A laboratory study into factors affecting stone mastic asphalt performance

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ABSTRACT

There has been considerable interest in stone mastic asphalt in Australia and elsewhere. Two of the distinguishing features of stone mastic asphalt are the relatively high proportion of binder mastic and the establishment of a stone-on-stone aggregate skeleton.

Binder mastic has been characterised in the past by a simple binder/filler ratio but with the greater proportion of binder mastic in stone mastic asphalts, this simple parameter does not fully explain the role of the mastic. The binder mastic can have a profound effect on stone mastic asphalt performance. Free binder ratio and fixed binder ratio are shown to be closely related to the mastic viscosity and are examined in the laboratory for a range of added fillers.

The aggregate skeleton of stone mastic asphalts is commonly described as a stone-on-stone skeleton highlighting that the large proportion of coarse sized aggregate particles carry a substantial share of the imposed loads. There have been numerous studies in developing methods to ensure that this type of aggregate skeleton is developed and the paper examines a dilation procedure (based upon the change in resilient modulus in laboratory compacted specimens).

INTRODUCTION

Stone mastic asphalt (SMA) is a hot mix asphalt with a coarse gap-graded particle size distribution most often used as a wearing course. The grading is characterised by a stone-on-stone aggregate skeleton which transfers the traffic loading to the underlying layer. In Australia, this type of mix is nearly always used as a surfacing layer (Wonson 1996). There have been some instances with the SMA not performing as anticipated (Glen 2006). The reasons for the poor performance were unclear and a project was established to develop a design procedure that produced an SMA that had a more consistent performance.

SMA was introduced in Germany in the 1960s by Dr. Zichner (Drüeschner 2005) to overcome problems arising from the use of studded tyres to provide traction in icy and slippery conditions experienced during winter. The use of studded tyres was banned in the early 1970s, however, SMA continued to be used because of its excellent performance characteristics.

The original design concept for SMA required the use of high quality crushed aggregates. The prime concept with SMA is that it must have an aggregate skeleton that ensures the coarse particles are in contact and support the traffic load and this is often referred to as a stone-on-stone gradation. Since about 1998, polymer modified binders have been used as the binder of choice in Germany.

Bituminous beton mince (Michaut 1995) is a French product very similar to SMA.

Asphalt manufacture and placement require temperatures similar to that of the more commonly used dense-graded asphalt. Typical production temperatures are about 165 ˚C.

An SMA workshop was held at ARRB Group offices in Melbourne and reported on by Dr. John Oliver (Austroads 2007). One of the main outcomes of the Workshop was a clear definition of what constituted an SMA and what the typical properties were. These are reproduced below.

The definition and properties of SMA adopted by the workshop were:

- Stone mastic asphalt is a hot mix asphalt surfacing which is characterised by a high percentage of crushed coarse aggregate with good 'stone-on-stone' contact and a relatively high percentage of bitumen rich mastic.
- The principal properties of standard SMA, relative to dense graded asphalt (DGA), are:
  - — increased deformation resistance
  - — increased durability (high binder content, low voids, relatively impermeable)
  - — increased fatigue resistance
  - — improved crack retardation properties.
- Secondary properties are:
  - — skid resistance (based on texture) is between DGA and open graded asphalt (OGA)
  - — noise reduction properties are between DGA and OGA.

The work described in this paper examines some of the factors that may need to be addressed in achieving the properties described above.

METHOD

There are two SMA issues that are evaluated in this paper, namely the effect of the binder mastic and the development of a stone-on-stone skeleton. However, during the course of the study an issue regarding the measurement of the bulk density of samples arose which is briefly discussed.

Mastic properties

SMA has typically about 10% (by mass) of mineral aggregates finer than 75μm and has a binder content of more than 6% (by mass). The sum total of these constituents can have a substantial effect on the behaviour of the mix. In this study the flow properties are examined to determine how different fillers affect mix performance and how varying the binder to filler ratio can affect mix behaviour.

A rotational viscometer (Brookfield DV-II+ Pro) was used to determine the mastic properties. A temperature of 135 ˚C was used for the Class 320 bitumen samples and a temperature of 165 ˚C was used for the polymer modified binder.

Most of the testing was performed using spindle SC4-31, though for the very viscous samples the LV4 spindle was used. The spindle speed was adjusted to maintain a viable torque value during testing.

A small amount of testing was also conducted using the ARRB Elastometer (see Fig. 1). This device is designed to measure binder flow properties for polymer modified binders. Testing was conducted at 60 ˚C using mould B (see Fig. 2) in accordance with the Austroads (2006) test method (AG/PT:T121).

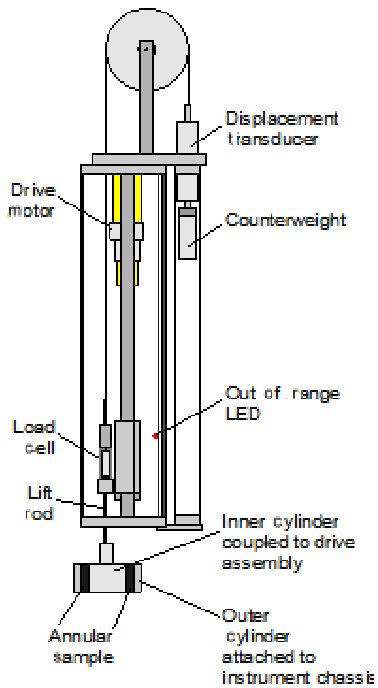


Figure 1: ARRB Elastometer

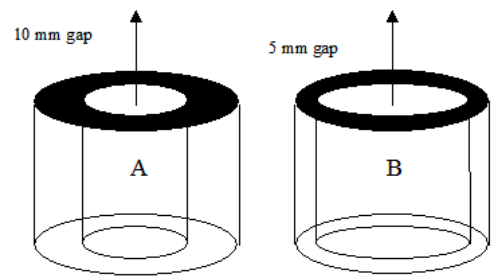


Figure 2: Commonly used ARRB Elastometer moulds

The study initially examined the effect the type of filler has on the mastic viscosity as the binder to filler ratio varied. This was supplemented by examining the mastic properties of some SMA mixes that had been used in real situations.

Development of a gradation

This part of the study focussed on examining the effects of introducing increasing proportions of fine aggregate into the gradation. The intention was to determine if it was possible to identify a point at which the coarse aggregate fraction no longer constitutes a stone-on-stone aggregate skeleton. The concept is often referred to as the dilation approach where the coarse aggregate needs to dilate in order to accommodate the increasing amount of fine aggregate (and mastic).

A ServoPac gyratory compactor was used to compact the SMA mixes. The Australian standard compaction method for 150 mm diameter samples was used and each sample was compacted for 120 gyratory cycles.

The bulk density was measured using the presaturation method, though in hindsight, this may not have been appropriate for all compacted samples. This is discussed further in the section dealing with the measurement of bulk density.

In addition, the compacted samples were subjected to the Australian standard resilient modulus test (AS 2891.13.1).

Three aggregates were used in this part of the study. The mix designs were based on 14 mm nominal size aggregate and were developed along the following principles:

- The mass of coarse aggregate was constant for all proportions of fine aggregate.
- Bitumen content was kept constant for each of the three mixes.
- The ratio of 14 mm aggregate to minus 5 mm crusher dust was varied.
- The passing 75 μm fraction was kept constant by adjusting the amount of added filler.

Measurement of the bulk density

Many of the SMA samples compacted during this study remained relatively open-structured due to the coarse gradations trialled. The bulk density of these samples was measured using a wax coating method (AS 2891.9.1), a presaturation dry method ((AS 2891.9.2) and a mensuration method (AS 2891.9.3) and from these the air void contents were determined.

RESULTS

Mastic properties

A class 320 bitumen was combined with a range of fillers. The binder to filler ratio was varied for each filler type and the viscosities measured using a rotational viscometer.

Six fillers were studied as shown in Table 1. The Rigden voids (voids in the compacted filler) for each filler were measured and the binder to filler ratio (by mass) was varied over the range one part binder to one part filler up to four parts binder to one part filler. Not all bitumen to filler ratios were tested for all fillers as some became extremely stiff and for others it was decided that once the mastic viscosity was below about 1.0 Pa.s there was little point in proceeding further. The information in shown in Table 1 and is shown graphically in Figure 3.

Table 1: Filler properties

Filler type	Ground limestone	Hydrated lime	Local stone dust #1	Local stone dust #2	Flyash	Silica-fume
Particle density (kg/m^2)	2705	2380	2702	2760	2210	2470
Rigden voids (%)	34.0	56.2	41.8	37.7	35.2	46.9
Ratio of bitumen to filler (by mass)	**Mastic viscosities (Pa.s)**					
1	2.21	71.19	2.87	3.00	2.53	5.00
2	1.00	2.53	1.16	1.16	1.20	1.30
3		1.34				0.97
4		1.08				0.84

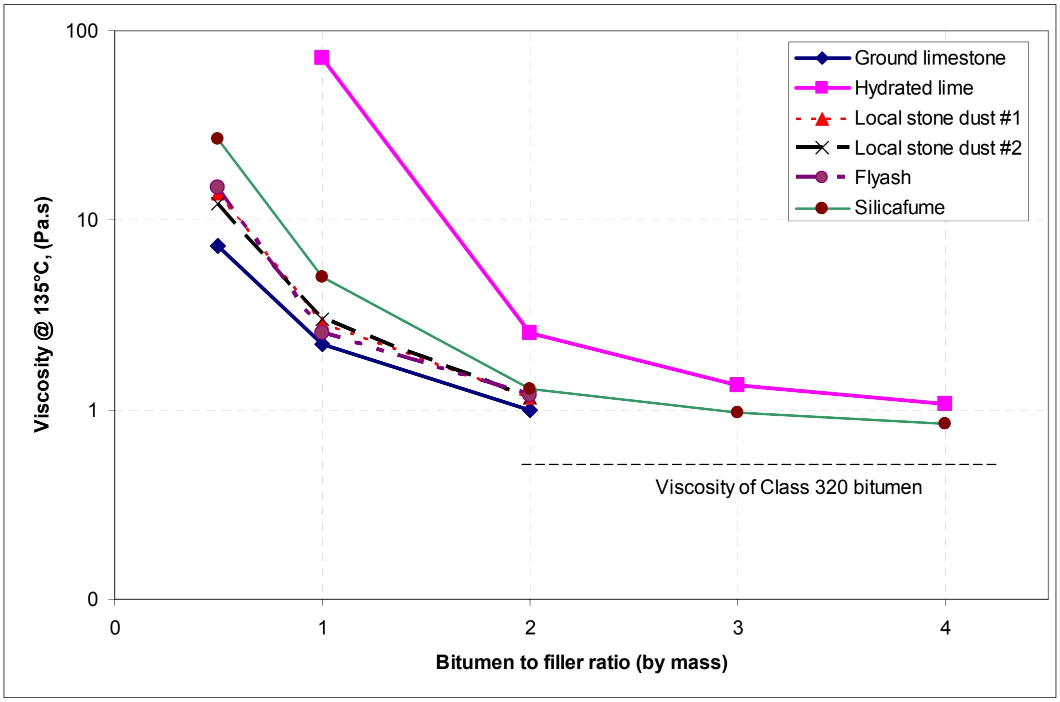


Figure 3: Viscosity of mastic for a range of filler types with varying bitumen to filler ratios

Silicafume was included as it is known to be relatively fine even though it is rarely used in asphalt these days due to its use as an additive in concrete. Hydrated lime is known to have high Rigden voids and yet is one of the least dense fillers. Hydrated lime can be compared to the flyash which also has a low density but also low Rigden voids. It would appear that there was no trend relating filler density to its packing characteristics.

In Figure 3 it can be seen that as the bitumen proportion in the mastic increases the viscosity decreases. All mastics are tending to asymptote to the viscosity of the Class 320 bitumen without any filler (0.5 Pa.s).

The terms free and fixed binder are used in Queensland (Bryant 2005) to describe the propensity of fillers to 'bind' a proportion of the bituminous binder with the remaining binder being 'free' to coat the aggregate. The ability of filler to 'fix' a proportion of the binder will influence the properties of the mastic within the SMA. Fillers which are able to fix a greater proportion of binder will result in mixes that appear drier and generally have a stiffer mastic.

Fillers that do not fix much binder can result in rich mastics with excessive free binder which may result in fatty mixes being placed on the road.

The proportion of binder required to fill the voids in the filler (referred to as fixed binder) can easily be calculated as shown below in equation 1.

$$f_{binder} = \frac{V_{voids}}{V_{binder}} = \frac{F}{B} \times \left(100 - \frac{B}{100}\right) \times \frac{\delta_{binder}}{\delta_{filler}} \times \left(\frac{V_{filler}}{1 - V_{filler}}\right) \qquad \text{Eqn. 1}$$

where

f_{binder} = fraction of fixed bitumen

V_{voids} = volume of voids in the compacted filler (i.e. Rigden voids)

V_{binder} = volume of binder in the mix

F = percentage by mass of filler in the combined aggregate fraction (%)

B = percentage by mass of binder in the mix (%)

δ_{binder} = binder density (t/m^3)

δ_{filler} = filler density (t/m^3)

V_{filler} = Rigden voids (voids in the dry compacted filler) (%)

Using the above formula the proportion of fixed binder was calculated for each filler type and for the various bitumen to filler ratios shown in Table 1. The results are shown in Table 2. A value of one equates to all the binder being used to fill the voids in the filler and this would mean the mix would be very dry and be unworkable.

Table 2: Proportion of binder fixed by the filler component

Binder to filler ratio	Ground limestone	Hydrated lime	Local stone dust #1	Local stone dust #2	Flyash	Silica-fume
0.5	0.37	1.04	0.51	0.42	0.47	0.69
1	0.18	0.52	0.26	0.21	0.24	0.34
2	0.09	0.26	0.13	0.11	0.12	0.17
3	0.06	0.17	0.08	0.07	0.08	0.11
4	0.05	0.13	0.06	0.05	0.06	0.09

The proportion of the fixed binder has been plotted against the mastic viscosities and the trend is shown in Figure 4. There was a correlation coefficient of 0.91 indicating a high probability that there is an interrelationship between the two properties.

Two SMA mixes from Queensland and an SMA mix from South Australia were examined. The Queensland SMA mixes were placed with a polymer modified binder that had a mastic that contained 46% hydrated lime: 54% stone dust and 59% hydrated lime: 41% stone dust for Qld1 and Qld2 respectively. The South Australian mix (SA1) was tested using a Class 320 bitumen and had a mastic with 80% baghouse fines and 20% stone dust.

All mixes were tested at 165 ˚C. Due to the stiff nature of the Queensland SMA mastics, the LV4 spindle was necessary and the filler proportions were reduced from those used in the examination of the effects of filler type.

The SC4-31 spindle was used in the testing of the South Australian mix.

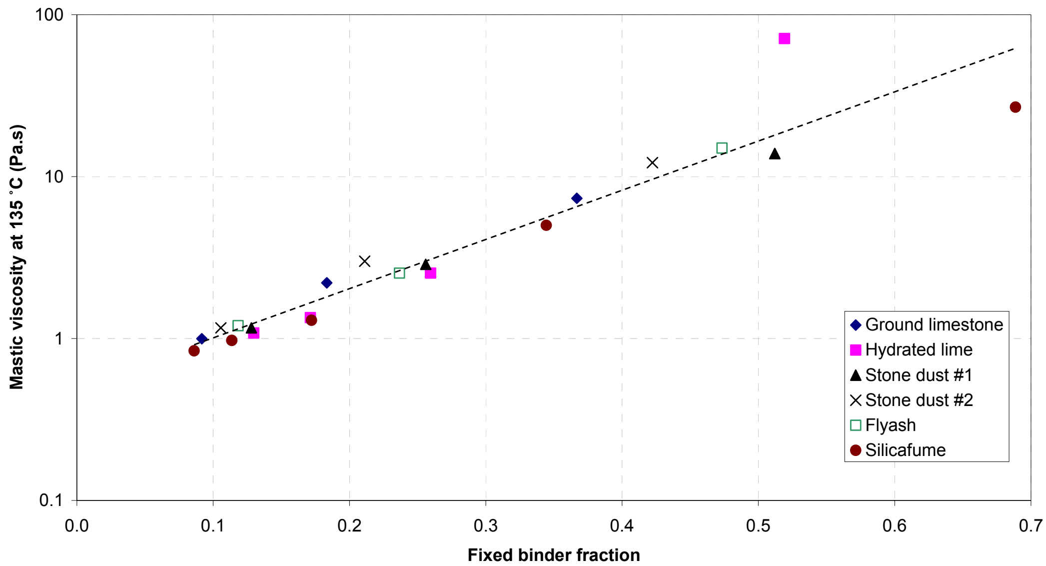


Figure 4: Mastic viscosities for a Class 320 bitumen combined with varying filler types and varying filler proportions

The data is shown in Figure 5 with the two Queensland mixes exhibiting similar performance (same base polymer modified binder). The South Australian SMA (SA1) had lower mastic viscosities due to the Class 320 bitumen used to manufacture the mastic.

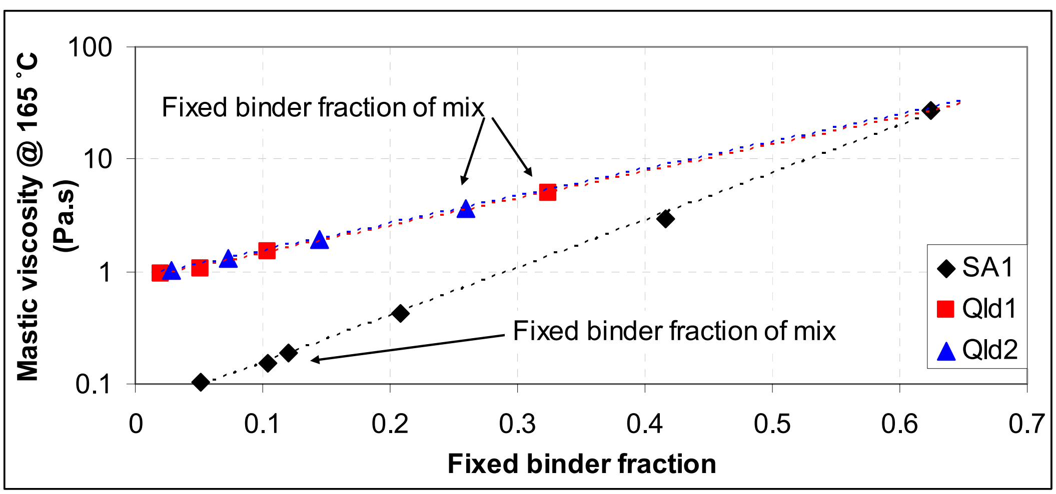


Figure 5: Mastic viscosities of three SMA mixes

The fixed binder fraction of the two Queensland mixes was calculated and by extrapolation of the data, the mastic viscosity was estimated as indicated in Figure 5. The estimated values would suggest that the mix would have been difficult to compact. The South Australian SMA had a lower mastic viscosity and a lower fixed binder fraction as shown in Figure 5.

The ARRB Elastometer data is shown in Table 3 rounded to three significant figures. The trends in the underlying viscosity data are shown in Figure 6 where the two Queensland SMAs have different performance. It was noted that the underlying viscosity and the consistency were similar for the SA1 mastic as expected for a straight grade bitumen.

Table 3: ARRB Elastometer data tested at 60 ˚C

Binder to filler ratio	SA1 mastic		Qld1 mastic		Qld2 mastic	
	Consistency (Pa.s)	Underlying viscosity (Pa.s)	Consistency (Pa.s)	Underlying viscosity (Pa.s)	Consistency (Pa.s)	Underlying viscosity (Pa.s)
0.25	Too soft	Too soft	22000	2020	15700	1310
0.50	460	444	43900	6950	25000	3060
1.00	792	751	134000	68300	80500	14200

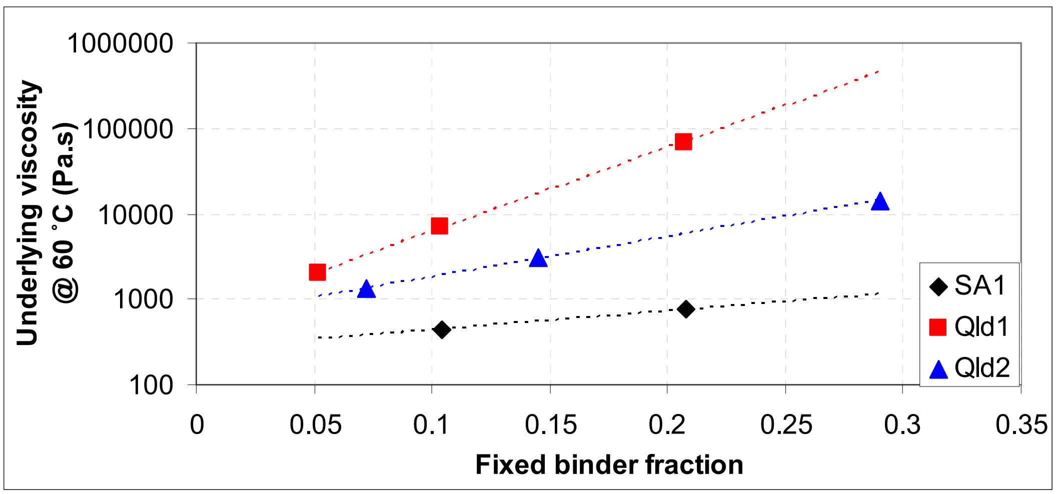


Figure 6: Variation in underlying viscosity with changes in the binder to filler ratio

Aggregate gradation

Asphalt mixes were prepared using three aggregate sources with varying proportions of fine aggregate (crusher dust with a maximum particle size of 5 mm). The mass of coarse aggregate was maintained at a constant value regardless of the amount of fine aggregate added. The mix compositions are presented in Appendix A (see Tables A1 to A3).

The compacted heights were measured using a linear variable displacement transducer fitted to the ServoPac. The average compacted heights of duplicate samples are shown in Table 4. The intention was to have three results either side of the dilation point.

Table 4: Compacted heights (mm)

Fine aggregate proportion (%)	Aggregate 1	Aggregate 2	Aggregate 3
5	Not required	Not required	78.6
10	70.4	72.0	81.0
15	69.0	71.8	80.8
20	69.9	73.8	83.8
25	71.3	72.3	88.0
30	73.3	75.3	93.0
35	76.1	79.8	Not required

The results are presented in Figure 7 which plots the change in height versus the proportion of fine aggregate added for the three aggregates.

The mixes with a low proportion of fine aggregates, i.e. those mixes in which the aggregate skeleton was not dilated, were expected to have the same height as there were sufficient voids between the coarse aggregate particles for all the fine aggregate particles. However, even though there may be sufficient voids in the coarse aggregate skeleton, not all the fine particles will find their way into the voids and some will dilate the coarse aggregate skeleton. As more fine aggregate is added, this tendency to dilate the coarse aggregate skeleton, even though sufficient voids exist, is exacerbated. The result is that the data trend is not exactly linear but has a slight curvature as the dilation point is approached. This can be seen in Figure 7 where the pre-dilation trend is only approximate.

After all the voids are filled, any addition of further material to the mould will result in a change in height. The mixes with a high proportion of fine aggregate, i.e. those with a dilated coarse aggregate skeleton, exhibit a linear change in height against the proportions of fine aggregate. For all three aggregates, this trend is very clear.

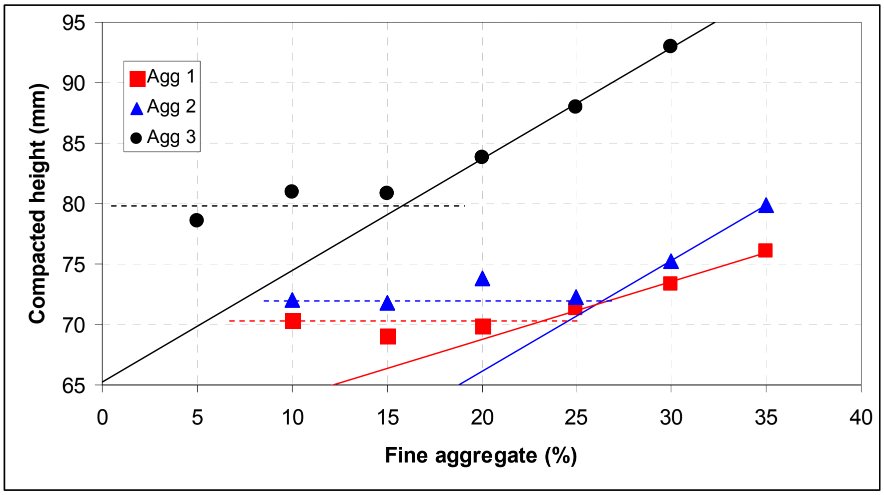


Figure 7: Dilation concept based on the compacted height for three aggregate types

The lines on the plot indicate the following:

- dashed lines – the initial compaction trend before the coarse aggregate structure was dilated
- solid lines – the compaction trend after the coarse aggregate structure had been dilated.

The dilation point is defined as the intersection of these two lines. For Aggregate 1, the dilation point occurred when the fine aggregate proportion was about 23%. Similarly the dilation points were 26% and 16% for Aggregate 2 and Aggregate 3 respectively.

The compacted 150 mm diameter samples were cored to yield 100 mm diameter specimens and these were trimmed to a height of 50 mm. These samples were then subjected to the resilient modulus test and the results are shown in Table 5. The values in Table 5 have been rounded to three significant figures.

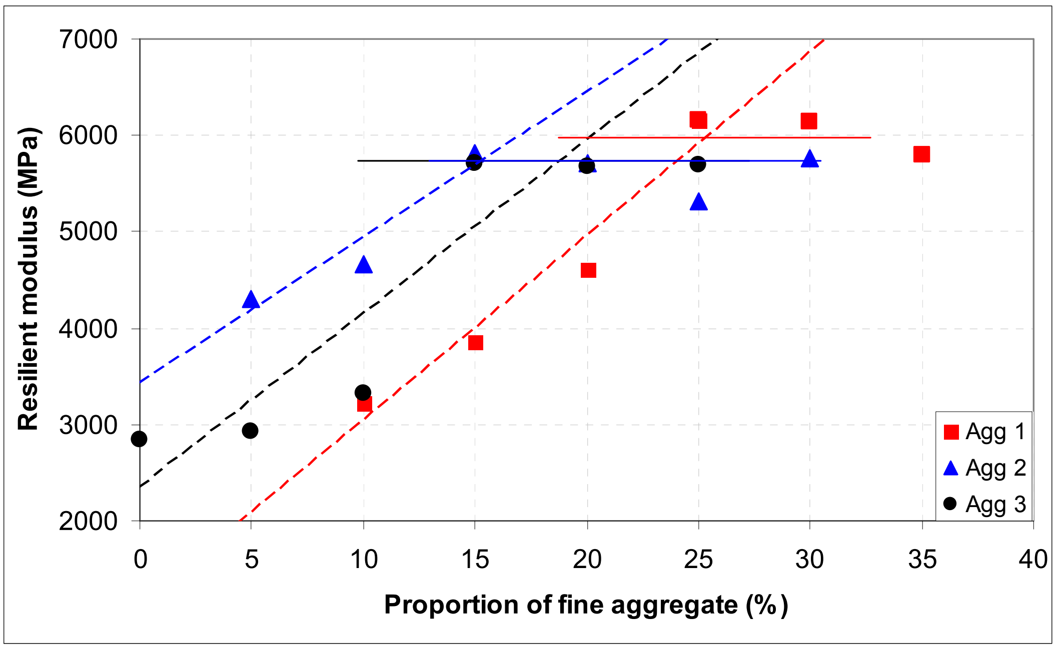
Table 5: Resilient modulus (MPa)

Fine aggregate proportion (%)	Aggregate 1	Aggregate 2	Aggregate 3
5	Not required	Not required	2840
10	3230	4310	2920
15	3860	4670	3320
20	4620	5820	5710
25	6160	5710	5670
30	6140	5310	5700
35	5800	5760	Not required

The addition of fine aggregate into the sample would initially provide increased resistance to movement in the coarse aggregate skeleton and this should result in an increase in the resilient modulus. This was found to be the case with relatively large increases in resilient moduli as the proportion of fine aggregate was increased.

The resilient moduli after the coarse aggregate skeleton had been dilated was expected to be more or less constant and would be a function of the aggregate type (shape, angularity, surface texture, etc.) and this was found to be a reasonable approximation of the trends in the data.

The same coding as was provided in Figure 7 (the compacted height plot) has been used in Figure 8. The initial modulus trend is presented as a dashed line and the dilated aggregate moduli trend presented as a solid line.



Source: ARRB (2009)

Figure 8: Dilation concept based on variation in modulus using three aggregate types

The dilation points for the compacted height data and resilient modulus data are shown in Table 6. Aggregate 1 and Aggregate 3 show reasonable agreement but Aggregate 2 had a different dilation point depending upon the data set used.

Table 6: Comparison of dilation points (proportion of fine aggregate)

Material	Aggregate 1	Aggregate 2	Aggregate 3
Compacted height dilation point (Percentage of fine aggregate)	23	26	16
Moduli dilation point (Percentage of fine aggregate)	25	15	18

Measurement of the bulk density

SMA mixes are coarsely graded when compared to dense-graded mixes. Coarse gradings result in fewer but larger interconnected voids within the compacted asphalt. The larger voids can fill with moisture and empty much more quickly than small voids. This has implications when it comes to measuring bulk density of compacted asphalt specimens.

A group of samples were compacted so as to achieve a variation in air void content. The bulk densities were measured using three Australian standard methods and the air voids calculated. The results are shown in table 7 and in Figure 9.

Table 7: Air void contents (%)

Wax coating method (AS 2891.9.1)	Presaturation method (AS 2891.9.2)	Mensuration method (AS 2891.9.3)
1.7	1.9	6.5
2.9	2.6	6.9
3.8	3.6	9.6
7.5	7.5	14.2
7. 9	8.4	17.1
8.6	9.6	18.4
11.3	10.1	24.2
12.3	10.3	24.6
15.9	7.2	27.6

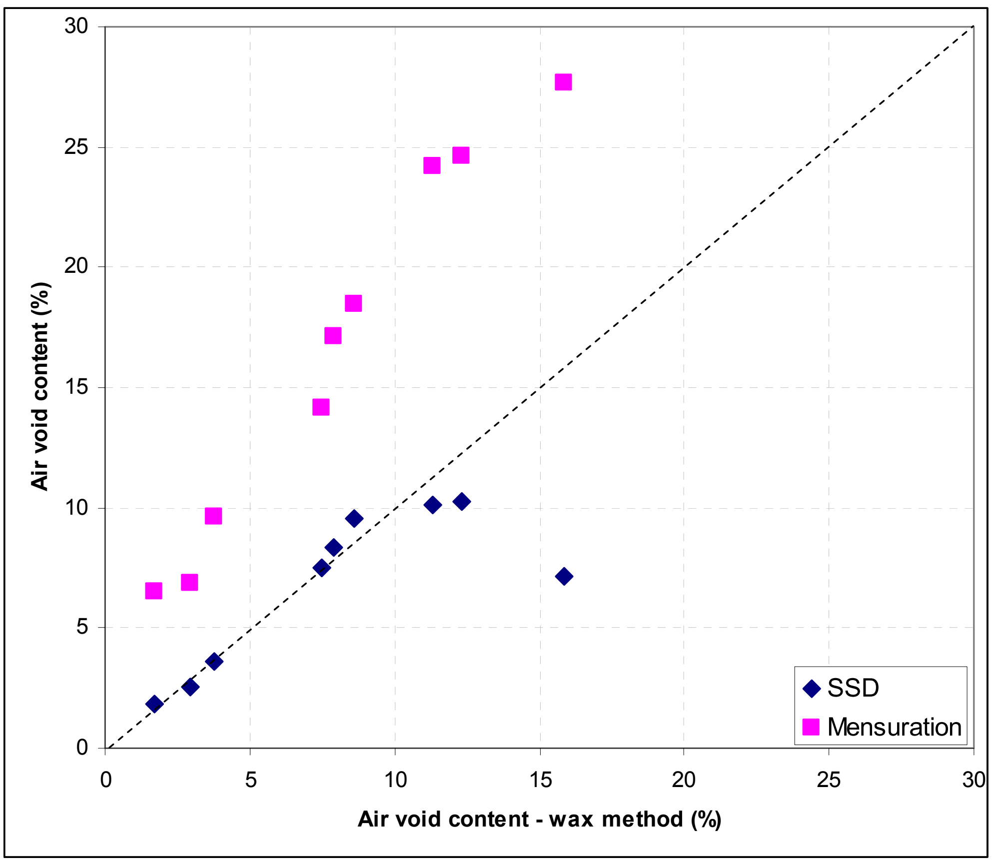


Figure 9: Correspondence between bulk density test methods

It can be clearly seen that there is a good correlation between the air voids measured by the waxing method and the pre-saturated (SSD) method up to about 8% voids. Beyond this level the pre-saturated bulk density method becomes less accurate.

This is believed to be due to the increasing inter-connectivity of the voids within the sample. Water is freely able to enter the internal voids spaces and exit again once the sample is removed from the water bath. Therefore the pre-saturated method discounts the free draining internal voids. The waxing method seals over these voids at or near the surface and the wax does not penetrate into the internal voids. This means the internal voids are accounted for in the measurement of the bulk density.

The mensuration method is clearly not accurate when used with coarse textured samples. Measuring the sample dimensions inevitably includes the surface texture voids of the samples as part of the measured sample volume.

The work has demonstrated that the method of measuring the bulk density has a significant effect on the calculated air void content. The true air void content of the sample would exclude surface texture effects but include all internal voids. Test methods that provide a form of 'skin' over the surface of the sample would best meet these criteria. The waxing method is generally not favoured as it is difficult to remove the wax if the sample is required for other tests.

Vacuum sealing samples in plastic bags could meet these criteria. The plastic would need to be clearly defined to ensure that the same level of correspondence to the surface texture was maintained for all testing and surface texture conditions.

CONCLUSIONS

A strong interdependence was noted between the viscosity of the binder mastic and the calculated fixed binder proportion. However, the viscosity of the mastic was found to be greatly dependent upon the base binder used. Mastics with polymer modified binders as the bituminous base were more viscous than conventional bitumens. This indicated that viscosity testing of SMA mastics needs to be done for each mix and that the fixed binder fraction is only of value in evaluating the effects of alternative fillers.

The ARRB elastometer was able to distinguish a difference in performance between the three mixes studied. In-service performance data will need to be gathered to determine if the ARRB Elastometer data correctly reflects the difference in performance found in this laboratory study.

No limits of performance for the mastic viscosity were able to be established from the data examined here. Additional SMA mixes need to be studied, particularly those mixes that have poor in-service performance.

The dilation concept was examined based on the compacted height and on the resilient modulus. Both data sets were able to estimate the point at which the coarse aggregate skeleton dilated though in the case of one mix, the dilation point differed in the two sets of data.

Coarse aggregate mixes were developed during this study and these highlighted the need for careful consideration as to which bulk density test provided a reliable and accurate result. It was found that when the actual voids exceeded about 8% voids the presaturation bulk density test became inaccurate.

The sample volume determined from the sample dimensions proved to be unreliable in determining the bulk density. This was due to the inclusion of the volume in the surface texture in the overall sample volume. It is recommended that a simpler alternative to the waxing bulk density test method be considered, such as vacuum sealing the sample in a plastic bag.

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APPENDIX A – MIX COMPOSITIONS

Table A1: Aggregate 1

Mix Specification						
Aggregate	**Mix 1**	**Mix 2**	**Mix 3**	**Mix 4**	**Mix 5**	**Mix 6**
14 mm	75.8	71.1	66.1	61.1	56.1	80.5
5- Dust	15.0	20.0	25.0	30.0	35.0	10.0
Added filler	9.2	8.9	8.9	8.9	8.9	8.9
Bitumen	6.0	6.0	6.0	6.0	6.0	6.0

Table A2: Aggregate 2

Mix Specification					
Aggregate	**Mix 1**	**Mix 2**	**Mix 3**	**Mix 4**	**Mix 5**
14 mm	80.0	75.0	70.6	66.0	61.0
5- Dust	10.0	15.0	20.0	25.0	30.0
Added filler	10.0	10.0	9.4	9.0	9.0
Bitumen	6.3	6.3	6.3	6.3	6.3

Table A3: Aggregate 3

Mix Specification						
Aggregate	**Mix 1**	**Mix 2**	**Mix 3**	**Mix 4**	**Mix 5**	**Mix 6**
14 mm	75.7	71.0	84.5	80.0	65.7	60.7
5- Dust	15.0	20.0	5.0	10.0	25.0	30.0
Added filler	9.0	9.3	9.3	10.0	9.3	9.3
Bitumen	6.3	6.3	6.3	6.3	6.3	6.3