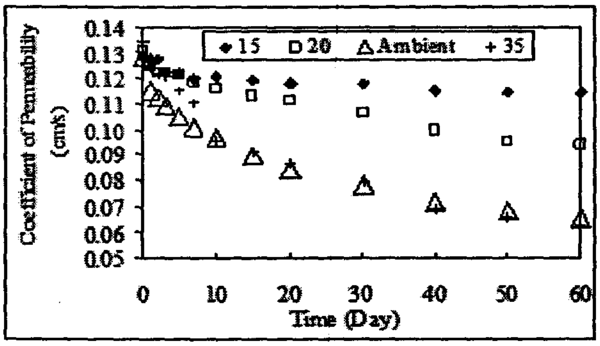
A lot has been mentioned about clogging of PA but there is a paucity of laboratory work to quantify or simulate clogging. A laboratory simulative clogging test was developed at the USM that enables researchers to realistically rank mixes in terms of their resistance to clogging. Samples were progressively clogged and permeability measured using suitable permeants and vacuumed once the critical permeability was attained. The process was repeated in a cylic manner until no amount of cleansing or vacuuming could alleviate the problem of clogging. The result indicated the obvious benefit of the two layer PA to mitigate clogging compared to the single layer PA. An on-going work at the USM looks at the effects of heat on cleansing efficiency of the clogged porous mixes.

The phenomenon of binder creep is another novel finding and is a unique USM initiative. It was earlier thought that PA looses permeability due to clogging and voids closure as a consequence of traffic overcompaction. Binder creep was discovered as another source of permeability loss in PA. To ascertain this, a falling head permeameter was used to monitor the permeability of PA mix conditioned at 15, 20, 30 and 35°C. Throughout the test, these specimens were not subjected to any applied load but protected from the intrusion of dust in the atmosphere. The result shown in Figure 2(a) shows the reduction in the coefficient of permeability, k, over time and the reduction is more pronounced on specimens conditioned at elevated temperatures. Specimens conditioned at 35°C lost approximately 50% of its permeability after 60 days compared to specimen conditioned and tested at 20°C. The phenomenon is attributed to binder creep. Bitumen is a visco-elastic material. At elevated temperature and long time of loading, it behaves as a viscous material and flow.

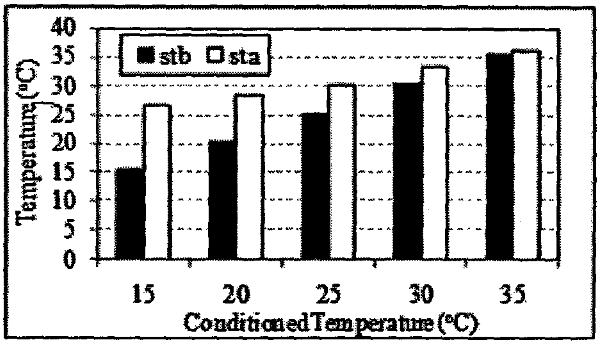
Over an extended period of time, gravitational forces act on the binder, causing it to flow. Elevated temperature will hasten the binder to flow and occupies the air voids within the porous mixes and subsequently disrupts voids continuity, thus causing the permeability to gradually reduce. Extended tests are being conducted to ascertain the phenomenon by carrying out CT scan at TU Delft on samples prepared at the USM. If binder creep is affirmed as one of the sources of permeability loss, it will change the manner in which permeability measurements are made whereby permeability measurements of PA must be conducted within a specified time frame after compaction. Otherwise, the coefficient of permeability results will be inaccurate due to binder creep interference.

Apart from clogging, the most serious problem that limits PA durability and service life is ravelling. In the laboratory, resistance to ravelling is evaluated via the Cantabro test while a more simulative rolling surface abrasion test was developed in the Netherlands by Van Bochove. In the field, PA is subjected to abrasive forces from vehicle tyres that could induce ravelling. In the laboratory, abrasive force is simulated by the Cantabro test. The Cantabro test criticized for it's lack of correlation with ravelling on roads, specifically since it favours polymer modified mixes which according to some countries do not correspond to practical results. Limiting values on abrasion loss are specified at either 18°C or 20°C or 25°C which typically represents the minimum value of the design bitumen content. However, the specified test temperatures may be more conducive for temperate countries. The European Union for instance, regulates an ambient temperature at 20±5°C while the European Standard 12697-17 (CEN, 2004) on Cantabro test recommends test temperature between 15°C and 25°C. From daytime temperature monitoring made at the USM, the average ambient temperature in the absence of air-condition, hovers around 30°C. Based on the limiting abrasion loss values, Malaysian researchers typically carry out the Cantabro test on specimens conditioned at 25°C without giving any regards to the effects of ambient temperature or temperature inside the Los Angeles steel drum. The Cantabro test is probably a very simple test. Typically, it took 10 minutes to complete the test and many researchers are not aware that subjecting the specimens to 300 drum rotations under Malaysian ambient temperature has caused the specimen temperature to increase while it is impossible to maintain the same specimen temperature throughout the 10 minute test duration. An investigation was carried out by separately conditioning PA samples at 15°C, 20°C, 25°C, 30°C (ambient) and 35°C. An infra-red thermometer was used to determine the change in specimen temperature before (stb) and after (stb) subjected to 300 rotations. Figure 2(b) shows the significant effect of initial conditioning temperature on temperature difference of the samples before and after tests. The temperature difference decreases as the initial conditioning temperatures increases. For instance, when the initial conditioning temperature of specimen is 15°C, at the end of the test, the PA specimen skin temperature has increased by approximately 11.16°C.

To investigate the effects of ambient temperatures, 15 cylindrical PA specimens were prepared with PG-76 binder at the USM and tested at the Section of Road and Railway Engineering, TU Delft. The ambient temperature at the time of test was about 24°C. The specimens were subjected to similar initial conditioning temperatures as at the USM. The abrasion loss test result and specimen temperature difference are different from the test results conducted at the USM as shown in Figure 3. In general, abrasion loss values at TU Delft are 31.7% higher and the difference between Dutch and Malaysian abrasion loss gets wider as the initial conditioning temperature increases. At TU Delft, specimens become warmer when initially conditioned at temperatures below ambient, if otherwise, the specimens become cooler. Hence, comparison of abrasion loss values worldwide can be erroneous without knowing the ambient temperature at which the Cantabro test was carried out. Ignoring the effects of initial conditioning and ambient temperatures probably explains the inconsistent Cantabro test results reported by researchers. This new finding is in addition to the well established fact that specimens are subjected to lesser abrasion loss as the binder content increases while incorporation of modified bitumen PG76 helps to improve the resistance to disintegration.

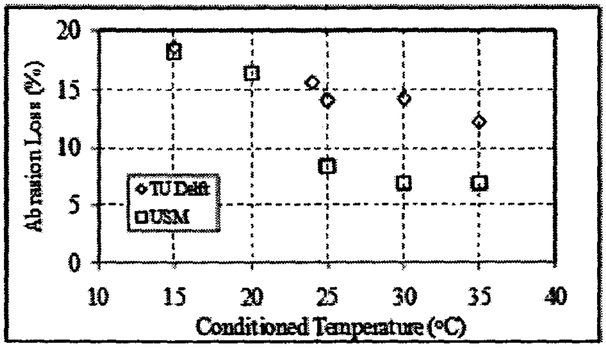


(a) Binder Creep

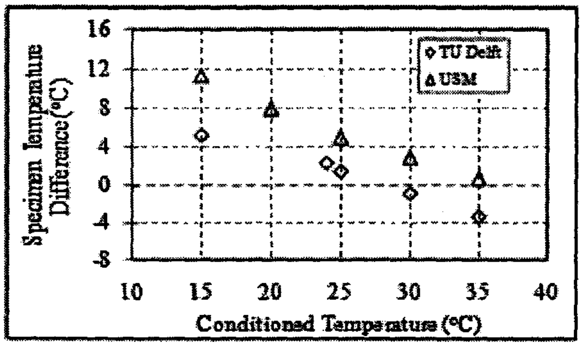


(b) Abrasion Loss

Figure 2: Porous Asphalt Binder Creep and Abrasion Loss Test Results



(a) Abrasion Loss Values



(b) Temperature Differences on Samples

Figure 3: Comparison on Cantabro Test at USM and TU Delft

4.0 THE WAY FORWARD AND SUSTAINABLE ASPHALT PRACTICES IN SOME EUROPEAN COUNTRIES

Malaysia is blessed with crude oil (hence bitumen) and enormous aggregate reserves. Nevertheless, these natural resources are depleting and once exploited will disappear forever. On the other hand, the oil industry prefers to crack bitumen to produce synthetic crude oil and other higher value added products. This further escalates the price of bitumen and is deeply felt by our asphalt industries. The European asphalt industries have responded to this scenario by various research efforts aimed at reducing asphalt cost production, conservation of the environment, quest for alternative binders and lengthening the service life of asphalt pavements. This more importantly, includes recycling, warm mix asphalt, quiet pavements and emphasis on performance based contracts.

In the area of warm mix asphalt, the French Laboratoire Central des ponts et Chaussees (LCPC) has oriented its research endeavour to developing materials that limits the consumption of energy and non-renewable resources. According to Chantal, (2009), the use of warm asphalt mix in France continually grows from 57000 tons in 2005 to 600000 tons in 2008 or the equivalent of 1.5% of French HMA production for that year. The importance of warm mix asphalt can be seen from the many EU sponsored projects related to the issue of low temperature mixes. Warm asphalts are developed by using additives that modify bitumen rheology or foam bitumen effects or successive (double) coating such that asphalt mixes can be produced and compacted at much lower temperatures. Depending on the technique used, energy savings up to 35% is feasible while CO_2 and NO_x emissions can be lessened up to 40% and 70% respectively. Apart from Sasobit®, many proprietary warm asphalt products has been developed, many of German and French origin, and this include Aspha-Min, WAM-Foam, Asphaltan-B, EvoTherm, CECA Base RT and many more. In 2009, foamed bitumen technology was tried in the Netherlands that enables asphalt to be laid at 100°C for use on Dutch heavily trafficked motorways.

Asphalt is 100% recycled. At the end of its service life, old asphalt is milled and new asphalt is laid. The milled aged asphalt is recycled with virgin mix for use in the asphalt pavement layers. In the Netherlands, specifications for RAP have been in place since 1990. In most European countries, asphalt recycling techniques such as hot in-place (HIR), cold in-place (CIR), full-depth reclamation (FDR) or in-plant recycling; is a common feature and has become a typical practice or norm rather than an exception. It is regarded as the environmentally preferred way of rehabilitating the existing pavements. Recycling involving RAP content exceeding 50% has been successfully tried for use in the binder course. However, recycling PA can be challenging since the recovered RAP binder has aged (hardened) considerably. Carrying out recycling in the conventional way may result in an aggregate coated with two layers of binder, a softer virgin binder coating a much stiffer aged binders. However, many PA surfacings has reached its service life and needs to be recycled, and there is more of it in view of its shorter design life (10,000 ton/year PA RAP in the Netherlands) while storage of RAP can be a problem. Most recycled products go into the lower layers, but many road rehabilitation projects implicate resurfacing of the upper layers hence there is more demand for it. In addition, PA mixes utilise good quality aggregates (minimum PSV 58 in the Netherlands) and it is not economical to recycle it for the lower layers. A Dutch company has successfully recycled 50% PA RAP into new PA mix having similar properties as a standard single layer Dutch PA. What is more fascinating is that the PA mix is regarded as WMA made possible using additives extracted from rapeseed. The PA material has been placed on a road trial for 3 years and still shows good performance.

From the environmental perspective, efforts to reduce traffic noise in Europe were geared subsequent to a series of European Union (EU) legislation on noise emission limitations. The Dutch Innovatieprogramma Geluid (IPG) program, concluded in 2007, was initiated in response to strict Dutch noise legislation rather than as a result of EU policy. However, the IPG 6 dBA noise reduction target from road surfacings was feasible with the two-layer PA, a Dutch invention. Despite its good noise reduction potential, the service life of the two-layer PA is shorter compared to the single layer PA. Thin surfacings, incorporating gap-graded gradation, is another viable material, although the noise reduction magnitude is not comparable to the two-layer PA. Poroelastic road surface has been experimented in several European countries. The material has been placed on trial in the

Netherlands in 2009 with noise reduction up to 8 dBA. However, the material suffers from durability problems. The binder used is polyurethane and escalates the cost of the mix.

The two-layer PA is made up of a very thin upper layer that cools rapidly during construction. A special paver has been invented to simultaneously pave the top finer and bottom coarser layers via a method as the hot-on-hot technique. On bridge decks, noise from joints can be a nuisance. A Dutch company, Heijmans Infrastructure BV has developed a 'jointless' joint comprising of a very flexible open-textured asphalt surface layer overlying a strain spreading interlayer made of specially patterned thin metal strips. Mitigation of noise at joints is a natural follow up to the noise reduction efforts on conventional road surfacings. Many European countries applied porous asphalt for its noise reduction potential. However, some of the most durable PA resides in the Netherlands. Among the factors contributing to the success story of Dutch PA include a rational aggregate gradation design, use of conventional binder in combination with favourable climatic conditions, good quarry and construction practices to meet functional contract requirements. The gradation design emphasised on the formation of a stone skeleton. The mortar part of the mix does not interfere or 'clog' the air voids, but rather serve as the 'glue' to bind the coarse aggregate particles together. This ensures air voids continuity with a very stable stone skeleton that is less susceptible to traffic overcompaction and a bitumen-filler system that is less prone to binder creep. Newly constructed PA surfacings was found to exhibit lower early life dry skid resistance due to bitu-planing. Dutch specifications state that the dry braking deceleration must be greater than 6.5 m/s^2, otherwise special sign "longer braking distance – new road surface" needs to be posted. Like elsewhere, ravelling is the dominant problem, especially at low temperatures. To mitigate this, an emulsion with a rejuvenator is sprayed on the PA surface just before ravelling starts. The rejuvenator activates the aged binder, restores its flexibility and enhances its self-healing property. If the ravelling process has already started, an open emulsion sand asphalt mix is applied in the upper part of the PA. It is important not to allow ravelling to take place. If left untreated, it will result in rapid pavement deterioration. The rotating surface abrasion test is a realistic equipment to evaluate ravelling in the laboratory. Only the hard shoulders are cleansed twice a year with the PA cleaner. The traffic lanes on the highways carry fast moving vehicles that provide the self cleansing effect. Field studies indicate that test sites with very open PA (voids content 25%) in hard shoulders resulted in reduced clogging. Clogging is caused by accumulation of dirt on bitumen layer or even embeds itself in it. This leads to work on dirt-repelling binders to prevent accumulation of clogging agents in the voids of the porous asphalt and improving cleansing efficiency. It is highly likely that in future, a very open PA will be specified on the hard shoulders. From the evaluation results of test sites, PA with 5.5% bitumen exhibits longer service life. The service life of PA with polymer modified binder and PA with pen grade bitumen with drainage inhibitors is similar. Current Dutch specify use of PA Plus, a mix with 5.5% 70/100 pen grade bitumen incorporating fibres as drainage inhibitors.

Studies on asphalt have been carried out on a microscopic level. In the field, the binder content of asphalt mixes was found to be lowest at the top and highest at the bottom, most likely due to binder drainage during construction. From CT scanning results, the two-layer PA was non-homogenous with the lowest voids at the transition zone between the finer and coarser layers. From thin sections studies at TU Aachen, the phenomenon of micro-ravelling has taken place in the asphalt mastics of aged pavement cores with pores being filled up by clean and bitumen-free fine grains. The study also modelled the ravelling process into deterioration of binder due to aging and loss of aggregate particles from the road surface. There is currently a major on-going research project on the self-healing of asphalt and other materials at TU Delft. In the 1970's, rest periods were introduced in laboratory fatigue tests and self-healing took place during this period which also simulates the duration when the pavement is not acted upon by traffic load. Repeated loading causes the development of micro cracks in asphaltic materials. Since asphalt binder is visco-elastic, it has the in-built mechanism or self-repairing property that can counteracts the damage. Innovative technologies using embedded microcapsules filled with maltenes and induction heating using steel wool is currently being experimented at TU Delft to hasten the healing rate of bituminous materials.

The amount of stored energy in 'black top' highway pavements have been largely ignored all this while. A Dutch company, Ooms Avernhorn Holding BV, has pioneered an innovative solution to meet man's energy needs by tapping energy from the pavements in the summer months, store them in underground acquifers and extracting them for use during the winter months. During the hot summer, pavement temperature in the Netherlands can rise to 55°C. This project has been implemented in several places in the Netherlands. In this respect, pavements in the tropics can be a source of supply of energy all year round.

Sustainability is very much related to efforts made to enhance durability hence lengthening pavement service life. Like the USA Perpetual Pavement concept, the Europeans have developed the Long Life Pavements. There is a tendency for road authorities to place more responsibilities on the contractor by lengthening the warranty period and to move towards performance based. Such contracts encourage contractors to innovate, provide solutions and conform to best construction practices in the field while contractors are given the freedom to innovate and choose their own materials and methods. Currently, Dutch road contractors have to provide a warranty of up to 7 years. Upon expiry of the contract, road measurements will be taken that enables the road authority to estimate the residual life of the pavement. The contractor will be granted a bonus if the pavement can still serve beyond 7 years. If the road fails during the warranty period, the contractor has to make do to the satisfaction of the road authority. If this implicates any road closure, the contractor has to pay to the road authority on the basis of a lane rental system.

On the other extreme, in view of the reliability of crude oil sources and developments in the oil industry, we have to think of the extreme scenario when one day there is no more bitumen for the road industry. To reduce dependency on bitumen, research and trials with a family of carbon-neutral plant-based binders that could eventually provide an alternative to the different grades of road bitumen, have been conducted in several European countries such as France, Norway and Poland. They are intended not only for use as conventional hot mix asphalt but also for pavement aesthetics. The binder can be made transparent, thus promoting the natural colour of the aggregates and hence creation of aesthetically-pleasing coloured surfacings particularly in urban settings. Such plant-based binder mixes could be laid using existing pavers at very low temperatures without significant loss of workability, hence reducing greenhouse gas emissions. Like conventional bituminous binders, plant-based binders can be emulsified for use as tack coat or prime coat materials. Similar to conventional HMA, mixes made with plant-based binders can be recycled. In France, the first road trial of the material took place in April 2003 by Colas in partnership with the General Council of the Département of Côtes d'Armor in the framework of its innovation project "The Road of the Future". After more than four years in service, Ballié and Delcroix (2008) reported that the road surfacings trial has performed in a very satisfactory manner under traffic. They provide surface characteristics that are at least as good as bituminous mixes of the same class, providing a safe comfortable ride for road users.

5.0 CONCLUSIONS

Research related to sustainable asphalt carried out at the Universiti Sains Malasia has been showcased. Incorporating Sasobit® in asphalt mixes enables reduction of mixing and compaction temperatures which helps to reduce road construction cost and emissions during construction. Mixes blended with waste materials such as reclaimed asphalt pavement, steel slag and crumb rubber in asphalt mixes exhibit equivalent if not better properties compared to virgin mixes. Incorporating asbuton as alternative material can reduce 50% usage of virgin asphalt binder in asphalt mixes without sacrificing mix properties. The binder creep phenomenon and the effects of initial temperature conditioning on the abrasion loss of porous asphalt are two interesting phenomena that merit further studies. In Europe, extensive research has been carried out aimed at road construction cost reduction and conservation of the environment to address this issue while many have been implemented in the field.

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REFERENCES

Alvarez, A. E., Martin, A. E., Estakhri, C. K., Button, J. W., Glover, C. J. and Jung, S. H. (2006), Synthesis of Current Practice on the Design, Construction and Maintenance of Porous Friction Courses, Texas Transportation Institute, The Texas A&M University System College Station, Texas.

Asphalt Institute, 2007. The Asphalt Handbook, Manual Series No. 4 (MS-4), 7th Ed, USA

Attanasi, E.D., (2008), Volatility of Bitumen Prices and Implications for the Industry. Natural Resources Research, Vol. 17, No. 4
Balliė, M. and Delcroix, T., (2008), Inovative Plant-Based Binder for Road Mixes and Pavement Surfacings, Proc. 4th Eurasphalt and Eurobitume Conference, Copenhagen, paper no. 402-018
Biro, S., Gandhi, T. and Amirkhanian, S.J., (2009). Determination of zero shear viscosity of warm asphalt binders, J.Construction and Building Material; 23(5):2080-2086
Brundtland, G.H.,et.al.(1987), Report of the World Commission on Environment and Development : Our Common Future, A/RES/42/187, United Nations
Cabrera, J.G. and Hamzah, M.O. (1996), Aggregate Grading Design for Porous Asphalt, In Performance and Durability of Bituminous Materials, Editors Cabrera J.G. and Dixon J.R., E&FN Spon Publisher (An Imprint of Chapman and Hall), London, p. 10-22
Chantal R. (2009), Developments with Low(er) Temperature Mix in France, Seminar on Low Energy Asphalt Mixes for Sustainable and Durable Pavement Solutions, Houten, the Netherlands
CEN, (2004), EN 12697-17:2004: E., Bituminous Mixtures - Test Methods for Hot Mix Asphalt – Part 17: Particle Loss of Porous Asphalt Specimen, European Committee for Standardisation, Brussels
Holtz,K, Eighmy,T.T, (2000) Scanning European Advances in the Use of Recycled Materials in Highway Constructio. Public Roads, Vol. 64, No.1
Huang, Y., Bird, R., Heidrich, O., (2009), Development of a life cycle assessment tool for construction and maintenance of asphalt pavements, Journal of Cleaner Production No. 17, pp 283 - 296
Huber, G. (2000), Performance of Open Graded Friction Course Mixes, NCHRP Synthesis of Highway Practice 284, TRB, National Research Council, Washington D. C
Hurley G, Prowell B. Evaluation of Sasobit® for use in warm mix asphalt. NCAT report 05-06, Auburn, USA (2005), pp 1-30
IEA, (2010), International Energy Agency: End-use petroleum product prices and average crude oil import cost March 2010. pp 1-6
Iriansyah, A.S., (2009) Full scale trial of warm-mix asbuton at Pangkalan Lima-Kumai Road, Central Kalimantan, Research and Development Centre for Road and Bridge Findings Colloquium. *(Translated from Indonesian language)*
JKR (2008), Standard Specification for Road Works (SPJ 2008-Section4: Flexible Pavement), Cawangan Jalan, Jabatan Kerja Raya Malaysia, Kuala Lumpur.
Kim, S., Loh, S. W., Zhai, H., and Bahia, H. U. (2001) Advanced Characterization of Crumb Rubber-Modified Asphalts, using Protocols Developed for Complex Binders. Transportation Research Record, 1767, Transportation Research Board, Washington, D.C., pp 15–24
Malaysia Country Report (2008), The 14th Asia Construct Conference, Tokyo, Japan, 22 – 23 Oct. 2008
Malaysia Country Report (2009), The 15th Asia Construct Conference, Kuala Lumpur, Malaysia, 19 – 21 Oct. 2009
Mohammad, L., S. Shadi, S.Cooper, (2008). Evaluation of asphalt mix containing warm mix additive, Proceedings of the Airfield and Highway Pavements Conference, ASCE
Mohamed,A.A.(2007). A study on the physical and mechanical properties of asphaltic concrete incorporating crumb rubber produced through dry process. PhD Thesis, Universiti Sains Malaysia
Palit, S. K., Reddy, K. S., and Pandey, B. B. (2004). Laboratory Evaluation of Crumb Rubber Modified Asphalt Mixes. Journal Materials in Civil engineering Vol. 16, No. 1, pp 45 -53

Roseen, R., Ballestero, T., Houle, J. (2007), University of New Hampshire Stormwater Center (UNHSC) Annual Report, University of New Hampshire, Durham
See, L.S. (1997), "Steel Slag Used in the Construction Industry", Natsteel Chemicals Ltd Internal report, Singapore.
See, L.S. and Hamzah, M.O. (2002), "Processed Steel Slag for Road Construction Industry", Fifth Malaysian Road Conference, Kuala Lumpur.
Subagio et al.,(2003), Development of laboratory performance of Indonesian rock asphalt (asbuton) in hot rolled asphalt mix. Proceeding of the Eastern Asia Society for Transportation Studies, Vol. 4, pp 436-449
Subagio et al.,(2005), Fatigue performance of HRA(Hot Rolled Asphalt) and Superpave mixes using Indonesian rock asphalt (asbuton) as fine aggregates and filler. Journal of the Eastern Asia Society for Tranportation Studies, Vol. 6, pp 1207-1216
Su, K., Hachiya, Y., and Maekawa, R.,(2009), Study on recycled asphalt concrete for use in surface course in airport pavement. Resources, Conservation and Recycling 54, pp 37-44
United Nations (1987), Report of the World Commission on Environment and Development." General Assembly Resolution 42/187, United Nations.
USDOT (1992), U.S. Department of Transportation, Report to Congress: A Study of the Use of Recycled Paving Material, FHWA-RD-93-147
USDOT (1997), U.S. Department of Transportation, Pavement Recycling Guidelines for State and Local Government, Participant's Reference Book,FHWA-SA-98-042.
Widyatmoko. I., (2008), Mechanistic-empirical mix design for hot mix asphalt pavement recycling. Construction and Building Materials 22, pp 77-87
Wasiuddin, N.M., S. Selvamohan, M.M. Zaman and Guegan, MLTA. (2008). Comparative laboratory study of Sasobit® and aspha-min additives in warm mix asphalt. Transportation Res Rec : J Transportation Res Board; (1998), pp 82-88
World Crude Oil Prices, (2010). U.S. Energy Information Administration http://tonto.eia.doe.gov/dnav/pet/pet_pri_wco_k_w.htm, accessed on 03/05/2010
Yi, T. C. (2008), Performance Evaluation of Steel Slag as Natural Aggregates Replacement in Asphaltic Concrete, M.sc Thesis, Universiti Sains Malaysia

Siri Syarahan Umum
Syarahan Umum Pelantikan Profesor
Pelantikan 2008/Bil. 6

PEMBANGUNAN LESTARI TEKNOLOGI ASFALT UNTUK MENJAMIN KELANGSUNGAN INDUSTRI ASFALT NEGARA

MEOR OTHMAN HAMZAH
Profesor dalam bidang Kejuruteraan Lebuh Raya
(Teknologi Asfalt)

PENERBIT USM
PENERBIT UNIVERSITI SAINS MALAYSIA
PULAU PINANG

4.2 Kitar Semula Asfalt

4.2.1 Penggunaan kitar semula asfalt

Di Malaysia, campuran asfalt direka bentuk untuk jangka hayat selama 10 tahun. Menurut praktik lazim, pada penghujung tempoh hayatnya, campuran asfalt usang dikisar dan lapisan asfalt baru (tindihan atas) dihamparkan. Memandangkan asfalt adalah bahan yang boleh dikitar semula sepenuhnya, asfalt yang dikisar wajar dikitar semula. *Reclaimed asphalt pavement* (RAP) merujuk kepada asfalt yang dikisar daripada turapan yang sudah usang. Bahan RAP masih lagi bernilai dan boleh digabungkan dengan campuran asfalt baru. RAP diadun bersama asfalt baru untuk menghasilkan campuran asfalt yang boleh dimanfaatkan semula. Memandangkan kos bitumen telah meningkat sebagai akibat kenaikan harga minyak mentah di pasaran dunia, kegiatan kitar semula semakin diberi perhatian. Kitar semula bukan hanya terbukti mendatangkan kebaikan daripada sudut ekonomi, malah dapat memulihara persekitaran dan sumber semula jadi serta selaras dengan amalan pembangunan lestari. Kitar semula turapan juga dapat mengekalkan aras permukaan jalan raya, mengurangkan penjanaan bahan terbuang dan menyelesaikan masalah berkaitan pelupusannya.

Di negara Belanda dan Denmark, 100% turapan usang dikitar semula, manakala 95% dan 55% RAP dikitar semula di Sweden dan Jerman, masing-masing (Holtz dan Eighmy 2000). Dalam tahun 1992 di Amerika Syarikat, 73 juta tan daripada 91 juta tan daripada bahan permukaan asfalt yang dihasilkan setiap tahun semasa proses pelebaran dan penurapan semula jalan dan lebuh raya, dikitar semula dan menjimatkan sekitar RM300 juta setiap tahun (USDOT 1997). Hal ini bersamaan dengan kadar kitar semula sebanyak 80% dan sekali gus menjadikan bahan turapan asfalt sebagai bahan yang paling banyak dikitar semula di Amerika Syarikat. Secara kuantitatif, isipadu asfalt yang dikitar semula di Amerika Syarikat adalah dua kali ganda gabungan isipadu kertas, gelas, plastik dan aluminium. Di Malaysia, kitar semula bermula dalam tahun 1993 dengan ketibaan jentera kitar semula yang pertama, sekali gus menjadikan Malaysia sebagai negara pertama di Asia yang menggunakan teknik kitar semula secara *cold-in-place* (CIPR) (Lewis 2004). Kajian ekonomi yang dilakukan oleh Horvath (2003) mendapati kos asfalt kitar semula yang menggabungkan 40% RAP dapat dikurangkan sebanyak 26% berbanding campuran tanpa RAP.

4.2.2 Kaedah kitar semula

RAP boleh dikitar melalui kaedah berikut:

(a) Kitar semula secara *hot-in-place* (HIPR)

Menurut kaedah HIPR, beberapa panel inframerah yang terdapat pada jentera *hot recycler* memanaskan permukaan turapan yang sudah terusia. Turapan yang panas digembur untuk menghasilkan RAP, dicampur dengan agen balik muda serta bitumen. Kemudian, skrid penurap meletakkan campuran untuk dipadat oleh jentera pemadat lazim. Kaedah ini juga boleh dilakukan oleh beberapa buah jentera; setiap jentera menjalankan aktiviti yang bebeza tetapi berturutan seperti yang digambarkan dalam Rajah 4.2. Dalam satu lagi kaedah, satu lapisan campuran baru diturap di atas lapisan bawah yang dikitar semula. Kesemua operasi dilaksanakan dalam satu laluan. Lazimnya, ketebalan maksimum lapisan yang boleh dikitar semula ialah 50 hingga 60 mm. Kaedah ini boleh memprofilkan semula geometri jalan raya dan merawat kerosakan permukaan turapan contohnya peretakan, ubah bentuk kekal, jurai atau pelucutan. Kaedah ini juga menjimatkan penggunaan bahan semula jadi, tidak mengganggu keterusan lalu lintas sedia ada dengan ketara dan kerja binaan dapat dilengkapkan dalam masa yang singkat.

Rajah 4.2 Lakaran proses kitar semula secara HIPR
Sumber: Wirtgen (2010)

(b) Kitar semula secara CIPR

Dalam kaedah CIPR, jentera kisar sejuk menghasilkan RAP dengan terlebih dahulu memecahkan dan mengisarkan permukaan turapan menggunakan perkakasan pemotongan yang dipasang pada sebuah gelendong. Selepas itu, agregat serta bahan pengikat seperti simen atau bitumen berbusa, gabungan bitumen berbusa dan simen dan bahan tambah lain yang diadun bersama RAP oleh jentera *cold recycler*

CIRI-CIRI ASFALT KITAR SEMULA

Oleh

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Pusat Pengajian Kejuruteraan Awam
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April 2009

MOISTURE SENSITIVITY OF ASPHALT MIXES INCORPORATING RECYCLED ASPHALT PAVEMENT (RAP)

By

NG TEE YIAN

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(Perak Branch)

Jabatan Kerja Raya
Perak Darul Ridzuan

Ref. No. IEM 10/PRK/102/C with 14 CPD hours

Prof. Dr. Meor Othman bin Hamzah

has participated as a speaker in the TWO-DAY COURSE on

Towards Sustainable Road Safety Design

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jointly organised by Jabatan Kerja Raya, Perak Darul Ridzuan
and The Institution of Engineers, Malaysia (Perak Branch)

Profesor Madya Dr. Md Azlin bin Md Said
Organising Chairman, The Institution of Engineers, Malaysia
(Perak Branch)

Dato' Ir. Dr. Safry Kamal bin Hj Ahmad, AMK, PPT
Director, Public Works Department
Public Works Department, Perak Darul Ridzuan