International Journal of Pavement Engineering

ISSN: 1029-8436 (Print) 1477-268X (Online) Journal homepage: http://www.tandfonline.com/loi/gpav20

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To cite this article: Abubeker Worake Ahmed, Safwat Fadhil Said, Xiaohu Lu & Håkan Carlsson (2018): Pavement performance follow-up and evaluation of polymer-modified test sections, International Journal of Pavement Engineering

To link to this article: https://doi.org/10.1080/10298436.2018.1435878

Published online: 13 Feb 2018.

INTERNATIONAL JOURNAL OF PAVEMENT ENGINEERING, 2018
https://doi.org/10.1080/10298436.2018.1435878

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Pavement performance follow-up and evaluation of polymer-modified test sections

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ABSTRACT
Between 2003 and 2006, a test road consisting of several conventional and polymer-modified structures was built on a motorway. Different combinations of styrene–butadiene–styrene (SBS) and ethyl vinyl acetate (EVA) polymer-modified binders were used. The test structures have been in service since then and have been monitored for over 9 years. The resistance of the different types of asphalt concrete mixes to rutting and cracking was measured and predicted. The impact of ageing on the mixes was also evaluated. Although all the sections are in good condition after 9 years of traffic, the predicted differences between the test sections based on the PEDRO (Permanent Deformation of asphalt concrete layers for Roads) approach and laboratory evaluations are noticeable. Lateral wander and transverse profile measurements indicated that studded winter tyre wear contributed to most of the rutting compared to permanent deformation due to heavy traffic. The unmodified mixes exhibited considerable ageing and the SBS-modified mixes were least affected by ageing. Furthermore, the SBS-modified base mix produced significantly better fatigue resistance than the conventional base mix. However, further investigations of the relationships between bitumen and mix properties and further follow-ups of the test sections are recommended to validate the findings.

ARTICLE HISTORY
Received 12 October 2017
Accepted 25 January 2018

KEYWORDS
Shear test; Polymer-modified asphalt mixtures; ageing; rutting; fatigue cracking

Introduction

Asphalt concrete mixtures deform or flow under the action of heavy traffic and climatic factors. The permanent deformation behaviour is characterised by two phases: the initial phase and the secondary phase. The initial phase is the volume decrease due to the effect of post-compaction that occurs during the first few years after construction. The secondary phase is characterised by shear deformation or flow rutting, which is mainly due to mix displacement at constant volume.

Manifestation of rutting depends on many factors; traffic variables, pavement structure, material characteristics and climate conditions. Traffic loading can vary depending on vehicle axle load configurations, axle loads, tyre types and inflation pressures, speeds and transversal distributions of vehicles. Vehicle loading induces stresses in the pavement through the tyre–pavement interface. The speed of vehicle loading influences the magnitude of stresses and strains in the pavement due to the viscoelastic characteristics of the bituminous mix and the mix properties in turn depend on pavement temperature. The magnitude of stresses and strains varies with both depth and horizontal distance from the loading centre of the tyre–pavement contact area. These parameters have a substantial effect on the precision of rut depth prediction on the pavement surface. The impact of wheel load is usually assessed by converting axle load into Equivalent Standard Axle Load (ESAL) using a load equivalence factor (LEF). The LEF is empirically determined from full-scale test road sections and it depends on traffic, structure, materials and climate at the field test in relation to pavement distresses (surface cracking and rutting). However, different test conditions and types of deterioration result in different LEFs. Ideally, one should combine the hourly traffic distribution with the hourly temperature distribution over a year due to the importance of temperature for rut prediction. Further, the tyre–pavement interface is usually assumed to have a circular contact area with a contact pressure equal to tyre pressure (Ahmed and Erlingsson 2015). Investigations in recent decades (Marshek *et al.* 1986, Tielking and Abraham 1994, Siddharthan *et al.* 2000) have reported that the tyre–pavement interface pressure may differ significantly from the tyre inflation pressure and is non-uniformly distributed within a circular contact area, which may induce considerable differences in pavement damage from using different types of tyre and varying tyre load and pressure. The studies recommended detailed vehicle traffic data and a better description of the tyre–pavement stresses to improve the predictive capability of the mechanistic pavement design models that are used in practice.

The main objective of this paper is to evaluate the resistance to permanent deformation of asphalt concrete mixtures from in-service test pavement sections. Both polymer-modified and unmodified bitumen are used in the different test road sections. Pavement structure and aggregate materials are kept identical for the test sections (Ulmgren and Aksell 2004, Malmqvist and Aksell 2006, Nordgren 2004, Lu *et al.* 2013). The evaluation is based on the prediction of rutting through laboratory-measured

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functional properties of mixes and field-measured rutting of the various test sections (Barksdale 1972, NCHRP 1-37A 2004). The linear viscoelastic permanent deformation model, PEDRO, (Oscarsson 2011, Said *et al.* 2011a, PEDRO software 2017, Said and Ahmed 2017) is employed to predict the long-term permanent deformation development in asphalt layers. The PEDRO model takes the effect of climate, traffic and material properties into account. The ranking of the test pavement structures based on the predicted permanent deformation is verified using laboratory test procedures.

Test pavement structures

The test section considered in this paper was built between 2003 and 2006 to study the performance of different types of polymer modifier. The base and binder course layers of the test sections were in service for about two years before the surface course was laid (Ulmgren and Aksell 2004, Malmqvist and Aksell 2006). The test road was divided into ten sections, each characterised by a combination of polymer modifiers applied to different layers, see Table 1. Figure 1 shows a cross section of the test structure. A plate loading test was conducted to ensure that the test sections were built on fairly similar unbound base layers.

Prior to construction of the test sections, a comprehensive laboratory study was conducted to select appropriate binder and polymer types. A detailed description of the selection of the binders and polymers can be found in (Nordgren 2004). The aggregate materials and grading curves are identical for all test sections and in accordance with Swedish specifications (ATB Väg 2002). It was concluded by Nordgren (2004) that three variants of the polymer-modified binders (PMBs) be used for the test structures. Two of the selected PMBs were variants of styrene–butadiene–styrene (SBS) (linear SBS and radial SBS) modified and the third one was ethyl vinyl acetate (EVA) copolymer modified. It is well known that SBS-modified binders have good elastic properties, thus resulting in small residual deformation after unloading. On the other hand, EVA-modified binders may have a relatively high modulus. The SBS-modified binders can therefore be suitable for pavement structures with relatively large elastic/recovered deformations, resulting in good resistance against fatigue cracking. The EVA-modified bitumen can be used to enhance resistance to permanent deformation in the binder layer (Nordgren 2004).

Table 1 shows the material descriptions of each test section and Table 2 the binder properties of the surface, binder and base course mixes. Figure 2 shows aggregate gradations. Note that PMB 50/70-53, 50/100-75 and 100/150-75 are currently 45/80-55, 40/100-75 and 90/150-75, respectively. A maximum aggregate size of 16 mm was used for the stone mastic asphalt surface course (SMA 16) and a maximum aggregate size of 22 mm was chosen for both the binder course (ABb 22) and the base course (AG 22).

Laboratory tests and material characterisation

Laboratory tests were conducted to establish the required material properties to investigate the permanent deformation behaviour and the fatigue cracking of the test sections. These tests included the dynamic shear modulus test (Said *et al.* 2014), the indirect tensile (IDT) stiffness modulus (EN 12697-26 2004 Annex C) and fatigue (EN 12697-24 2004 Annex E) tests, the repeated load creep test (RLCT) (EN 12697-25 2005 Test Method A) and for the binder the multiple stress creep recovery (MSCR) test (ASTM D7405) were conducted. The asphalt concrete samples were drilled from various sections of the test pavement before opening for traffic and after 6 years of service. Note that the surface layer was laid one year after the base and binder course layers; hence the surface layer was 5 years in service when samples were taken. The binder tests were conducted on the virgin and extracted bitumen.

Dynamic shear modulus tests

The master curves for the dynamic shear modulus and the phase angle for the surface, binder and base course mixes of the test structures are shown in Figures 3–5. The frequency sweep tests were conducted at temperatures of −5, 5, 20, 35 and 50 °C, and loading frequencies of 0.05, 0.1, 0.5, 1, 4, 8, 16 Hz. At least three specimens having a diameter of 150 mm and thickness of 35–40 mm were tested for each mix. A sigmoidal fitting function, shown in Equation (1), was used to fit the dynamic shear modulus. A fitting function shown in Equation (2) was used to fit the master curve for phase angle (Said *et al.* 2014). The Arrhenius' equation, Equation (3), was used as a shifting function.

$$\log(G) = \delta + \frac{\alpha}{1 + \exp\left(\beta - \gamma \log\left(f_r\right)\right)} \tag{1}$$

$$\phi = d\left(1 - \frac{e^{\frac{f_r - a}{e}}}{1 + e^{\frac{f_r - a}{e}}}\right) + \frac{c}{1 + (\frac{f_r - a}{b})^2} \tag{2}$$

$$\log(a_T) = R\left(\frac{1}{T + 273} - \frac{1}{T_{ref} + 273}\right) \tag{3a}$$

Table 1. Description of test structures.

Test Section/ Layer	Ref 1	1a	1b	2a	2b	3a	3b	4a	4b	Ref 2
Length (m)	400	275	275	124	226	128	292	398	102	260
Surface Course (SMA 16)	70/100	70/100	50/100-75 4% SBS-Radial			70/100	70/100	70/100	70/100	70/100
Binder Course (ABb 22)	50/70	50/70	50/70-53 6% EVA					50/70-53 3% SBS-linear		50/70
Upper Base Course (AG 22)	100/150	100/150-75 6% SBS-linear		100/150						
Lower Base Course (AG 22)	100/150	100/150-75 6% SBS-linear		100/150	160/220	160/220	100/150	160/220	100/150	100/150

Table 2. Mix composition and properties of the binder.

Layer	Binder types	Binder content % weight	Air void content %	Polymer % weight	Penetration @ 25 °C, 1/mm	Softening point, °C
Surface	70/100	6.2	1.6	0	77	46
	50/100-75 SBS	6.2	1.6	4	58	98
Binder Course	50/70	5.2	1.4	0	55	50
	50/70-53 EVA	5.2	1.4	6	52	66
	50/70-53 SBS	5.2	1.4	3	58	58
Base Course	100/150	4.5	5.1	0	127	43
	100/150-75 SBS	4.5	5.1	6	123	90
	160/220	4.5	5.1	0	190	38

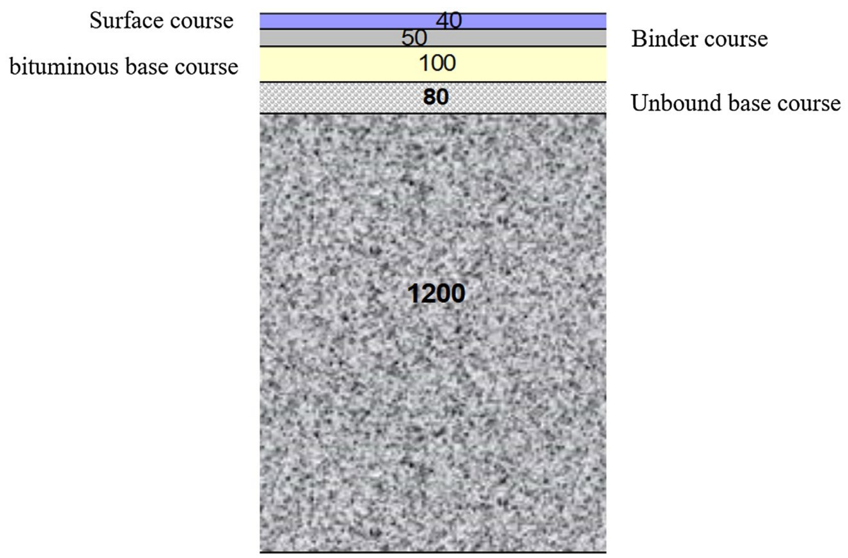

Figure 1. Test structure with layer thicknesses in mm.

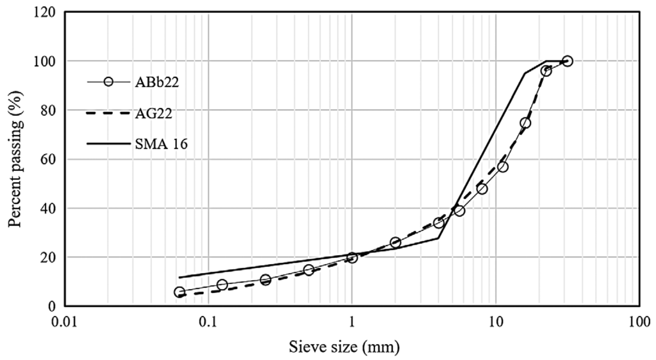

Figure 2. Grain size distribution of the surface, binder and base course mixes.

$$a_T = \frac{f_r}{f_T} \tag{3b}$$

where ϕ is phase angle, a, b, c, d and e are phase angle master curve fitting parameters, G is the dynamic shear modulus, f_r is the reduced frequency, f_T is the frequency at temperature T; α, β, γ and δ are sigmoidal fitting function parameters for the dynamic modulus master curve, a_T is the shift factor, T is the temperature in °C, T_{ref} = 10 °C is the reference temperature and R is the Arrhenius constant, R = 10,920. The shear tests were conducted only on cores drilled after 5 or 6 years of service.

The shear modulus master curves in Figure 3 indicated that the modified and unmodified surface mixes demonstrated a similar performance over a broader frequency and/or temperature range. Furthermore, it is known that asphalt concrete mixtures exhibit both viscous and elastic properties. These properties are measured using the phase angle parameter; for a purely elastic material the phase angle is thus 0° and for a purely viscous material 90°. From Figure 3(b) it can be seen that the phase angle of the SBS-modified surface mix was lower than the unmodified mix, indicating a better elastic property.

The unmodified binder mixes, on the other hand, produced a higher shear modulus and lower phase angle than both the EVA- and the SBS-modified mixes, Figure 4, signifying that the unmodified binder mix might demonstrate better rutting resistance than modified binder mixes in relation to the shear modulus and phase angle results. This might be attributed to a higher degree of ageing of the unmodified binder mixes after 5–6 years *in situ*. In contrast polymer modification might have improved the ageing characteristics of the mixes, as the SBS-modified mixes were found to be the least affected by ageing as it will be discussed in the following sections. Note that, though ageing has improved the resistance to rutting of the mixes, it might increase the risk of problems related to cracking due to the increase in stiffness.

Similarly, the SBS-modified base mixes produced better elastic properties, as shown in Figure 5(b) and (d). Figure 5(a) and (c), on the other hand, illustrate that the unmodified base course

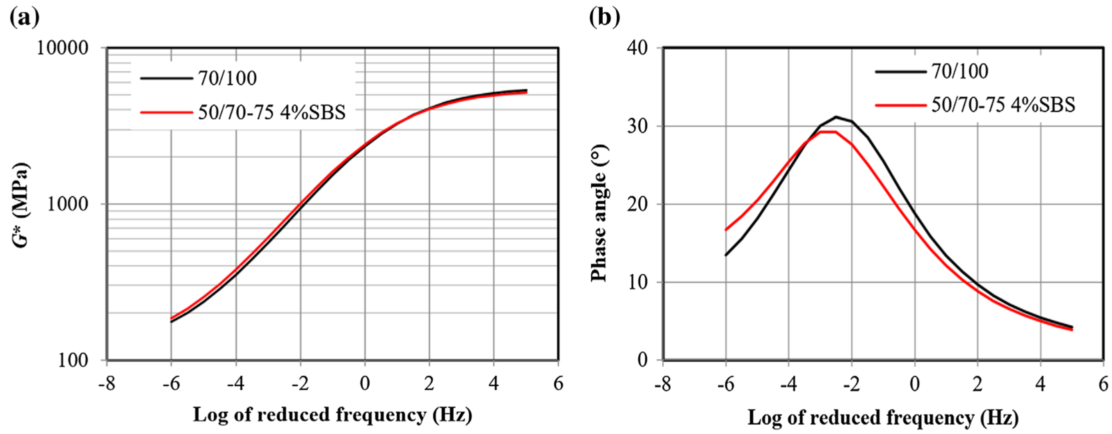

Figure 3. Master curves of (a) Dynamic shear modulus, (b) Phase angle of the surface course mixes SMA 16 with different binders at a reference temperature of 10°C.

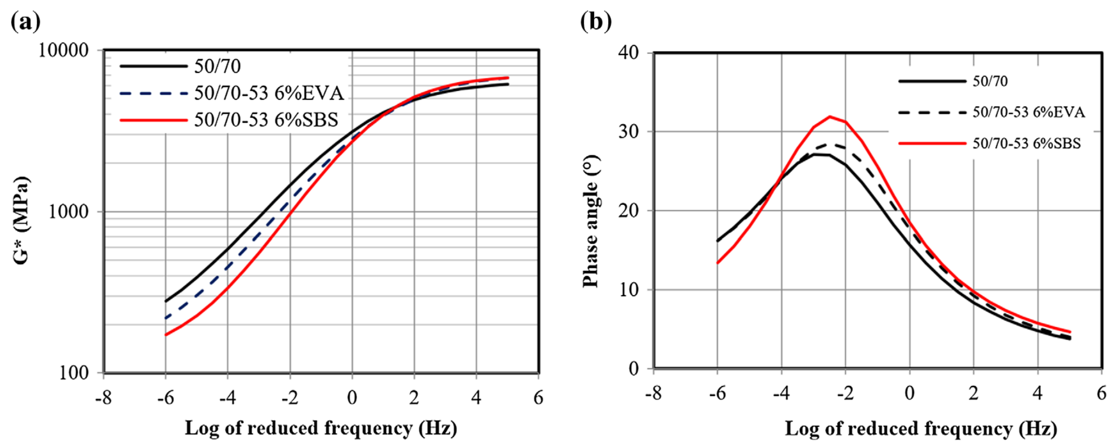

Figure 4. Master curves of (a) Dynamic shear modulus, (b) Phase angle of the binder course mixes ABb 22 with different binders at a reference temperature of 10 °C.

mixes produced a higher shear modulus than SBS-modified mixes. Note that both high shear (and/or stiffness) modulus and low phase angle are essential characteristics to enhance resistance of asphalt concrete against rutting. Further analysis was therefore carried out to clearly understand the effect of the polymer modifiers on resistance to rutting of the mixes by determining the viscosity of the mixes. The viscosity of mixes is a crucial factor in evaluation of permanent deformation in asphalt concrete materials (Hopman 1996, Said *et al.* 2011a). Mixes with higher viscosity are expected to show better resistance to permanent deformation but less resistance to fatigue cracking.

The combined effect of shear modulus and phase angle can be shown by viscosity (Said *et al.* 2014).

$$|\eta^*| = \frac{|G^*|}{\omega} \tag{4}$$

where η^* is complex viscosity in Pa s, G^* is complex shear modulus in Pa and ω is angular frequency rad/s at maximum value of phase angle, when the mix has the least resistance to permanent deformation.

As shown in Figure 6(a), the SBS-modified surface mix produced a higher viscosity compared to the conventional mix. This might have resulted from the harder binder (Pen 50/100) used for the modified surface mix (Table 1). On the other hand, the binder mix with SBS-modified bitumen clearly shows lower viscosity values than the other binder mixes. However, a slight difference in viscosities is obtained between the EVA polymer-modified and the unmodified mixes (Figure 6(b)). Note that the final mix composition of cores might have affected the results. Nevertheless, this suggests that the unmodified mix might demonstrate a better rutting resistance after five years *in situ*.

The effect of ageing might have played a significant role in the viscosity of the unmodified mix (Vonk *et al.* 1993). Similarly, as shown in Figure 6(c) and (d), despite the upper and lower unmodified base mixes having higher phase angles (Figure 5(b) and (d)), the unmodified base course mix (with pen 100/150) showed better resistance than modified base course mix (with 100/150-75 SBS) against permanent deformation based on the viscosity of the mixes, i.e. the influence of the combined effect of shear modulus and location of peak phase angle with respect to reduced frequency was of greater significance than the modified base course mix. Note that the position of the peak phase angle with respect to the reduced frequency in Figures 3–5 has an impact on the mix viscosity and mix resistance to permanent deformation.

The RLCT and the IDT stiffness tests on specimens from surface, binder and base course layers, presented in the subsequent sections, confirmed the findings from viscosity analysis.

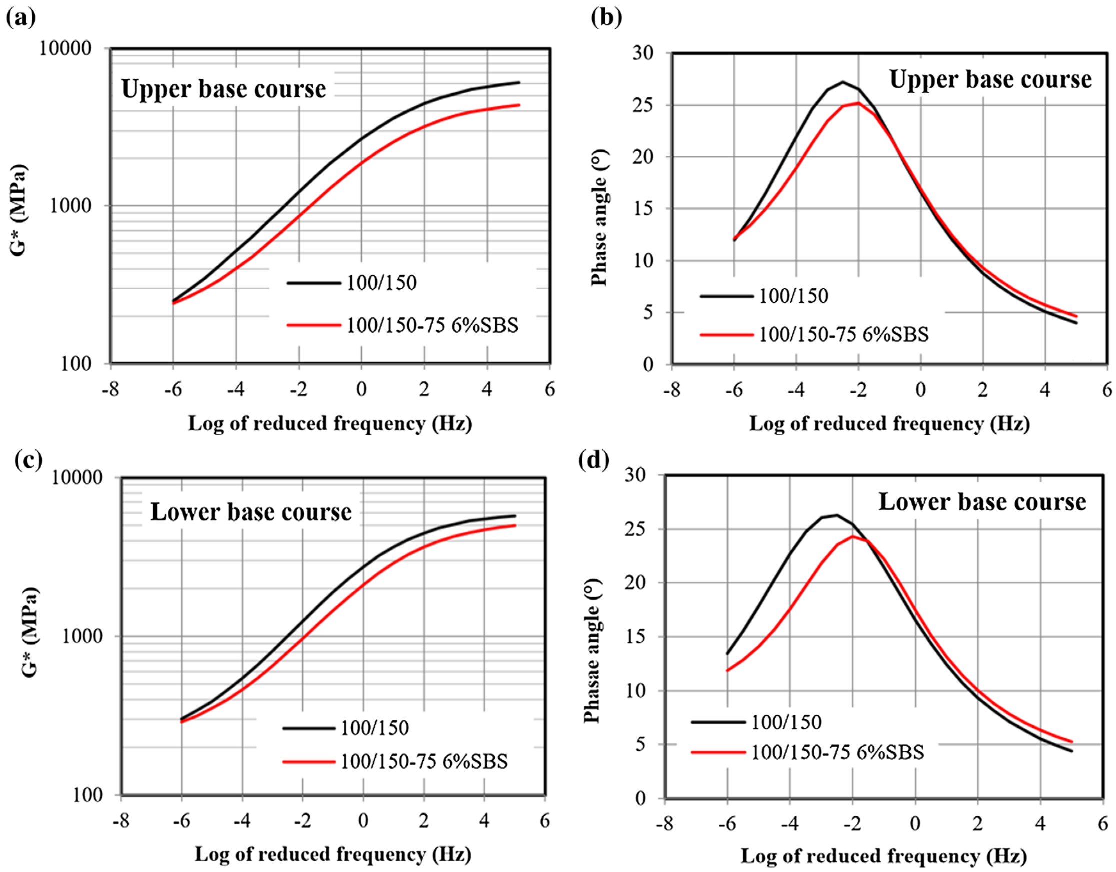

Figure 5. Master curves of (a) Shear modulus of upper base course, (b) Phase angle of upper base course mixes, (c) Shear modulus of lower base course and (d) Phase angle of lower base course, at a reference temperature of 10 °C.

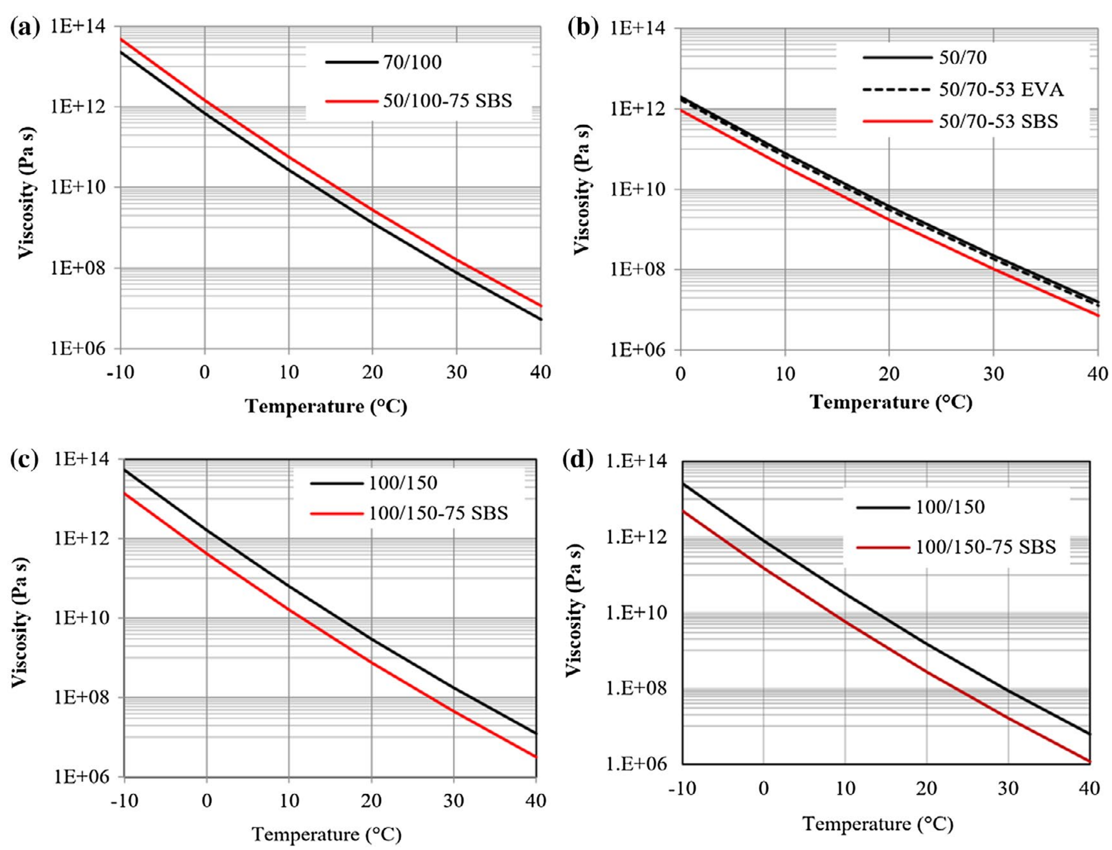

Figure 6. Complex viscosity at peak phase angle; (a) surfacing (SMA 16), (b) binder course (ABb 22) and (c) and (d) upper and lower base courses (AG 22), respectively.

Effect of ageing

To further investigate the effect of ageing through mechanical tests, RLCT and IDT stiffness modulus tests were conducted on field-cored samples from 2004/2005/2006, 2011 and 2016. The RLCT was conducted at a test temperature of 40 °C on cylindrical specimens having a diameter of 150 mm and thickness of 60 mm (sometimes two cores stacked together) (EN 12697-25 2005 Test Method A). The relative standard deviation (RSD) of the RLCT was below 20%. The IDT stiffness test was performed at three temperatures (5, 10 and 20°C) on 100 mm diameter specimen (EN 12697-26 2004 Annex C). The RSD of the IDT stiffness tests was generally between 5 and 10%. The long-term ageing indices, defined as the percentage of the relative increase in stiffness modulus per year (Said 2005), representing the pavement's ageing after its first year in service, were calculated based on IDT stiffness measurements at 10 °C. As shown in Figure 7, the SBS-modified surface and binder mixes were least affected by ageing. A similar conclusion was reached based on binder test results reported in Lu *et al.* (2014a, 2014b). However, unlike the surface and binder mixes, the SBS-modified base course mixes exhibited a slightly higher tendency to ageing than the corresponding unmodified mix that is unforeseen. The exact reason for this difference is unknown. The ageing process of asphalt concrete mixes is dependent on many factors (Glover *et al*. 2005, Petersen 2009, Lu *et al*. 2011). In addition to the climate conditions and elapsed time, origin of crude oil and refining process, aggregate mineral composition, type and content of additives or modifiers (like polymers) and their ageing process are believed important factors affecting ageing of a mix. However, this work is limited to evaluation of the pavement deterioration related to the properties of the mixes thus studying the ageing susceptibility of the asphalt mixes is beyond the scope of this paper. Note that the base course mixes, in general, exhibited higher ageing than the surface and binder mixes. This might be due to the higher air void content (5%), softer binders and thinner binder film of the base course mixes than the surface and binder course mixes (air voids <2%), thus permitting easier access to oxygen to accelerate the ageing process. This agrees with Read and Whiteoak (2003), i.e. that void content of the mixture is the main factor influencing bitumen hardening on the road.

The RLCT also revealed the effect of ageing of the binder and base course mixes, as observed by the change in measured creep strain. Similar to the IDT stiffness tests, the RLCT on binder mixes clearly indicated improved ageing properties of the SBS- and EVA-modified binder mixes, Figure 8, the SBS-modified binder mix being the least affected by ageing. Moreover, the RLCT on specimens from base course mixes, Figure 9, confirm the observations from viscosity analysis and ageing index based on the IDT stiffness measurements. Note that the SBS-modified base course mix exhibited much lower ageing than the unmodified mix. Accordingly, selection of mixes should be done considering the impact of differences in long-term ageing of mixes that might have a significant influence on pavement design. The impact of ageing on rut development will be discussed in a following section.

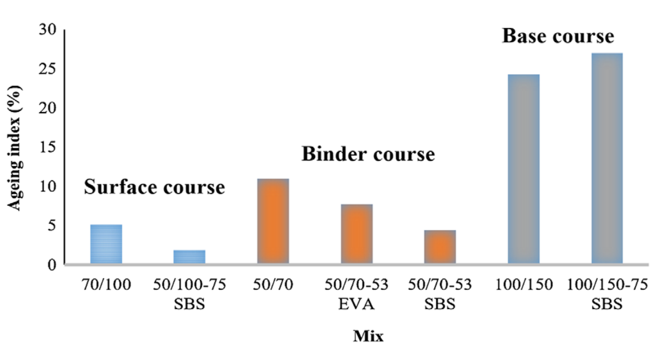

Figure 7. Ageing index at 10 °C.

Binder DSR and MSCR tests

The master curves of the complex shear modulus of the extracted binders were determined using DSR. The binders were extracted from the binder layers and the European standards EN 12697-1 and EN 12697-3 were followed for binder extraction and recovery. The solvent used was dichloromethane. Figure 10 shows that the PMBs produced a higher shear modulus compared to that of the conventional bitumen. Note that the EVA- and SBS-modified binders produced identical master curves. However, the observations from binder tests did not agree well with what was observed from tests on the asphalt mixtures.

Also, the MSCR test was performed at 60 °C on original (unaged), laboratory-aged, and the binders extracted from the test road. Typical examples of the MSCR curves are shown in Figure 11. In the MSCR test, 10 cycles of creep and recovery are performed at 2 stress levels (100 and 3200 Pa), each cycle consisting of one second loading and nine seconds recovery. Three replicates were tested for each binder and for the binders reported, repeatability (RSD at 3200 Pa) was less than 5% in terms of non-recoverable compliance, and 5–10% for strain recovery.

It is evident that the polymer-modified binders showed much higher strain recovery and lower non-recoverable strain as compared to the unmodified bitumen, suggesting strong polymer networks and higher rutting resistance for the modified binders.

Field measurements

The test road has been monitored since its opening to traffic. To follow up the deterioration in the pavement sections, measurements of deflection using a Falling Weight Deflectometer (FWD), rut depth measurements using a high-speed Road Surface Tester (RST) are performed periodically. In addition, a low-speed laser profilometer (PRIMAL) was used to measure a few four-metre wide transversal profiles for determination of a more accurate transversal profile.

Falling Weight Deflectometer (FWD)

The FWD measurements have been used to evaluate the road structures according to the Swedish standard (TRVMB 114 2012) with at least 10 measurements per test section during the autumn period (11–24 °C). Figure 12 shows calculated horizontal strains at the bottom of the bituminous layers after adjustment of strains at 10 °C (TRVMB 114 2012). Measurements were performed using a load of 50 kN. In general, the strain levels are less than 90 μmm/mm, indicating structures with good conditions

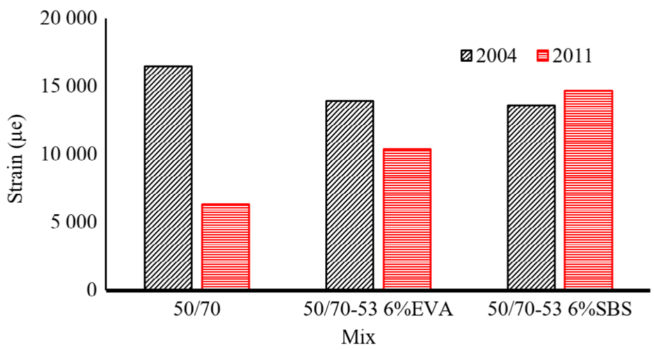

Figure 8. RLCT creep strain of the binder mixes from 2004 and 2011.

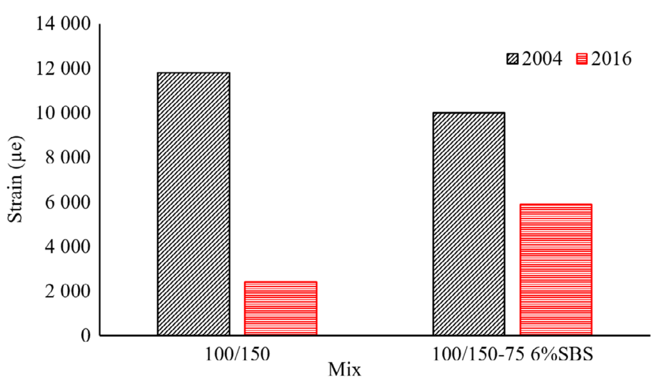

Figure 9. RLCT creep strain of base course mixes from 2004 and 2016.

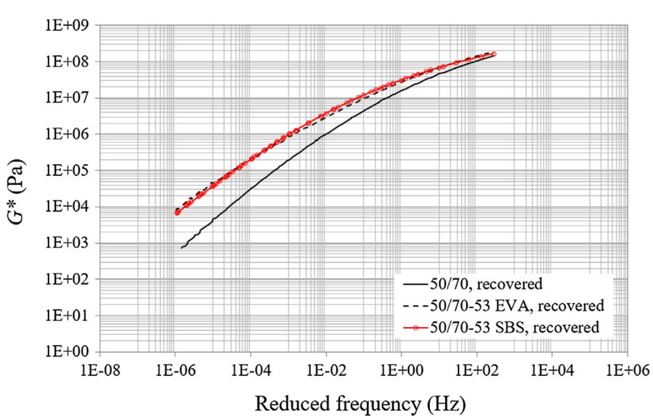

Figure 10. Master curves of the binders extracted from binder courses.

according to Swedish Transport Administration classification. The tendency of decreasing in strain level with time may be due to post-compaction of the pavement structures in the initial stage and ageing. The estimated strains for the year 2012 are used later in the evaluation of fatigue cracking performance of the pavement structures.

Rut depth and transverse profile measurements

A low-speed laser profilometer, Primal (Carlsson 2015), was used to measure transverse profiles of the test road over a lane width of approximately 4 m. The profilometer produces highly accurate measurements at intervals of 2 cm with an accuracy of 0.1 mm. Since measurement is time-consuming and could affect traffic flow, it is performed only to a limited extent. The aim is to elucidate the transverse profile of the road sections with the intention to determine the main source of rutting shown in Figure 13. Figure 14 shows the lateral distribution of three main groups of vehicle classes on test sections with the fitted normal distribution curves. The left-hand side of tyre position for all three groups is almost the same. However, the positions of the tyres on the right-hand side are quite different depending on the axle tracks of vehicle types (trucks and cars). The axle tracks are 1.5 and 1.8–2.1 m for cars and heavy vehicles, respectively. For further details of lateral wander measurements, see McGarvey (2016). Note that the distances between the maximum rut depths (rut bottoms) of left and right tyre tracks (Figure 13) are approximately 1500 mm for all test sections. Comparing the lateral wander of the traffic with transverse profiles indicates that the rutting is mainly dominated by wearing due to studded tyres of passenger cars. Therefore, only a small part of the total deformations could be associated to heavy vehicles.

To make a detailed evaluation of rutting and differentiate between wearing caused by studded tyres on cars and deformation caused by heavy traffic, the left-side of the wheel track was chosen for further analysis, where the left side tyre of all traffic types is positioned. The total rut development on the test sections was measured using a high-speed Road Surface Tester (RST) covering a lane width of 3.2 m. Figure 15 shows the normalised surface rut depth at the first measurement of the test sections in 2007 until 2016. A rut depth development of approximately 4 mm was measured after 9 years of service and rather small differences, less than 1 mm, were observed between the different sections. The differences are below the marginal error of the measurements. The section Ref 1 shows a greater rutting rate until 2009 that may be an effect of non-fully aged conventional mixes. Later, the rate of rutting is almost the same for all pavement structures. Note that the actual rutting of the test sections could be larger compared to the Profilometer measurements covering a lane width of four metres. Wiman *et al*. (2009) reported that rut depth using the RST covering 3.2 m results in 68% of rut depth from a profile covering a width of 4 m. Nevertheless, the average studded tyre wearing of SMA16 70/100 surface mix, described in the preceding sections, on a motorway with a similar traffic volume, is on average 0.3 mm per year, excluding the initial wearing after the first winter (Wiman *et al*. 2009). The total wear of the surface layers, which is the same for all sections, is therefore estimated to be approximately 2.7 mm, and the rut depth caused by heavy traffic is thus less than approximately 1.6 mm over 9 years in service depending on the test sections. It is important to note that the pavement's unbound layers and subgrade were checked to ensure that the test sections were built in a similar condition as discussed above. Therefore, no excessive differences in deformations, if they exist, are expected in the unbound layers and subgrade. The above argument is helpful in the modelling of permanent deformation described in the subsequent section. Further follow-ups of the test road are obviously needed for more accurate evaluation of the performance of the pavement structures with polymer-modified mixes.

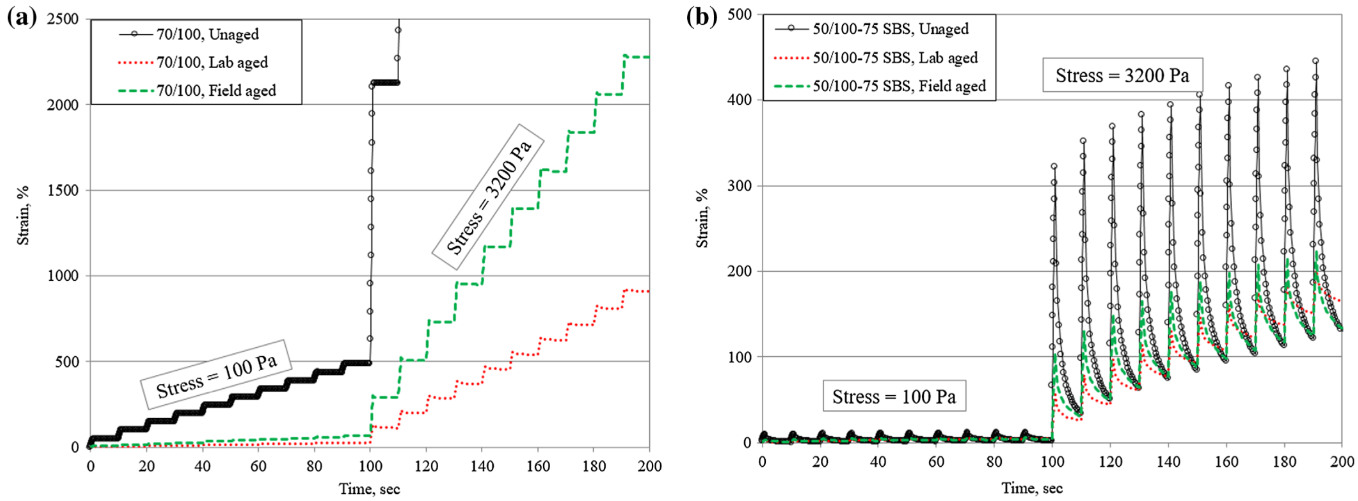

Figure 11. Examples of MSCR test at 60 °C for unmodified and SBS-modified binders.

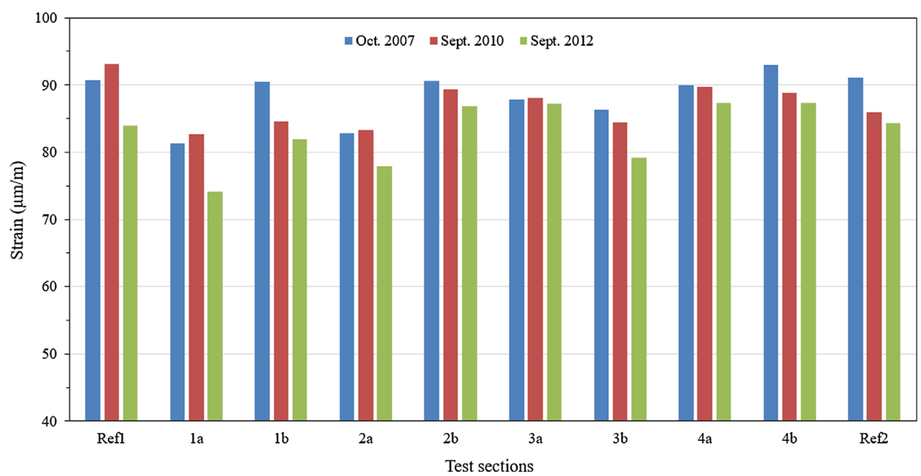

Figure 12. Estimated horizontal strain at the bottom of the bituminous layers adjusted to 10 °C.

Impact of ageing on rut development

Figure 16 shows the rut development based on transverse profile measurement using the Primal profilometer mentioned earlier. The reference sections (Ref 1 and Ref 2) consisting of the conventional mixes are compared with the sections 1b and 2a consisting of the PMB mixes in all bitumen-bound layers of the pavement structures (Table 1). The sections with the PMB mixes show less initial deformation than the sections with the conventional mixes, indicating higher resistance in the structures with the PMB mixes. This agrees with the results of the creep test performed in 2004 (Figures 8 and 9). In the secondary zone (linear zone of rut development), the rutting rate of the reference sections is obviously lower than the rutting rate of the sections with the PMB mixes. This agrees with the results of the creep tests performed in 2011 and 2016 (Figures 8 and 9), indicating less rut development in the pavement structures with the conventional mixes. Note that the changes in rutting resistance with time should be related to the differences in the ageing properties of the modified and unmodified mixes. This is consistent with the results reported by Airey (1997) and Vonk *et al.* (1993) that SBS-modified bitumen is less sensitive to ageing.

Performance evaluation of the test sections

Resistance to fatigue cracking

To study the differences between the test sections with conventional and polymer-modified base mixes, the fatigue life of each section was predicted with regard to damage caused by traffic loading. The reference sections Ref 1 and Ref 2 with conventional base mixes and sections 1a and 1b with SBS-modified bitumen base mixes were investigated in respect of bottom-up fatigue cracking. Sections 1a and 1b are identical structures except that section 1b also has polymer-modified surface and binder layers, see Table 1. Note that sections 2b, 3a and 4a with the base mix containing bitumen pen 160/220 is not included here due to missing fatigue test data on their base mixes. The evaluation was performed at 10 °C since the FWD measurements and the fatigue tests were performed only at approximately 10 °C (strains were adjusted to 10 °C according to the TRVMB 114 (2012) test method). Using the strain measurements based on the FWD of September 2012, shown in Figure 12, the estimated allowable number of ESALs to fatigue cracking of the base courses were determined and are shown in Figure 17(a) and (b). The resistance

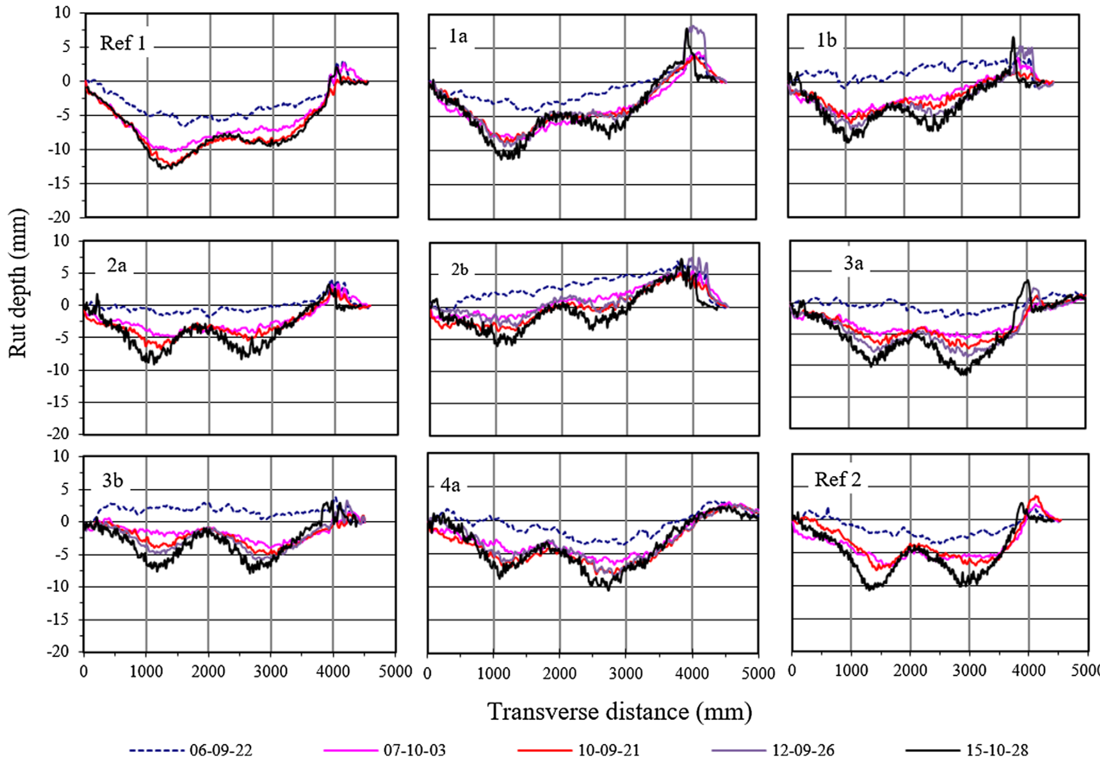

Figure 13. Transverse profiles of the test sections developed over time.

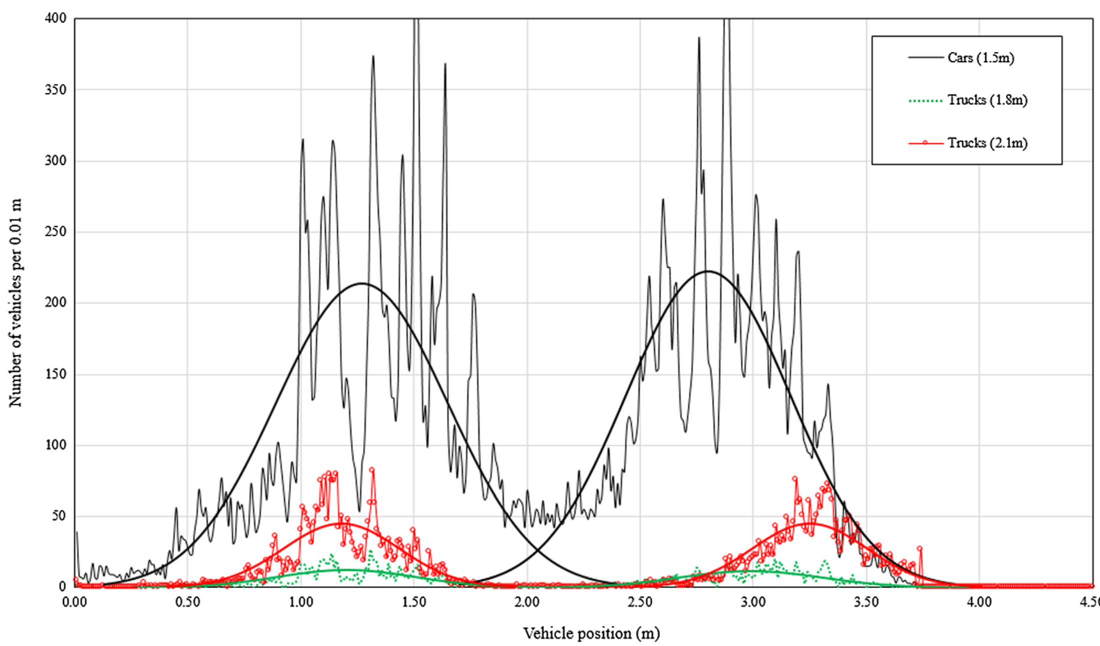

Figure 14. Distribution of lateral position of traffic at the test road motorway E6.

to fatigue cracking of the reference sections and the sections with SBS-modified base mixes, Figure 17(a), is on average 0.6E6 and 3.5E6 loading repetitions, respectively. The SBS-modified base mix produced significantly better fatigue cracking resistance. Figure 17(b) illustrates the accumulated 100 kN ESALs extrapolated based on traffic monitoring between 2005 and 2009. An equivalent axle load factor of 1.3 was used in the prediction of accumulated ESALs according to Swedish practice. The conventional structures were estimated to withstand the traffic loading until around 2024 and the structures with modified bitumen were estimated to withstand the traffic loading until between 2058 and 2071. The laboratory to field shift factor, when predicting pavement fatigue cracking performance, is approximately 24 depending on the thickness of the bitumen-bound pavement layers for similar pavement structures in Sweden (Said *et al.* 2011b). The approach shows that the conventional

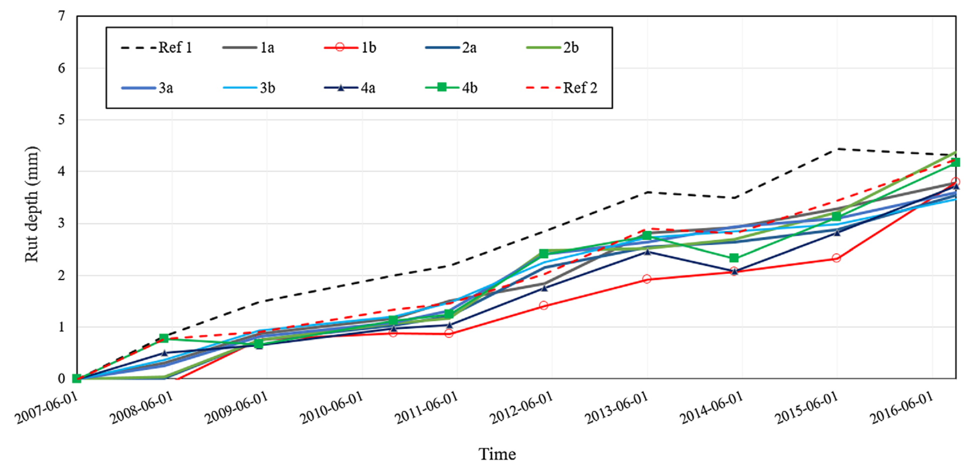

Figure 15. Average rut development in the left wheel track from RST measurements.

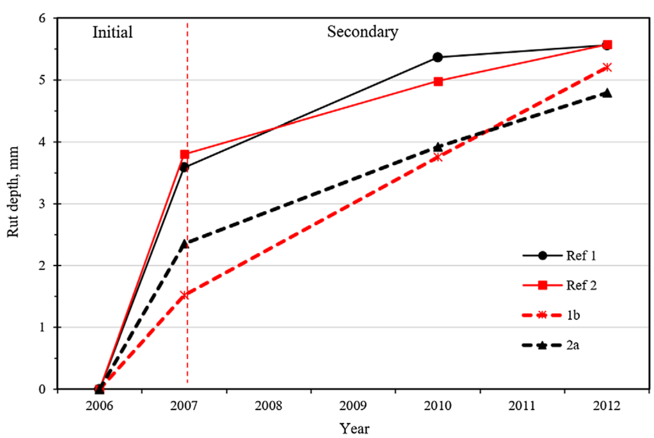

Figure 16. Impact of ageing on modified and unmodified mixes at initial and secondary zones.

structures (Ref 1 & Ref 2) were optimised with respect to fatigue cracking in accordance with the Swedish specification for roads with a technical design life of approximately 20 years' resistance against fatigue cracking. The structures (1a & 1b) with modified bitumen in the base mixes demonstrate at least 34 years' longer fatigue life than the conventional structures based on measurements at 10 °C. This could mean that the pavement structures with the SBS-modified bitumen base mixes are over-designed and can thus be classified as long-lasting pavements and it might be possible to reduce the pavement thickness of the structures with the SBS-modified bitumen in base mix in future pavement design.

Modelling resistance to permanent deformation

In this work, the linear viscoelastic uni-layer permanent strain model for asphalt layers PEDRO (the PErmanent Deformation of asphalt concrete layers for ROads) is employed (Said *et al.* 2011a, PEDRO 2017) to predict the permanent deformation in the bituminous layers. The PEDRO model has two components, viz. the primary and secondary phases, to accurately predict the effect of the initial stage of compaction or volume decrease and shear flow, respectively. The input data for the PEDRO model are the viscosity of bituminous layers, hourly traffic volume, traffic loading and speed, standard deviation of the lateral traffic wander, hourly climate data and the thickness of the bituminous layers. The model calculates the permanent strain at a desired location in a Cartesian coordinates system. In the model, the viscosity at the peak phase angle of the asphalt mixtures is used as an input. The complex function PEDRO model is shown in Equation (5). A detailed description of the model can be found in Said *et al.* (2011a) and derivation of the constitutive relationship in Björklund (1984).

$$\varepsilon_p = \frac{\sigma_0(1-2v)}{V\eta_p}\mathrm{Re}\left[\sqrt{(z+ix)^2+a^2}-(z+ix)\right] + \frac{\sigma_0 z}{V\eta_p}Re\left[1-\frac{z+ix}{\sqrt{(z+ix)^2+a^2}}\right] \quad (5)$$

where ε_p is the permanent vertical strain (μm/m), σ_0 is the contact pressure (Pa), v is the Poisson's ratio, η_p is the shear viscosity at peak phase angle (Pa s), a is the contact radius (m), x and z are the coordinates of the computation point (m), V is the vehicle speed (m/s) and i is the imaginary number, $i = \sqrt{-1}$.

Traffic and climate data

The traffic axle load spectra near the test site from the national WIM database were selected for the analysis. Figure 18 presents the axle load spectra for the selected WIM station and Table 3 the vehicle-related parameters. Vehicle speed and traffic growth rate were acquired from the Swedish Transportation Administration database. Measured standard deviation of heavy vehicle lateral wander for the site is 0.28 m (McGarvey 2016).

Similarly, the weather data from a nearby station were processed to derive the hourly and monthly temperature distribution. The daily and monthly temperature distribution at the centre of the surface, binder and base course layers was calculated using a climate model (Hermansson 2002). Figure 19 shows the daily and monthly distribution of the temperature at 2 cm below the surface of the pavement.

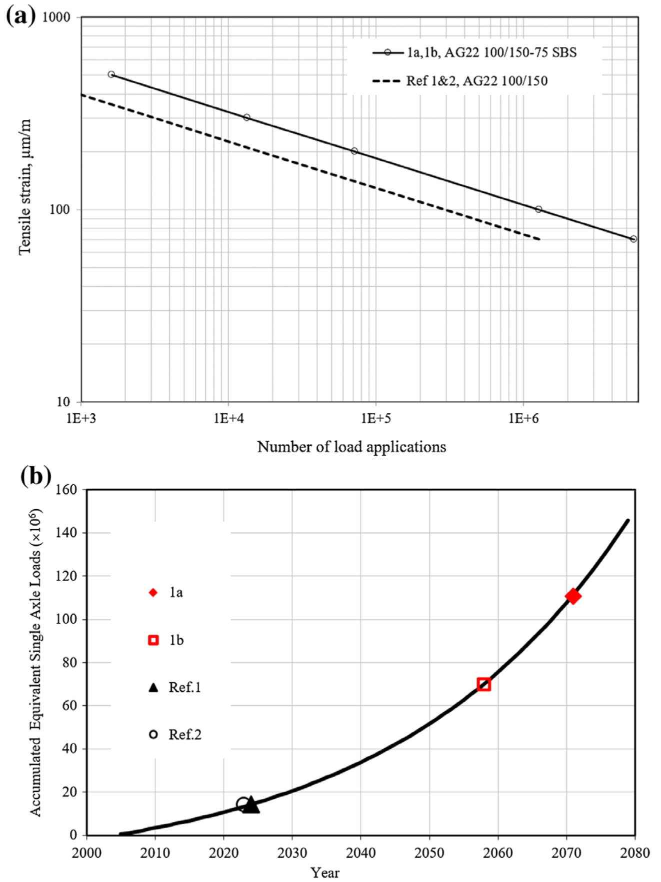

Figure 17. Laboratory-determined fatigue life relationships (a) and service lives of pavement structures based on estimated strains from FWD deflection measurements at 10 °C (b).

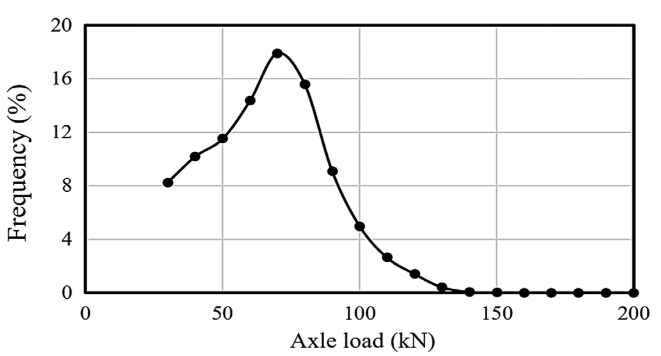

Figure 18. Axle load spectra of the selected WIM station.

Table 3. Analysis parameters.

Vehicle speed, measured	90 km/h
Traffic wander standard deviation, measured	0.28 m
Per cent dual wheels, assumed	50%
Centre to centre distance between wheels	0.3 m
Tyre pressure, assumed	800 kPa
Annual average daily truck traffic	8500
Traffic growth rate, measured	3%
Design or analysis period	20 years

Results and discussion of permanent deformation modelling

In this investigation, rut development of the test pavement structures subjected to traffic loading over a nine-year period is evaluated using the predicted rutting from the PEDRO model. The deformation is calculated successively for each axle loading in the axle load spectra. The rut depth is calculated by integrating the permanent deformation over the thickness of the asphalt concrete layers.

Figure 20 presents the predicted permanent deformation of the asphalt layers for test structures Ref 1, 1a, 1b, 2a, 3b, 4b and Ref 2. All rut depth measurements were normalised at the time of the first RST measurement in June 2007. An analysis period of 20 years was considered for the purposes of this study. The inputs to PEDRO for the test road include 50% dual tyre (11R22.5) and 50% single tyre (425R22.5). Note that the tested asphalt concrete cores were already aged at the time of testing (about 6 years) and the viscosity of the asphalt concrete layers was therefore adjusted to include the effect of ageing (ATB Väg 2002, Said 2005).

The test pavement structures have been monitored since the road opened to traffic in 2006. So far, no fatigue distress has been observed in the structures. A rut depth development including wearing and rutting of about 0.5 mm per year was observed after 9 years of service and rather small differences (about 1 mm) were observed between the different sections based on annual road surface tester (RST) measurements. The differences between test sections are thus not significant. Nevertheless, the PEDRO model was calibrated based on the 9 years of rut measurements. Single-field calibration factors were found effective for each test section. A calibration procedure should, however, be interpreted based on the variables of the model. In this case, the variables are traffic parameters, temperature, layer thicknesses and asphalt concrete viscosities. The asphalt mixtures' viscosity is the only variable in this work and the differences in the rut depths on the test sections are within the marginal errors of the RST measurements, and further studies are therefore needed before establishing a calibration procedure. Also, the calibration procedure should be related to rut formation at the primary and secondary stages.

The evaluation of the rutting performance of the different test structures was carried out based on the predicted permanent deformation over an analysis period of 20 years. Accordingly, all test structures show high stability based on nine-year field measurements and estimated rut depth, with less than 5 mm rutting in bituminous layers. The differences in rut depths between test sections are insignificant, whereas the conventional test structures, Ref 1 and Ref 2, indicated somewhat lower rut resistance even though rutting is only 4.6 mm and 2.7 mm respectively after 20 years. Ageing might have played a significant role in respect of the rutting performance, as discussed in the preceding sections. Mixtures with conventional bitumen (Ref 1 and Ref 2) have been aged significantly greater than mixtures with SBS-modified binder. The structures with polymer-modified binders were designed to be more rut-resistant than conventional structures. The benefit of a high-viscosity binder layer located at the pavement depth where the maximum shear stresses occur is clear in reducing the permanent vertical strains. It is also concluded

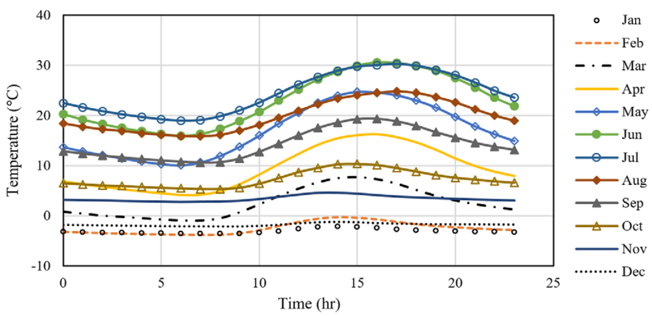

Figure 19. Monthly and hourly temperature variation at 2 cm below the surface.

that the base layers located deeper than 90 mm in the pavement are exposed to high shear stresses, showing relatively high permanent strains.

Annual field measurements and inspections showed that all the sections are in good condition. Furthermore, other distress types such as stripping and low-temperature cracking have not been observed on the test road.

Conclusions and recommendations

This paper has evaluated the performance of conventional and polymer-modified asphalt concrete mixes of an in-service test

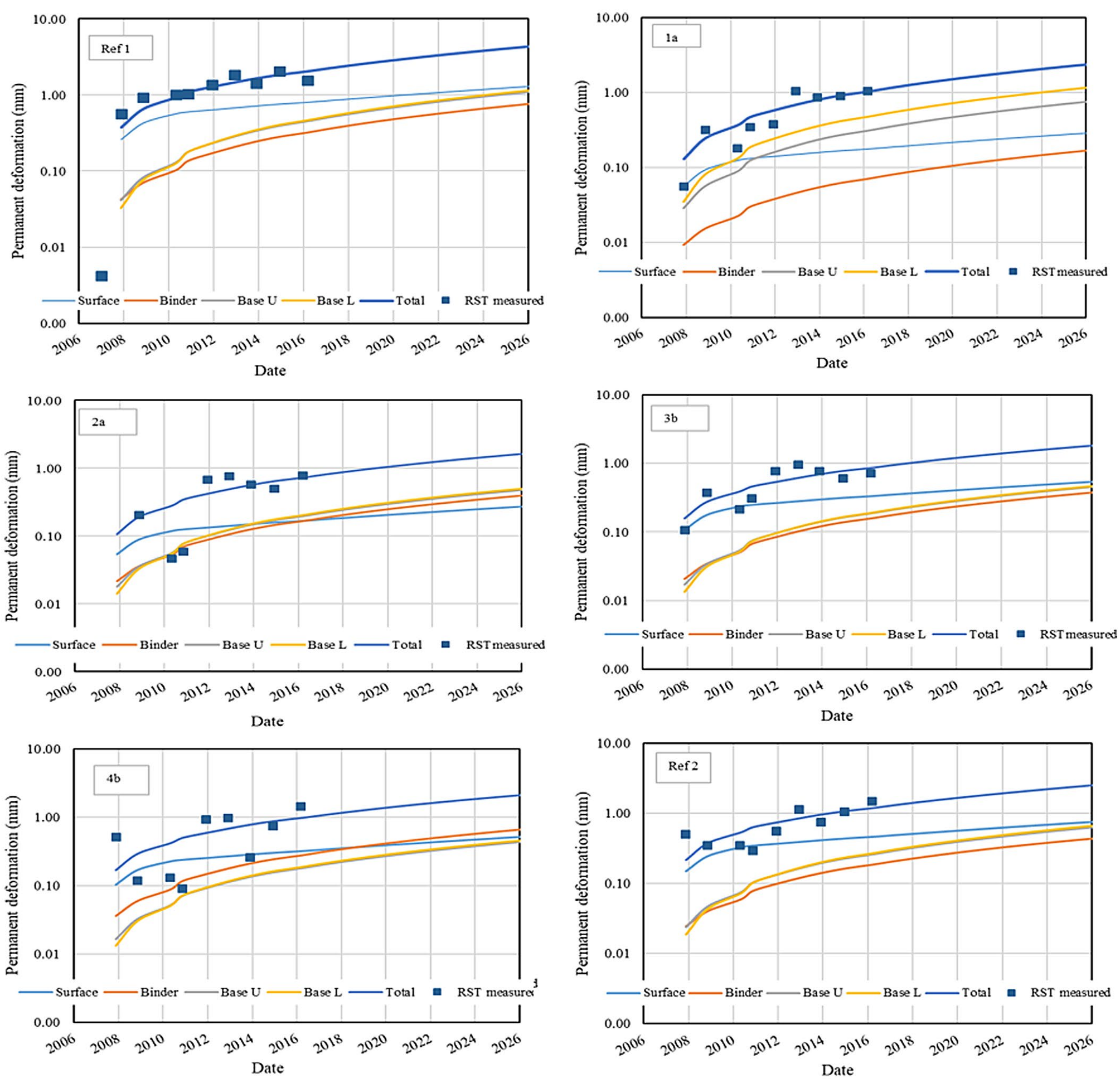

Figure 20. Measured and modelled permanent deformation of the pavement structures.

pavement structure based on shear box tests and other mechanical laboratory tests conducted on asphalt field cores. Results of indirect tensile test (IDT) stiffness modulus tests and repeated load creep tests (RLCT) were used to evaluate the ageing properties of the mixes. It was observed that both IDT stiffness tests and the RLCT indicated that the unmodified mixes exhibited considerable ageing, while the SBS-modified surface and binder mixes were least affected by ageing. The SBS-modified base mixes, however, exhibited slight ageing, as indicated by the ageing indices. This is in accordance with earlier investigations. The SBS-modified bases mixes therefore produced significantly better fatigue cracking performance than the conventional base mix.

In addition, it may be concluded that the shear box tests and the resulting shear modulus, phase angle and viscosity could be used to evaluate the performance of conventional or modified asphalt concrete mixes over a broader range of loading frequencies and temperatures. However, the observations from the tests on the original and extracted binders did not agree with what was observed from tests on the asphalt mixtures and further investigations are needed to clarify the contradictory results.

Finally, *in situ* rutting measurements show somewhat higher rutting in the reference sections in the initial stage but a slightly lower rutting rate in the secondary stage compared to the pavement structures with modified binders. The reason could be higher ageing characteristics of conventional mixtures than mixes with modified binders when ageing mostly occurs at the initial stage (2–3 years). However, all the test structures show high stability of bitumen bounded layers against rutting and the differences are within the marginal measurement errors of the Road Surface Tester.

The PEDRO model was employed successfully to evaluate the resistance to rutting of the test pavement structures. Accordingly, both the predicted rutting and the measurements show high stability of the test pavement structures against permanent deformation.

Further investigations are recommended to study the influence of the ageing properties of mixes on rut development since mixtures with modified binders exhibit a better resistance against ageing than mixes with neat binders that could have a significant influence on rut formation and fatigue cracking that should be taken into account in pavement design.

Disclosure statement

No potential conflict of interest was reported by the authors.

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