The contribution of asphalt mastic to shear resistance

Greg White

Fulton Hogan, Brisbane, Australia

greg.white@fultonhogan.com.au

Digital Object Identifier (DOI): dx.doi.org/10.14311/EE.2016.305

ABSTRACT

An Australian airport was to be resurfaced by asphalt overlay. The airport had experienced variable performance from similarly designed asphalt overlays in the past. Previous investigations had established that changes in the asphalt mastic had likely led to a lack of asphalt shear stress resistance.

Various sources of fine aggregate and binder had been used in previous resurfacing works. Mastic combinations containing two specific fine aggregates sources and two specific binder sources resulted in variable asphalt performance. The two fine aggregates were similar except that one contained predominantly Hisingerite clay minerals, which had potentially detrimental properties. The two binder sources were the same grade of acid-modified bitumen manufactured from different crude oil source blends with significantly different properties.

A combination of repeated shear stress testing of mastic and bitumen determined that both the fine aggregate and binder sources significantly affected the mastic response to shear stress, and therefore expected asphalt shear resistance. No adverse impact associated with the Hisingerite-rich fine aggregate was found. The two sources of bitumen showed significantly different properties.

It was concluded that apparently similar fine aggregate and binder sources could have significant impact on mastic, and therefore asphalt, performance in high shear stress states during hot temperatures. It is recommended that the current Australian specifications for airport asphalt should be reviewed to prevent significantly different surface performance from nominally identical and compliant mastic constituents.

Keywords: Creep, Heavy-duty pavements

1. INTRODUCTION

An Australian airport was to be resurfaced by asphalt overlay. The planned construction operation included milling of the existing surface with a cold planing machine, cleaning and tack coating with bitumen emulsion. The overlay was designed to be generally 50-60 mm thick. The asphalt mixture was specified as an airport-quality nominal 14 mm maximum aggregate size and contained acid modified bitumen, known locally as M1000.

The airport had experienced variable performance from similarly designed asphalt overlays in the past. Approximately six months after the completion of previous surfacing works, a number of localised horizontal surface deformations were identified (Figure 1). Through previous investigations it had been established that changes in the asphalt mastic had likely led to a lack of shear stress resistance. A common source and grading of coarse aggregate had been used in both poor and adequately performing surfaces. The deformations were triggered by shear stresses induced by heavy braking commercial aircraft such as the B737 and B767.

Figure 1: Example of typical horizontal shear creep failure

Various sources of fine aggregate and binder had been used in previous resurfacing works. In some cases, the material sources used could not be traced to specific locations (of differing performance) on the airport. Two fine aggregates (Dust A and Dust B) and two binder sources (Bitumen 1 and Bitumen 2) were confounded and their relative contribution to asphalt shear resistance could not be determined from correlation with observed field performance. Two specific asphalt mastic combinations had commonly been used (Table 1) within the consistent coarse aggregate skeleton (Appendix 1).

Table 1: Two common Mastic Combinations

Asphalt Mixture	Fine Aggregate Source	Binder Source
Asphalt A/1	Dust A	Bitumen 1
Asphalt B/2	Dust B	Bitumen 2

The aim of this investigation was the measure the shear resistance of the asphalt mastics and to assess the impact of the fine aggregate and binder source on the measured differences. Mastic was evaluated as a proxy for the shear deformation resistance of the asphalt surfaces expected within the common coarse aggregate skeleton. Conclusions address the impact of minor mastic constituent changes on the expected performance of asphalt in high shear states as well as the contribution of the fine aggregate and binder to predicted asphalt shear creep resistance.

2. BACKGROUND

Asphalt is a complex heterogeneous material consisting of aggregates, air voids and binder [1]. Asphalt is used around the world in many applications including the surfacing of airport pavements [2]. The time and temperature dependence of bituminous binder properties has a significant impact on the response of asphalt to loading [3]. At the extremes, asphalt is purely elastic at low temperatures and fast loading, and a viscous liquid at high temperatures and slow loading [4]. At typical service temperatures and traffic loading rates, bitumen responds to shear stress in a complex visco-elastic manner.

2.1 Asphalt Shear Resistance

Permanent vertical asphalt deformation (rutting) was previously considered to be caused by viscous flow of the asphalt binder/mastic [5]. Asphalt rutting is correctly defined as the cumulate permanent deformation of the asphalt layer(s) through incremental densification under loading [6]. True vertical deformation in asphalt rarely occurs and is often confused with what are actually shear failures, characterised by heaving or slip-circle type deformation. True rutting is free of heaving at the extremities. Recent research has shown that most asphalt permanent deformation is the result of shear creep rather than viscous flow [5].

In high shear stress conditions, asphalt can permanently deform horizontally, without any vertical deformation [7]. Runways experience high horizontal shear forces during aircraft braking [8] where shear stresses can be up to 68% of the vertical stress [9]. A number of instances of horizontal asphalt surface shearing failures have occurred at airports [10-12]. The temperature dependent nature of bitumen increases the risk of such failures at elevated pavement temperatures, when the binder is less creep resistant.

2.2 The Importance of Mastic

Mastic is the 'real' binder in an asphalt mixture [13]. Tashman et al. [14] supported this by stating that the micro-constituents governed behaviour of the overall mixture. It follows that testing mastic provides greater insight into asphalt performance than testing binder [2]. Mastic has greater impact on the performance of asphalt mixtures with dense grading and high binder content, than the performance of stone-to-stone mixes, such as stone mastic and porous asphalt [15]. Airport asphalts are commonly specified to be dense graded with high binder content and rely heavily on mastic performance to resist stresses and deformation [16]. It follows that airport asphalt mixtures are highly affected by the properties of the mastic and that mastic performance provides a proxy for asphalt mixture performance.

Despite the recognised importance of mastic for asphalt performance, less is known about mastic properties than those of bitumen [17]. However, it is acknowledged that mastics of seemingly similar constituents can behave differently [18]. This can only be explained by physio-chemical interaction between the bitumen and mineral elements. Such interactions cannot be assessed by considering the bitumen and mineral components separately.

The ratio of mineral content (fine aggregate and active filled) to binder in mastic is critical to mastic characterisation. As the mineral portion increases, the mastic stiffens [19-20]. Liao et al. [17] found that the mineral portion had a greater impact on mastic response than aggregate and active filler properties. When testing asphalt mastic, accurate replication of the binder:filler:aggregate ratio in the asphalt mixture is critical.

2.3 Factors affecting mastic performance

As detailed above, mastic performance is affected by the relative proportion of the constituent materials. Voids in the active filler, bitumen properties and fine aggregate properties can also impact mastic, and therefore asphalt, response to stress. Filler may absorb binder into its voids, reducing the volume of 'effective' binder available to bind the aggregate together. Different filler types and sources have different chemical compositions, as well as different shapes, densities and voids, so can reduce the effective binder content differently [21].

The temperatures, pressures, blowing and other processes performed in producing paving-grade bitumen impact the bitumen properties [22]. As does the crude oil source [23]. The different visco-elastic properties of the resulting bitumen directly impact the properties of the mastic and subsequently the asphalt mixture [24].

Fine aggregate properties contribute more to asphalt performance than the large aggregate does. Fine aggregate shape and packing properties impact significantly on asphalt deformation resistance [25]. Aggregate containing significant levels of clay (< 2 µm in diameter) that exhibit significant plasticity are potentially deleterious and should either be avoided or treated with active filler such as hydrated lime [26].

2.4 Multiple Stress Creep Recovery

The USA introduced the Performance Grading (PG) system for grading of bitumens in the 1990s [27]. The Multiple Stress Creep Recovery (MSCR) protocol subsequently replaced the complex modulus as the PG high temperature criterion in 2010. This repeated shear test was developed to be blind to modification and site location and to assess binder response to shear in both the linear and non-linear stress ranges. MSCR has shown better correlation to full scale and field deformation of asphalt mixtures [28]. MSCR represents best practice for performance-based assessment of paving grade bituminous binders.

MSCR has been demonstrated to be easy to perform in the laboratory using modern Dynamic Shear Rheometer (DSR) equipment [27] and takes only around 15 minutes to complete [29]. Six parameters are calculated from the MSCR

protocol [28]. Of these the cumulative unrecovered strain over ten high (3.2 kPa) stress level cycles is the primary PG criterion. This is termed the creep compliance or Jnr(3.2) and for a specific asphalt mixture design, is indicative of shear stress deformation. Other parameters are the Average Recovery (AR) at 0.1 kPa and 3.2 kPa stress levels as well as the Jnr at 0.1 kPa.

It is normal to artificially age samples using the Rolling Thin Film Oven (RTFO) prior to MSCR testing. RTFO conditioning is intended to simulate bitumen ageing during asphalt production [30]. Most researchers have performed MSCR after RTFO conditioning [31-35].

Some MSCR research has been performed on mastic [36]. It has been demonstrated that for mastics containing particles less than 250 μƐ in diameter, the standard 1 mm gap between the parallel is adequate [37]. For samples containing larger particles, a cup-and-bob or plate-and-cone arrangement was preferred. It follows that for mastic samples manufactured from fine aggregate passing the 75 μƐ, no change to the standard arrangement is warranted.

3. RESEARCH METHODS

It will be shown below that the two dusts were similar in origin and rock type but contained two different predominant clay minerals. The two binders were the same grade of acid modified bitumen procured from two difference sources and with different properties. Performance-based testing of mastics manufactured from combinations of fine aggregate and binder was assessed to separate the potential contribution of each to mastic shear deformation.

Three binder samples from each source (Bitumen 1 and Bitumen 2) were used to make two mastic samples, one with each fine aggregate (Dust A and Dust B). Mastic samples were subject to MSCR testing at 64°C, 70°C and 76°C. Neat binder samples were also subject to MSCR evaluation at the same temperatures. MSCR is a relatively new test and repeatability data is not available for bituminous mastic. Therefore, multiple replicate samples were tested and statistical analysis performed. Both the binder and fine aggregate sources were subject to detailed characterisation.

Mastic samples were manufactured in the laboratory from the retained binder and representative dusts. The common hydrated lime (active filler) was added to all mastic samples. First, the hydrated lime was mixed into the heated bitumen. The fine aggregate was then incorporated. All samples were manufactured to a common 6:1:7 (binder:filler:aggregate) mass ratio. This ratio was selected to replicate the average mastic composition within the two asphalt mixture designs.

4. RESULTS AND ANALYSIS

4.1 Materials

Both fine aggregates were from basalt quarries and were shown to contain non-plastic fines. Petrographic analysis of the two materials found both to be olivine basalt of hard, grey, robust particles of slight to moderate weathering (Table 2). Dust B was slightly more weathered than the Dust A as indicated by the higher Smectite (clay) and accessory mineral contents. Both fine aggregates contained secondary minerals in and around cracks in the olivine structure. Secondary minerals in both dusts were initially identified as Nontronite (clay).

Table 2: Summary of fine aggregate petrographic report

Item / Mineral Content	Dust A	Dust B
Rock type	Olivine basalt	Olivine basalt
Apparent density (t/m^3)	2.89	2.79
Absorptivity (%)	2.0%	2.5%
Methyl Blue Value (%)	4	8
Plagioclase	71%	59%
Magnetite	12%	4%
Olivine	4%	5%
Augite	4%	13%
Smectite Group	8%	13%
Glass and Accessory minerals	1%	6%

Initial X-Ray Diffraction (XRD) assessment (oven dried) of the chemical composition of the fine aggregates agreed with the petrography, indicating the two dust sources contained comparable amounts of clay minerals within an otherwise typical olivine basalt. Further XRD analysis was performed glycolated (held in a desiccator overnight at 30°C in ethylene glycol vapour) and heat treated (550°C for several hours before cooling). This indicated the

observed brown chips within Dust B determined that what the initial petrographic assessment reported as Nontronite clay was in fact the rare clay mineral called Hisingerite. Negligible Hisingerite existed in Dust A (Table 3). The higher percentage of clay minerals in Dust B explained the higher absorption of methylene blue solution (Table 2).

Table 3: Fine aggregate Hisingerite content

Dust Source	Dust A	Dust B
Percentage of Clay (< 2 µm) Minerals in Dust	8%	13%
Percentage of Clay (< 2 µm) that was Hisingerite	<1%	82%
Percentage of Hisingerite in Dust	Negligible	10.7%

Bitumen samples were assigned a two-digit code, the first indicating the feedstock and the second the sample number. All bitumen samples complied with the Australian specification for paving grade bitumen (Table 4). The primary specification criteria for Australian acid-modified paving grade bitumen are the viscosity at 60°C after conditioning with RTFO and penetration at 25°C after RTFO. The same parameters, measured before RTFO conditioning, are reported for information purposes.

Table 4: Binder Sample Properties

Bitumen Sample	RTFO Vis. 60°C	Vis. 60°C	RTFO Pen. 25°C	Pen. 25°C	Vis. 135°C
1.1	4,740	1,085	31	46	1.138
1.2	5,860	1,057	32	48	1.106
1.3	4,077	987	31	45	1.036
2.1	6,221	1,260	35	48	1.199
2.2	6,274	1,229	36	47	1.170
2.3	6,388	1,249	38	46	1.211
Specification Limit	4,000-6,500	Report only	> 26	Report only	< 1.500

4.2 Results

The MSCR results for RTFO conditioned bitumen are contained in Appendix 2. The corresponding results for the RTFO conditioned mastic MSCR are in Appendix 3. Mastic samples are referred to by a three character alpha-numeric code. The first two characters indicate the bitumen sample and the third indicates the dust source. For example, mastic sample 2.1.B was manufactured from bitumen source 2, retained sample 2.1 (Table 4) and Dust B (Table 2 and Table 3).

4.3 Analysis

4.3.1 Binder and Fine Aggregate

Binder samples from Bitumen 2 were significantly different to those from Bitumen 1 (Table 5). The pre- and post-RTFO viscosities were higher, indicating a 'harder' binder. Unusually, the Bitumen 1 penetrations after RTFO were also higher, which would normally indicate a 'softer' binder. Pre-RTFO penetrations were not significantly different. The differences in viscosity and penetration trends indicated binders of significantly different rheology and origin.

Table 5: Binder compliance testing summary statistics

Statistics	RTFO Vis. 60°C		Vis. 60°C		RTFO Pen. 25°C		Pen. 25°C	
	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2
Average	4969	6294	1022	1246	32	36	47	47
Std Dev	1261	85	49	16	1	2	2	1
CV	25%	1%	5%	1%	2%	4%	5%	2%
p-values	0.06		< 0.01		0.56		0.04	

Dust A contained predominantly Nontronite clay minerals, while Dust B contained mainly Hisingerite (Table 3). Hisingerite is a rarely encountered and poorly studied clay mineral. Specialist geotechnical interpretation of its unique properties indicated potentially adverse impact on mastic stability and asphalt shear response. The two fine aggregates were otherwise not significantly different.

4.3.2 Mixture Design

The previous commonly used asphalt mixture designs were reviewed. The two asphalt mixtures were similar except one contained Dust A and Bitumen 1 (Asphalt A/1) while the other contained Dust B and Bitumen 2 (Asphalt B/2). Both used the same course aggregate source, the same grade of binder and the same hydrated lime filler. Both targeted the same volumetric composition. It followed that the two mixtures had similar Marshall properties measured during mix design (Appendix 1). The Asphalt B/2 design returned 14% higher Marshall Stability and a 6% lower Marshall Flow. The difference in air voids after Marshall compaction was negligible.

The additional mix design properties were also comparable for the common asphalt mixtures (Appendix 1). Asphalt B/2 had a 21% lower resilient modulus but 6% higher tensile strength. Asphalt B/2 also showed 8% lower rut depth after wheel tracking and an almost identical tensile strength ratio to that of Asphalt A/1. Overall the mix design properties, volumetrics and additional mix design testing did not indicate any significant difference in mastic or asphalt performance resulting from the two different mastic compositions.

4.3.3 Mastic and Binder Response to Shear

The shear strains measured during the MSCR testing of neat binder and mastic samples are illustrated in Figure 2 for 70°C test temperature. Figure 3 shows the same data with the neat binder samples removed and the axis scales modified to focus on the 3.2 kPa stress level cycles for the mastic samples only. Trends were consistent at 64°C and 76°C, with the magnitude of the strains increasing with higher temperature as expected.

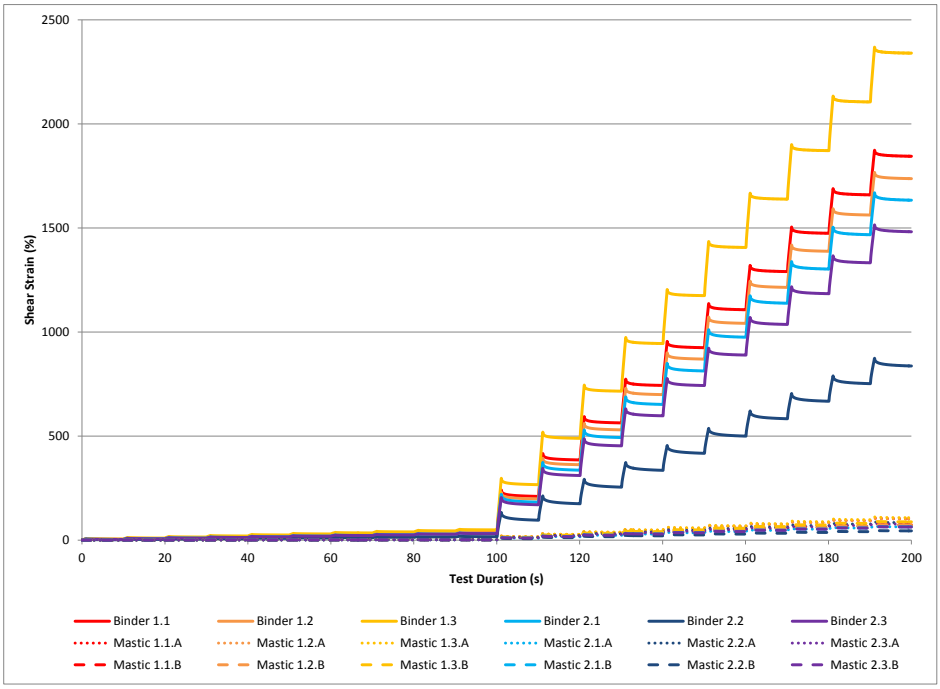

Figure 2: Binder and Mastic MSCR Strains at 70°C during 0.1 and 3.2 kPa shear stress cycles

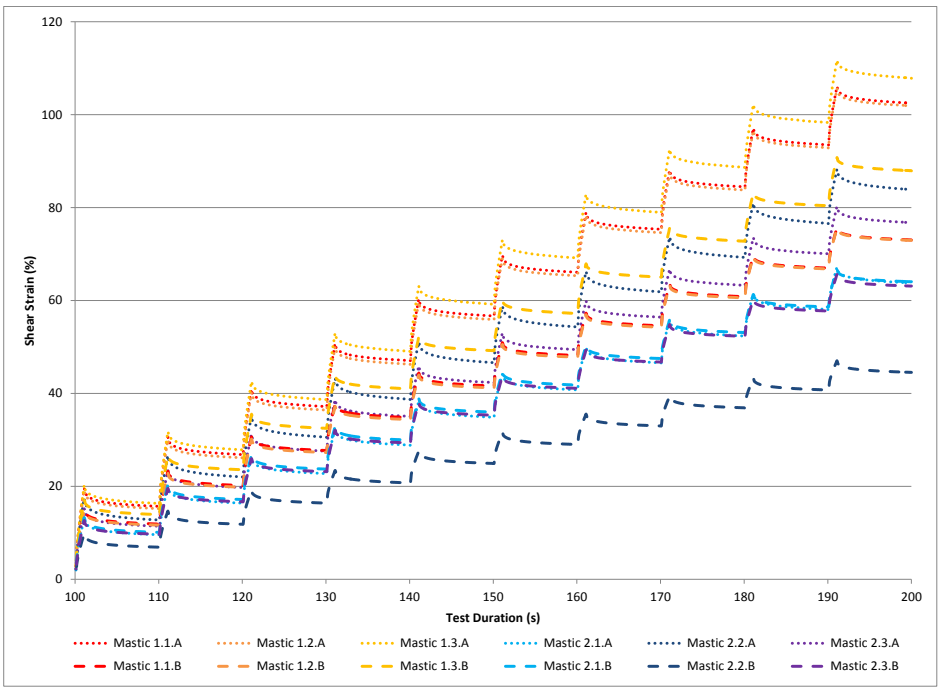

Figure 3: Mastic MSCR Strains at 70°C during 3.2 kPa shear stress cycles

The mastic accumulated strain was consistently a full order of magnitude lower than for the bitumen samples (Figure 2). The significant impact of the applied shear stress level was also evident. The response to shear stress was consistent across all test temperatures (Figure 4). All binder and mastic samples showed similar increases in Jnr(3.2) with increasing temperature. The range of Jnr(3.2) across mastic samples increased at higher test temperature. Mastic Jnr(3.2) values were two orders of magnitude smaller than typical bitumen Jnr(3.2) values. At 70°C and 76°C the order of Jnr(3.2) for mastic samples mirrored the order of the binder sample Jnr(3.2) values. However, at 64°C, some mastic sample Jnr(3.2) values were ordinally different to the bitumen samples. This is demonstrated by the crossing of lines between 64°C and 70°C for some mastic samples. This reflects the small strains experienced by mastic at lower temperatures in comparison to the accuracy of the measuring equipment. Expected errors in measurement became significant when the total strains were relatively small (lower temperature testing of mastic).

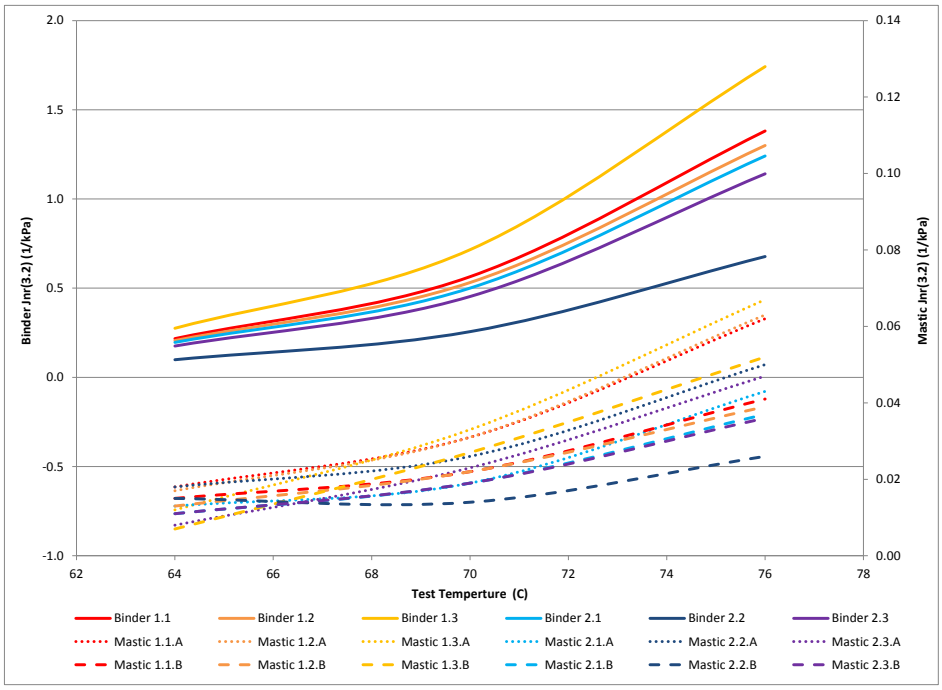

Figure 4: Binder and Mastic MSCR Jnr(3.2) at all test temperatures

4.3.4 Effect of Fine Aggregate and Binder on Mastic

The effect of the two different dust sources on each of the bitumen samples is illustrated in Figure 3. The mastic samples containing Dust A consistently deformed more under cyclic shear stress than Dust B samples did. This was also reflected in Jnr(3.2) values. Summary statistics for mastic Jnr(3.2) values and associated p-values for t-tests of differences in paired means are presented in Table 6.

Table 6: Fine aggregate source impact on Mastic Jnr(3.2)

Statistics	At 64°C		At 70°C		At 76°C	
	Dust A	Dust B	Dust A	Dust B	Dust A	Dust B
Average	0.014	0.012	0.027	0.021	0.055	0.039
Standard Deviation	0.004	0.003	0.005	0.004	0.009	0.008
CV	25%	22%	19%	21%	17%	22%
p-value (paired t-test)	0.10		0.01		<0.01	

Although marginal at 64°C, at both 70°C and 76°C the Dust A mastic samples had significantly higher Jnr(3.2). This indicated that Dust B mastic, and therefore asphalt containing Dust B fine aggregate, would deform less under shear stress. This result implied that Hisingerite in the fine aggregate was advantageous to asphalt shear resistance. It is more likely, in fact, that the lower mastic deformation in Dust B samples reflected the lower apparent density and higher absorptivity of Dust B (Table 3) as described further below.

The impact of the binder source on mastic shear response was also significant. This is illustrated in Figure 3 and reinforced by the p-values from t-tests for average Jnr(3.2) between Bitumen 1 and Bitumen 2 (Table 7). At 70°C and 76°C test temperatures, mastic samples containing Bitumen 2 had significantly lower Jnr(3.2) values than the Bitumen 1 mastic samples. The lack of significance at 64°C likely reflects the complications associated with small strains (compared to measurement accuracy) and the resulting non-ordinal agreement with binder samples as described above.

Table 7: Binder source impact on Mastic Jnr(3.2)

Statistics	At 64°C		At 70°C		At 76°C	
	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2
Average	0.014	0.013	0.028	0.020	0.054	0.040
Standard Deviation	0.004	0.004	0.005	0.004	0.012	0.009
CV	29%	28%	17%	20%	22%	22%
p-value (paired t-test)	0.65		0.01		0.04	

Both the binder and fine aggregate sources had a significant effect on the measured Jnr(3.2) values. Similar analysis for Jnr(0.1), AR(0.1) and AR(3.2) found different trends. For these other MSCR parameters, neither the binder nor the fine aggregate source had a consistently significant impact. This indicated that differences in bitumen and dust properties can affect mastic (and therefore asphalt) deformation response to high temperature and high shear stresses, even though significant differences are not observed at lower temperatures and lower stress levels.

Linear regression was performed to estimate the average impact of each factor (fine aggregate, binder and test temperature) on Jnr(3.2). The linear relationship (Equation 1) showed a high level of fit to the data with a correlation coefficient of 92%.

$$Jnr(3.2) = -0.161 + 0.00281 \times T - 0.00761 \times B_2 - 0.00817 \times D_B \quad \text{Equation 1}$$

Where T (°C) = test temperature
B_2 (a dummy variable for Binder) = 0 for Bitumen 1 and 1 for Bitumen 2
D_B (a dummy variable for Fine Aggregate) = 0 for Dust A and 1 for Dust B

The change in binder source and the change in fine aggregate source resulted in comparable reductions in mastic Jnr(3.2). It follows that Asphalt A/1 (containing Dust A and Bitumen 1) had less resistance to deformation during high temperature shear stress states than Asphalt B/2 (containing Dust B and Bitumen 2). This indicated that the Hisingerite clay minerals present in Dust B did not adversely affect the mastic. It follows that Hisingerite would not adversely affect asphalt shear resistance.

The properties and results for the individual constituents (fine aggregate and binder) were examined in light of the mastic testing outcomes. First, Dust B had lower apparent density and higher absorptivity than Dust A (Table 2). The lower apparent density required a larger volume of Dust B in order to maintain the target 6:1:7 mass ratio. Although the

binder:filler:aggregate ratio was the same for all mastic samples by mass, when expressed by volume, the Dust B mastic samples had a lower portion of bitumen than Dust A mastics. The higher volume of aggregate in Dust B mastic samples stiffened the mastic to a greater extent than Dust A did. The higher absorptivity of Dust B exacerbated this by reducing the 'effective' binder available. The lower Jnr(3.2) associated with Dust B likely reflected the lower 'effective' binder volume available in Dust B mastic samples and an associated increase in mastic stiffening.

Second, Bitumen 2 was significantly harder than Bitumen 1 based on pre- and post-RTFO viscosity, but softer based on post-RTFO penetration (Table 4). Summary statistics and p-values for differences in mean confirmed significant differences between the two identically graded binder sources. Jnr(3.2) was significantly higher for Bitumen 1 at all test temperatures (Table 8). Consistent with the viscosity results, this indicated that Bitumen 2 was harder and more resistant to deformation than Bitumen 1. However, the stress sensitivity (%Jnr) results show a different trend (Table 9). The two binders did not have significantly different %Jnr at 64°C. This was marginal at 70°C and became significant at 76°C. In all cases, the average %Jnr of Bitumen 2 was higher than for Bitumen 1. This indicated that Bitumen 2 would be more susceptible to shear deformation at high stress levels during high temperatures, despite the lower Jnr(3.2) and higher viscosity at 60°C.

Table 8. Binder Jnr(3.2) summary statistics

Statistics	At 64°C		At 70°C		At 76°C	
	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2
Average	0.23	0.16	0.60	0.40	1.47	1.02
Standard Deviation	0.04	0.05	0.10	0.13	0.24	0.30
CV	16%	33%	16%	32%	16%	29%
p-value (paired t-test)	0.05		0.05		0.05	

Table 9. Binder %Jnr summary statistics

Statistics	At 64°C		At 70°C		At 76°C	
	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2	Bitumen 1	Bitumen 2
Average	30	31	47	53	58	72
Standard Deviation	0	5	1	6	4	11
CV	2%	15%	3%	12%	7%	15%
p-value (paired t-test)	0.35		0.09		0.05	

5. CONCLUSIONS

With a lower mastic Jnr(3.2) it was concluded the asphalt manufactured with Dust B/Bitumen 2 would likely have better resistance to shear stress based on MSCR testing. No adverse impact of using a fine aggregate with predominantly Hisingerite clay minerals was identified. Rather, the slightly reduced apparent density and slightly higher absorptivity of the Hisingerite-rich dust reduced the 'effective' binder content of the mastic and reduced the Jnr(3.2) of Dust B mastic to significantly below that of the Dust A mastic samples. There is no basis for recommending the avoidance of fine aggregate sources containing Hisingerite clay minerals for asphalt production.

Regardless the measured mastic properties, the two sources of common acid-modified (M1000) binder returned significantly different, and inconsistent, results to MSCR and other testing. Bitumen 2 had a higher measured viscosity (indicating harder binder) but a higher penetration (indicating softer binder) after RTFO conditioning. Similarly, Bitumen 2 returned a lower Jnr(3.2) (higher shear stress resistance) but excessive %Jnr (stress sensitivity). Further work is recommended to better understand the two bitumen sources, the differences in their measured properties and the potential impact that may have on asphalt shear stress resistance. Concerningly, these measured differences in binder properties all fell within the Australian viscosity-based paving grade bitumen specification limits. All binder samples were compliant with the specification. Revision of the specification may be required to ensure consistent performance of asphalt surfaces in high stress and high temperature environments, such as airports located in the northern half of Australia.

ACKNOWLEDGEMENTS

The bitumen and mastic testing reported in this paper was performed and managed by John Lysenko, Khoa Vo, Glynn Holleran and Irina Holleran of Fulton Hogan's Sydney and Auckland binder laboratories. Chemical composition testing of dust samples was provided by Emeritus Professor of Geology, Philippa Black of Auckland University.

REFERENCES

[1] Interpreting asphalt concrete creep behavior through non-Newtonian mastic rheology, Zelelew, HM & Papagiannakis, AT, *Road Materials and Pavement Design*, vol. 13, no. 2, 2012, pp. 2766-278.

[2] Experimental investigation of bitumen mastics under shear creep and creep-recovery testing, Elnasri, M, Airey, G & Thom, N, *Proceedings T&DI Airfield and Highway Pavement Specialty Conference*, Los Angeles, California, USA, 9-12 June 2013, American Society of Civil Engineers, pp. 921-932.

[3] Finite element simulation of damage development in asphalt concrete pavements, Scarpas, A, Al-Khoury, R, van Gurp, CAPM & Erkens, SMJG, *Proceedings 8th International Conference on Asphalt Pavements*, Seattle, Washington, USA, 10-14 August 1997, International Society of Asphalt Pavements, pp. 673-692.

[4] Creep behaviour of hot-mix asphalt due to heavy vehicular tire loading, Al-Qadi, IL, Yoo, PJ, Elseifi, MA & Nelson, S, *Journal of Engineering Mechanics*, vol. 135, no. 11, 2009, pp. 1265-1273.

[5] Revision of the Superpave high temperature binder specification: the multiple stress creep recovery test, D'Angelo, J, Kluttz, R, Dongré, R, Stephens, K & Zanzotto, L, *Proceedings Asphalt Paving Technology*, San Antonio, Texas, USA, 11-14 March 2007, Association of Asphalt Paving Technologists, vol. 76, pp. 123-162.

[6] *Development and Use of the Repeated Shear Test (Constant Height): An optional Superpave Mix Design Tool*, Sousa, JB, Solaimanian, M & Weissman, SL, Report SHRP-A-698, Strategic Highway Research Program, National Research Council, August, 1994.

[7] New Airport Pavement Technologies from the USA, White, G, *Proceedings 2014 Australian Airport Association National Conference*, Gold Coast, Australia, 23-27 November 2014, Australian Airports Association.

[8] ACN-PCN system and higher aircraft tyre pressures and wheel loads: the impacts for airports into the future, White, G, *Proceedings 2015 Australian Airport Association National Conference*, Hobart, Australia, 12-16 October 2015, Australian Airports Association.

[9] A case study: quantification and modeling of asphalt overlay delamination on an airport pavement, Horak, E, Maina, J & Emery, S, *Proceedings Eight International Conference on the Bearing Capacity of Roads, Railways and Airfields*, Urbana-Champaign, USA, 29 June - 2 July 2009, pp. 1475-1483.

[10] Investigation into asphalt pavement slippage failures on Runway 04R-22L Newark International Airport, Bognacki, CJ, Frisvold, A & Bennert, T, *Proceedings 2007 FAA Worldwide Airport Technology Transfer Conference*, Atlantic City, USA, 16-18 April 2007, Federal Aviation Administration.

[11] Asphalt mix studies – San Francisco International Airport, Monismith, CL, Vallerga, BA, Harvey, JT, Long F & Jew, A, *Proceedings of the 26th Air Transportation Conference*, San Francisco, California, USA, 19-21 June 2000, American Society of Civil Engineers, pp. 113-124.

[12] Case study of an innovative high stability asphalt mix for heavy aircraft wheel loads, Vallerga, BA, Jew, A & Nokes, WA, *Proceedings of the 26th Air Transportation Conference*, San Francisco, California, USA, 19-21 June2000, American Society of Civil Engineers, pp. 334-345.

[13] Filler and binder influence on the linear viscoelastic behavior of mastics, Delaporte, B, Di Benedetto, H, Chaverot, P & Gauthier, G, *Proceedings Advanced Characterisation of Pavement and Soil Engineering Materials*, Athens, Greece, 20-22 June 2007, Taylor and Francis, pp. 3-13.

[14] Microstructure characterization for modeling HMA behaviour using imaging technology, Tashman, L, Wang, L & Thyagarajan, S, *Road Materials and Pavement Design*, vol. 8, 2007, pp. 207-238.

[15] Contribution of asphalt mix components to permanent deformation resistance, Muraya, PM, Molenaar, AAA & van de Ven, MFC, *Proceedings Eight International Conference on the Bearing Capacity of Roads, Railways and Airfields*, Urbana-Champaign, Illinois, USA, 29 June - 2 July 2009, pp. 259-268.

[16] Asphalt on Australian Airports, Emery, S, *Proceedings AAPA Pavements Industry Conference*, Australian Asphalt Pavements Association, Surfers Paradise, Australia, 18-21 September 2005.

[17] Mechanical properties of filler-asphalt mastics, Liao, M-C, Airey, G & Chen J-S, *International Journal of Pavement Research and Technology*, vol. 6, no. 5, 2013, pp. 576-581.

[18] Modelling of asphalt mastic in terms of filler-bitumen interaction, Faheem, AF & Bahia, HU, *Road Materials and Pavement Design*, vol. 11, 2010, pp. 281-303.

[19] Effect of the nature and filler content on the behavior of the bituminous mastics, Pérez-Jiménez, FP, Miró Recasens, R & Martínez, A, *Road Materials and Pavement Design*, vol. 9, 2008, pp. 417-431.

[20] Influence of filler-bitumen ratio on performance of modified asphalt mortar by additive, Qiu, H, Tan, X, Shu, S & Zhang, H, *Journal of Modern Transportation*, vol. 21, no. 1, 2013, pp. 40-46.

[21] Filler: is it fixing your binder, Bryant, P, *Proceedings AAPA Pavements Industry Conference*, Surfers Paradise, Queensland, Australia, 18-21 September 2005, Australian Asphalt Pavements Association.

[22] Bitumen supply and testing, Neaylon, K, *Proceedings International Public Works Conference*, Darwin, Northern Territory, Australia, 11-15 August 2013, Institute of Public Works Engineering Australasia.

[23] Comparative field performance using multiple crude oil sources, Harnsberger, PM, Farrar, MJ, Huang, S-C & Robertson, RE, *Proceedings Transport Research Board Annual Meeting*, Washington, USA, 23-27 January 2011.

[24] The application of locally developed pavement temperature prediction algorithms in Performance Grade (PG) binder selection, Denneman, E, *Proceedings 26th Southern African Transport Conference*, Pretoria, South Africa, 9-13 July 2007, pp. 257-266.

[25] The effect of particle shape and angularity on SMA mix characterisation, Holleran, G, McCaffrey, L & Wilson, D, *Proceedings 23rd ARRB Conference*, Adelaide, South Australia, Australia, 22-23 July 2008, ARRB Transport Research.
[26] The mechanics of hydrated lime modification of asphalt mixtures: a state of the art review, Lesueur, D, Petit, J & Ritter, H-J, *Road Materials and Pavement Design*, vol. 14, no. 1, 2013, pp. 1-16.
[27] Current status of Superpave binder specification, D'Angelo, JA, *Roads Materials and Pavement Design*, vol. 10, suppl. 1, 2009, pp. 13-24.
[28] *The Multiple Stress Creep Recovery (MSCR) Procedure*, FHA, Technical Brief FHWA-HIF-11-038, Federal Highways Administration, USA April 2011, viewed 8 February 2015, <http://www.fhwa.dot.gov/pavement/materials/pubs/hif11038/hif11038.pdf>.
[29] Correlation between MSCR results and polymer modification of binder, DuBois, E, Mehta, Y & Nolan, A, *Construction and Building Materials*, vol. 65, 2014, pp. 184-190.
[30] Aging bitumen in bulk versus the aging of bitumen in an asphalt mixture, Wu, J, Lepercq, E & Airey, GD, *Proceedings Advanced Characterisation of Pavement and Soil Engineering Materials*, Athens, Greece, 20-22 June 2007, Taylor and Francis, pp. 1631-1640.
[31] Influence of test geometry, temperature, stress level and loading duration on binder properties measured using DSR, Motamed, A & Bahia, HU, Journal of Materials in Civil Engineering, vol. 23, 2011, pp. 1422-1432.
[32] Practical use of multiple stress creep and recovery test, D'Angelo, J & Dongré, R, *Transport Research Record: Journal of the Transportation Research Board*, no. 2126, 2009, pp. 73-82.
[33] Importance of elastic recovery in the DSR for binders and mastics, Clopotel, CS & Bahia, HU, *Engineering Journal*, vol. 16, no. 4, 2012, pp. 99-106.
[34] Rheological behaviour of bitumens modified with PE and PPA at different MSCR creep-recovery times, Domingos, MDI & Faxina, AL, *International Journal of Pavement Engineering*, 2014, pp. 1-13.
[35] Comparison of neat and modified asphalt binders using rheological parameters under virgin, RTFO and PAV aged conditions, Riaz, K, Hafeez, I, Khitab, A, Hussain, M, Ali, F, Ashiq, SZ, Tariq, MK & Ahmed, I, *Life Science Journal*, vol. 10, 2013, pp. 2041-2047.
[36] High temperature rheology property of asphalt mastic with different mineral filler, Li, X, Nan, X & Wei, D, *Advanced Materials Research*, vols. 450-451, 2012, pp. 527-531.
[37] Evaluation of new dynamic shear rheometer testing geometry for performance testing of crumb rubber-modified binder, Baumgardner, G & D'Angelo, JA, *Transport Research Record: Journal of the Transportation Research Board*, no. 2293, 2012, pp. 73-79.

APPENDIX 1 – Asphalt Mixture Details

Comparison of key asphalt characteristics

Parameter	Asphalt A/1	Asphalt B/2	Specific Target/Limit
Fine Aggregate	Dust A	Dust B	Basaltic
Binder Source	Bitumen 1	Bitumen 2	Acid modified
M1000 Binder Content (%)	5.8	5.8	> 5.6
Hydrated Lime Content (%)	1.0	1.0	0.5 - 1.5
Marshall Stability (kN)	15.3	17.5	> 12.0
Marshall Flow (mm)	3.3	3.1	< 3.5
Air Voids (%)	4.4	4.2	3 - 5
Resilient Modulus (MPa)	3,550	2,790	Report only
Indirect Diametrical Tensile Strength (kN)	903	960	Report only
Tensile Strength Ratio (%)	99	98	Report only
Wheel Tracking (mm)	3.7	3.4	Report only

Comparison of asphalt particle size distributions

Australian Standard Sieve (mm)	Percentage Passing by Mass (%)		
	Asphalt A/1	Asphalt B/2	Specification Target
19.0	100	100	100
13.2	99	98	100
9.5	84	83	82
6.7	70	71	70
4.75	60	62	60
2.36	63	47	44
1.18	29	31	33
0.600	20	22	25
0.300	13	15	16
0.150	8.8	9.8	10
0.075	6.1	6.5	5

Appendix 2 – Binder MSCR Test Results

Binder MSCR testing results at 64°C

Bitumen Sample	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr	PG Rating
1.1	45.8	28.0	0.17	0.22	30	PG 64 E
1.2	46.5	29.0	0.16	0.21	30	PG 64 E
1.3	42.2	24.2	0.21	0.28	31	PG 64 E
2.1	52.3	33.3	0.14	0.20	37	PG 64 E
2.2	60.9	46.8	0.08	0.10	30	PG 64 E
2.3	49.0	33.4	0.14	0.18	28	PG 64 E

Binder MSCR testing results at 70°C

Bitumen Sample	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr	PG Rating
1.1	39.6	14.4	0.38	0.56	47	PG 70 V
1.2	40.4	15.3	0.36	0.53	47	PG 70 V
1.3	35.3	11.4	0.49	0.72	45	PG 70 V
2.1	46.6	18.6	0.32	0.50	58	PG 70 V
2.2	56.1	31.5	0.17	0.26	55	PG 70 E
2.3	42.7	18.9	0.31	0.45	46	PG 70 E

Binder MSCR testing results at 76°C

Bitumen Sample	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr	PG Rating
1.1	33.0	5.6	0.86	1.38	61	PG 76 H
1.2	33.8	6.1	0.81	1.30	61	PG 76 H
1.3	27.9	4.0	1.13	1.74	54	PG 76 H
2.1	39.5	7.6	0.71	1.24	76	PG 76 H*
2.2	49.5	15.5	0.37	0.68	82	PG 76 V*
2.3	35.3	7.7	0.71	1.14	60	PG 76 H

* denotes samples that would not receive a PG grading at this temperature due to high stress sensitivity.

Appendix 3 – Mastic MSCR Test Results

Mastic MSCR testing results at 64°C

Mastic Sample	Bitumen	Dust	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr
1.1.A	1.1	A	69	34	0.012	0.015	28
1.1.B	1.1	B	65	35	0.012	0.018	45
1.2.A	1.2	A	66	37	0.012	0.017	44
1.2.B	1.2	B	65	36	0.009	0.013	49
1.3.A	1.3	A	61	49	0.006	0.007	9
1.3.B	1.3	B	65	49	0.009	0.012	29
2.1.A	2.1	A	71	40	0.007	0.011	67
2.1.B	2.1	B	70	41	0.009	0.013	49
2.2.A	2.2	A	70	34	0.012	0.015	28
2.2.B	2.2	B	72	35	0.012	0.018	45
2.3.A	2.3	A	68	41	0.007	0.011	74
2.3.B	2.3	B	66	47	0.007	0.008	14

Mastic MSCR testing results at 70°C

Mastic Sample	Bitumen	Dust	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr
1.1.A	1.1	A	63	29	0.019	0.022	19
1.1.B	1.1	B	61	27	0.027	0.031	18
1.2.A	1.2	A	60	29	0.020	0.022	12
1.2.B	1.2	B	60	27	0.027	0.031	16
1.3.A	1.3	A	56	26	0.026	0.027	4
1.3.B	1.3	B	57	27	0.030	0.033	10
2.1.A	2.1	A	58	32	0.020	0.019	2
2.1.B	2.1	B	63	36	0.018	0.019	11
2.2.A	2.2	A	68	38	0.011	0.014	19
2.2.B	2.2	B	66	35	0.020	0.026	28
2.3.A	2.3	A	59	31	0.017	0.019	15
2.3.B	2.3	B	60	32	0.020	0.023	19

Mastic MSCR testing results at 76°C

Mastic Sample	Bitumen	Dust	AR(0.1)	AR(3.2)	Jnr(0.1)	Jnr(3.2)	%Jnr
1.1.A	1.1	A	58	21	0.035	0.041	17
1.1.B	1.1	B	57	19	0.047	0.062	32
1.2.A	1.2	A	59	21	0.030	0.039	29
1.2.B	1.2	B	56	19	0.048	0.063	31
1.3.A	1.3	A	51	19	0.047	0.052	10
1.3.B	1.3	B	53	19	0.050	0.067	32
2.1.A	2.1	A	55	23	0.032	0.037	13
2.1.B	2.1	B	59	25	0.033	0.043	29
2.2.A	2.2	A	62	28	0.022	0.026	18
2.2.B	2.2	B	61	27	0.039	0.050	28
2.3.A	2.3	A	54	22	0.031	0.036	17
2.3.B	2.3	B	56	23	0.037	0.047	26