Review of the Effect of Moisture on Asphalt Pavement Performance

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Summary

Moisture can enter the asphalt pavement in various ways and forms. The performance of asphalt pavements can be severely influenced by moisture ingress as it may lead to loss of adhesion between the binder and aggregate and/or a loss of cohesion within the asphalt binder itself resulting in the reduction in asphalt mix stiffness. Common mechanisms of stripping are detachment, displacement, spontaneous emulsification, pore pressure and hydraulic scouring. Asphalt mixes have varying resistance to moisture damage, therefore it is crucial to mitigate the risk of moisture damage in the design and construction phases.

A literature review of the effect of moisture on asphalt pavements revealed that aggregate type and mineralogy are more dominant factors than bitumen type affecting moisture sensitivity of the asphalt mix. In Perth, approved mixes are commonly manufactured using crushed granite aggregate which is categorised as hydrophilic rock. However, no data was found related to any investigation to ascertain whether or not stripping issues are associated with the use of granite aggregates in Western Australian mixes.

Main Roads Western Australia (Main Roads) specifies the tensile strength ratio (TSR) test for assessing the stripping potential of asphalt using the same test methods and minimum requirements as Department of Transport Victoria (DoT Vic) and the Queensland Department of Transport and Main Roads (TMR). The Hamburg wheel tracking test (HWTT) is used by several US transportation agencies to estimate the moisture resistance of asphalt mixes. Main Roads may also consider using the HWTT in the future. Other tests that could be considered are related to aggregate adhesion with bitumen such as the rolling bottle, saturation ageing tensile stiffness (SATS), pneumatic adhesion tensile testing instrument (PATTI) and the immersion-mechanical test.

Drainage is vital for pavement performance; therefore, provision of appropriate surface and sub-surface drainage should be emphasised in technical documents. This will ensure the application of designs that inhibit the entry of water and ensure that any water that enters the pavement does not build up to the level that it generates cyclic pore pressure and thus increases the risk of stripping. Similarly, asphalt compaction is key to inhibit moisture damage to asphalt pavements as moisture ingress is influenced by in situ air void content. The Main Roads target in situ asphalt air void content (3–6%) appears to be in line with other road and transport agencies within Australia and overseas; however, efficient compaction procedure is required to avoid any deviation from the target.

Main Roads current practice of applying a sprayed seal on the uppermost layer of the size 14 mm asphalt intermediate course and a tack coat of diluted emulsion on the prepared surface is in line with other agency practice.

The outcome of the virtual workshop highlighted the need for an investigation of the extent of asphalt stripping in Western Australia and also the stripping potential of the asphalt mix manufactured using granite aggregate. In relation to the extent of stripping, no data was available for review; however, Main Roads staff advised that stripping is not a major cause of distress in Western Australian asphalt pavements.

Main Roads data related to the Gateway field trials was analysed to provide insights about the use of granite aggregate, asphalt mix properties and construction practice. Two petrographic test results were investigated and found that granite used in the Gateway mixes was not sensitive to moisture damage in asphalt. Not all granites are prone to stripping, and stripping is a combined result of several factors such as the composition of aggregate, bitumen type, mix design parameters, air void content and drainage and construction-related issues. As granite is a widely used aggregate in asphalt mixes in Perth, more investigation is required to evaluate the suitability of the granite aggregate for the mix.

Although the Gateway field trials and stripping issues reported on other sites in Western Australia show considerable improvement from the 2nd to the 3rd generation mix (improved asphalt mixes), it would be beneficial to investigate stripping on other sites to confirm this conclusion. If stripping is identified on other sites, an appropriate technical solution should be warranted. It may include provision of appropriate laboratory tests for moisture susceptibility of asphalt mixes and evaluating asphalt manufacturing plants and procedures.

The main recommendations are:

- Consider implementing laboratory tests to indicate moisture sensitivity and stripping potential of asphalt mixes, aggregate binder adhesion stability and aggregate susceptibility to moisture.
- Investigate clauses or the provision of surface and subsurface drainage in the technical documentation.
- Consider a double application rate of the binder on joints and chases for better waterproofing, tack coat at vertical edges between old and new pavements and on the top of an existing asphalt layer to ensure an adequate bond between pavement layers.
- Investigate the selected sections of the full depth asphalt (FDA) pavement which have undergone stripping to endorse the above conclusion and pinpoint the relationship of stripping to different factors and recommend appropriate solutions.

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1 Introduction and Scope

1.1 Project Background

In recent years there has been an increase in the use of full-depth asphalt pavements (FDA) in Western Australia for heavily trafficked urban roads. The performance of asphalt pavements can be severely influenced by moisture (Jitsangiam et al. 2019). Moisture ingress may lead to loss of adhesion between the binder and aggregate and/or a loss of cohesion within the asphalt binder itself resulting in the reduction in asphalt mix stiffness. Asphalt mixes have varying resistance to moisture damage; therefore, it is crucial to mitigate the risk of moisture damage in the design and construction phases.

Moisture damage of asphalt pavements may manifest itself in several forms ranging from loss of mix integrity throughout one or more layers or at the bottom of a layer working its way to the top. The damage can visually appear as several forms ranging from rutting, fatigue cracking, potholes to ravelling across an entire pavement surface.

Past investigations have focussed on material issues, but pavement design, asphalt mix design and construction standards were subsequently identified as the critical factors (Australian Asphalt Pavement Association (AAPA) 2005). The Western Australian Road Research and Innovation Program (WARRIP) Project 2021-002 has the objective of improving guidance on the potential contributing factors to stripping and provide recommendations to reduce its incidence.

1.2 Scope and Objectives

The aim of the project was to improve guidance on the potential contributing factors to stripping and the scope of works comprised the following important objectives:

- Improve technical capability by enhancing understanding of stripping mechanisms as well as asphalt moisture interaction under operational conditions.
- Suggest improvement to Main Roads Western Australia (Main Roads) current practice for consideration in order to achieve moisture-resistant asphalt pavements.

1.3 Structure of the Report

This report presents the findings of the investigations in relation to the effect of moisture on asphalt pavement performance. The structure and contents are as follows:

- Section 1 provides an overview of the project objectives and scope.
- Section 2 outlines the scope of the literature review.
- Section 3 summarises the asphalt stripping research findings including general mechanisms, critical design factors aggregate mineralogy, testing procedures and road design and construction considerations.
- Section 4 provides a comparison of Australian state road and transport agencies' practices.
- Section 5 summarises the international road agencies' practices.
- Section 6 outlines the stripping occurrences in Western Australia.
- Section 7 provides a summary of the discussion at the virtual workshop.

- Section 8 provides the outcome of data analysis and establishes linkages to the Main Roads current practice.
- Section 9 outlines the conclusions.
- Section 10 provides the recommendations.
- Section 11 outlines future research direction.

2 Scoping the Literature Review

The literature review covered the most critical areas related to the effect of moisture on asphalt pavements based on the project budget and timeframe. The scope of the review comprised the following:

- an overview of stripping mechanisms in asphalt pavements
- a review of Main Roads current construction practices, specifications and asphalt mix design procedures
- a comparison of Main Roads practices with other Australian state road and transport agency practices
- a review of selected overseas road agency practices for asphalt pavements
- a review of national and international literature related to asphalt mix design, pavement design, construction and testing procedures.

The detailed results of the literature review are presented in Sections 3, 4 and 5 as outlined below.

2.1 Stripping Mechanisms of Asphalt Pavements

This section includes a review of moisture-induced damage and stripping mechanisms in asphalt. It also covers the sources of moisture ingress, modes of asphalt pavement failures and how these failures are related to moisture susceptibility of different asphalt mixes.

The following areas are discussed:

- sources of moisture and the movement of moisture in asphalt pavements
- loss of binder adhesion (e.g. due to aggregate mineralogy, bitumen type, pumping mechanism of water within asphalt under traffic loads, temperature etc.)
- modes of moisture-related failures in asphalt
- critical design factors influencing asphalt permeability
- pavement structure and layer thickness
- pavement drainage and compaction levels.

2.2 Comparison of Australian State Road and Transport Agencies' Practices

Most of the Australian state road and transport agencies (SRTAs) have their own specifications for asphalt materials and pavement construction. Austroads has also published a specification framework for asphalt mixes and asphalt pavement design details in the *Guide to Pavement Technology*.

This section provides a comparison of Main Roads specifications with other Australian jurisdiction specifications and technical documents issued by different organisations such as Australian Flexible Pavement Association (AfPA) and Institute of Public Works Engineering Australasia (IPWEA).

Full-depth asphalt pavements (FDA) are mainly composed of dense graded asphalt (DGA) in the structural layers, stone mastic asphalt (SMA), open graded asphalt (OGA) and crumb rubber asphalt (CRA) as wearing courses depending upon performance requirements. High modulus asphalt (EME2) has been used as a base layer of heavy-duty pavements. OGA is used as a drainage layer wherever required.

The main focus of the project is on DGA mix, however, other mix types such as SMA, OGA, CRA and EME2 have been covered briefly.

The focus areas for all types of mixes were:

- mix design methods (e.g. Marshall, Bailey, Strategic Highway Research Program (SHRP))
- mix nominal sizes, mix components and grading

- type of aggregate and binder content
- mix volumetric properties
- mix performance requirements
- field compaction requirements
- pavement design and construction practices
- sampling and testing procedures
- asphalt manufacturing process.

2.3 Overseas Road and Transport Agencies Practices

This section summarises the asphalt specifications and practices adopted by overseas road and transport agencies such as the NZ Transport Agency (NZTA), Southern African Bitumen Association (SABITA), Highways England (UK road agency) and selected US road agencies.

2.4 National and International Research

National and international literature related to the effect of moisture on asphalt pavements was investigated including peer reviewed publications and conference proceedings. The main areas covered include:

- effect of moisture on asphalt pavements
- asphalt mix design and manufacturing process
- laboratory and field testing
- pavement design and construction practices.

3 Asphalt Stripping Research Findings

Asphalt stripping research findings cover the stripping mechanisms, moisture ingress into asphalt pavements, critical design factors, type and mineralogy of aggregate for asphalt mixes, testing procedures and road design and construction considerations.

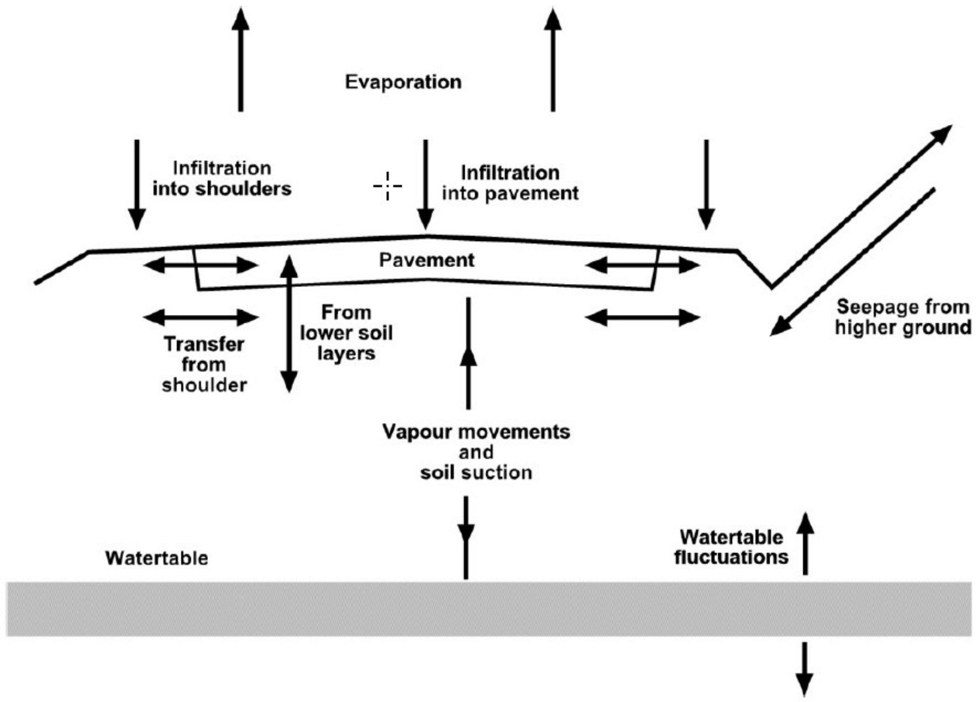
3.1 General Mechanisms

Asphalt properties are complex and its performance requirements vary considerably with its application (Austroads 2009). Pavement materials are subjected to several environmental and operational conditions during their life cycle (e.g. moisture variation, binder ageing, increased traffic loading etc.). The performance of full-depth asphalt pavements is largely based on asphalt quality, characteristics and construction. Moisture-induced damage is one of the most significant flexible pavement damage types (Htet 2015).

3.1.1 Moisture Ingress

Moisture can enter the asphalt pavement in various ways and forms. The major modes of water ingress are infiltration of surface water due to mix permeability, capillary rise of subsurface water and permeation or diffusion of water vapours (Dhakal 2015). Moisture can enter from the sides and bottom of the pavement as seepage from ditches and high watertables in the cut areas (Kandhal 1992). Surface water may cause water flow through the connected macro-pores of the asphalt wearing surface (e.g. OGA) or may reside in the macro-pores of the mix (e.g. residual moisture after rainfall) or water may be present in the aggregates even before construction (due to inadequate drying procedure). Moisture movements in pavements are illustrated in Figure 3.1.

Figure 3.1: Moisture movements in pavements



Source: Austroads (2017).

3.1.2 Moisture-induced Damage

Moisture-induced damage to asphalt mixes is a complex problem and the contribution of moisture to asphalt pavement degradation is not well understood (Kringos et al. 2008). Moisture damage can be defined as the progressive functional deterioration of the asphalt mixture by loss of the adhesive bond between the bituminous binder and the aggregate surface and/or loss of the cohesive resistance within the bituminous binder mainly by the action of water (Kiggundu & Roberts 1988).

Tarrer and Wagh (1991) described 5 different mechanisms of stripping in asphalt pavements due to water or moisture as follows:

- Detachment
  Detachment is the separation of bitumen film from an aggregate surface by a thin layer of water with no obvious break in the bitumen film (Asphalt Institute 1981; Majidzadeh & Brovold 1968). It is based on the theory of interfacial energy, emphasising the effect of polarity of the molecules present at the surface of the 2 phases. Most aggregates have electrically charged surfaces. Bitumen exhibits little polar activity, therefore, the bond that develops between bitumen and an aggregate is primarily due to relatively weak dispersion forces. Water molecules, on the other hand, are highly polar and are attracted to aggregates by much stronger orientation forces (Road Research Laboratory 1962).
- Displacement
  Stripping can occur when water penetrates the bitumen at the aggregate surface through a break in the asphalt film (Asphalt Institute 1981; Fromm 1974; Majidzadeh & Brovold 1968; Scott 1978). Stripping by displacement can result from pinholes in the asphalt film, which can form soon after coating a dusty aggregate (Fromm 1974) or by incomplete coating of the aggregates or by film rupture (Asphalt Institute 1981; Fromm 1974; Majidzadeh & Brovold 1968; Road Research Laboratory 1962). This mechanism is congruent with a thermodynamic approach to adhesion: that is, water will displace bitumen from an aggregate surface when a three-phase interface exists.
- Spontaneous emulsification
  Formation of inverted bituminous emulsion leads to stripping which is further aggravated by the presence of emulsifiers such as mineral clays and asphalt additives (Asphalt Institute 1981; Fromm 1974; Scott 1978).
- Pore pressure
  This stripping mechanism can occur in high voids mixes where water may circulate freely through interconnected voids (Asphalt Institute 1981; Majidzadeh & Brovold 1968). Water may become trapped in impermeable voids that previously permitted water circulation upon densification of the mix under traffic loading. Traffic loading may induce high excess pore pressures in the trapped water causing stripping of the bitumen film from the aggregate.
- Hydraulic scouring
  Hydraulic scouring occurs only in surface courses as water in the saturated pavement surface is pressed down in front of the vehicle tyre and immediately sucked away from the pavement behind the tyre (Asphalt Institute 1981).

Moisture can damage asphalt pavements in different ways. Traffic action increases pore water pressure in the voids of the mixes particularly in pessimum voids. At a low percentage of air voids (less than 4 to 5%), the voids are not connected and the potential for water intrusion and stripping is low. At a high percentage of air voids (greater than 15 to 20%), voids are interconnected such that the mixture is free draining. In the pessimum voids range (generally 5 to 20%), some of the air voids are interconnected and water may become trapped in the mixture. Traffic stresses can also rupture the thin bitumen films that occur at the junction of the crushed faces of individual aggregates which act as avenues for moisture into the interface (Bagampadde 2004). The effect of traffic is reinforced by stripping firstly occurring in the more heavily trafficked outer traffic lanes (Kandhal 1992).

3.2 Critical Design Factors

Advisory Note 19 (AAPA 2005) identified that critical design factors influencing permeability of an asphalt mix are air voids, aggregate grading, nominal size of mix, binder film index, and workability and compactability. Asphalt mixes are generally designed to achieve air voids of around 3 to 7%. At air voids of less than 7% the permeability is low due to the low extent of the interconnected voids. Therefore, achievement of low in situ air voids (less than 7%) becomes more critical with larger mix sizes and coarser grading. Permeability can be greater at lower air voids at larger nominal maximum particle size. Cooley et al. (2002) have shown that pavement can become excessively permeable with lower in-place air voids at larger nominal maximum particle size. According to Horak et al. (2017), the permeability of asphalt mixes is generally very low at air void contents less than 6% and increases significantly between 6% and 7%. In addition, a higher in-service temperature results in lower binder viscosity and hence increased susceptibility to moisture damage. Environmental factors such as high traffic loading, high rainfall and inadequate pavement drainage increase the risk of stripping.

Kennedy and Anagnos (1984) carried out an extensive study over a six-year period to identify techniques for reducing moisture damage in asphalt as part of a research project for the Texas State Department of Highways and Public Transportation. Key findings indicated that the stripping potential of asphalt was influenced by the types of aggregates used: more prevalent for siliceous aggregate and rhyolite whereas limestone aggregates were generally resistant to stripping. Moreover, higher viscosity bitumen was more resistant to stripping. Moisture penetration in asphalt depends on the density and gradation of the mix (i.e. dense graded mixes retard moisture penetration whereas open graded friction courses allow moisture to enter the underlying layers).

Moisture-induced damage to asphalt can be minimised by:

- adequate compaction (at air void content of more than 7%, water can readily penetrate the mix)
- avoiding use of moisture-susceptible aggregates and asphalt
- provision of adequate drainage
- sealing the asphalt-aggregate surfaces
- treating moisture-susceptible materials (aggregate and asphalt) using antistripping agents (e.g. cement and hydrated lime).

Thodesen and Hoff (2010), as a part of a joint study at the University of Nottingham UK, ZAG Slovenia, VTT Finland and SINTEF Norway related to water effects on asphalt, indicated that the following significant factors increase the risk of stripping:

- inadequate pavement drainage
- inadequate compaction of hot mix asphalt
- excessive dust coating on aggregate
- use of friable or weak aggregates
- use of overlays and seal coats when there is moisture beneath the pavement
- inappropriate use of open graded asphalt.

The risk can be reduced by:

- appropriate pavement and asphalt mix design and testing procedures to assess the compatibility of the bitumen-aggregate mix to be used
- use of liquid antistripping agents, hydrated lime and modified bitumen
- appropriate construction practices.

Kiggundu and Roberts (1988) investigated the relationships between the theories of adhesive bond loss and stripping mechanisms as shown in Table 3.1.

Table 3.1: Relationships between theories of adhesive bond loss and stripping mechanisms

Mode of failure	Proposed operation mode	Theory								
		Mechanical interlock			Chemical reaction			Interfacial energy		
Proposed operating mode		P	C	P-C	P	C	P-C	P	C	P-C
Stripping mechanism	Detachment	S						S	W	
	Displacement					S		S		
	Spontaneous emulsification				S	W				
	Film rupture	S								
	Pore pressure	S								
	Hydraulic scouring	S								
	pH instability					S				S

Key: P: Physical, C: Chemical, P-C: Physical – Chemical, S: Primary contributor, W: Secondary contributor.

Source: Kiggundu and Roberts (1988).

No single theory seems to completely explain the loss of adhesion, rather it is most likely that 2 or more mechanisms may occur simultaneously in any one mixture, thus leading to loss of adhesion. Terrel and Al-Swailmi (1993) reported several factors appearing to affect adhesion, namely:

- surface tension of the bitumen and aggregate
- chemical composition of the bitumen and aggregate mineralogy
- bitumen viscosity
- aggregate characteristics (i.e. surface texture, porosity, absorption, cleanliness)
- aggregate moisture content and temperature at the time of mixing with bitumen.

Taib et al. (2019) summarised the factors influencing moisture damage in asphalt pavements as shown in Table 3.2.

Table 3.2: Factors influencing moisture damage

Factors	Determining characteristics	Favourable properties
Aggregate properties(1,2,3,4)	Surface texture, mineralogy, porosity, surface moisture, surface chemical composition and surface coating	Rough surface texture, carbonaceous aggregate, low silica content, optimum amount of porosity, surface dry aggregate, no coating
Bitumen characteristics(1,4,5)	Asphalt film thickness, viscosity, physical and chemical structure	High asphalt film thickness, high viscosity, existence of phenol and nitrogen
Construction method(2,4,5,6)	Compaction method, drainage system, air voids mechanism	Adequate compaction, proper drainage system, low air void percentages, adapt water resistance additives on each layer of pavement
Environmental condition(1,4)	Climates, environmental temperature	Warm climates, mild temperature (low rate of changing in temperature), no freeze-thaw cycles
Imposed traffic load(2)	Traffic load	Low traffic

1. Hicks (1991).
2. Emery and Seddik (1997).
3. Hanz et al. (2007).
4. Austroads (2007a).
5. Birgisson et al. (2005).
6. Kumar and Anand (2012).

Source: Taib et al. (2019).

3.3 Aggregate Type and Mineralogy

Aggregates constitute approximately 95% of the asphalt mix by weight and absorption of bitumen into aggregate is influenced by aggregate chemistry. Basic aggregates generally form stronger bonds with bitumen than acidic aggregates; therefore, stripping is higher in mixes with acidic granites with hydroxylated SiO_4 (Fromm 1974).

Common minerals and their relationships to stripping are summarised in Table 3.3.

Table 3.3: Mineral types in aggregates and their relationship to stripping

Category	Mineral type	Rock	Comment
Silica[1]	Quartz – SiO_4	Granite Rhyolite Sandstone Quartzite	Poor adherends as water attaches due to H-bonding.
Ferro-magnesian[2]	Olivine – $(MgFe)_2SiO_4$ Augite – $(Ca,Mg,Fe)(Si,Al)_2O_6$ Hornblende – $(Ca,Na)_{2\text{-}3}(Mg,Fe^{2+},Fe^{3+},Al)_5(Al,Si)_8\text{-}O_{22}(OH)_2$ Biotite – $K(Mg,Fe^{2+})_3(Al,Fe^{3+})\text{-}Si_3O_{10}(OH)_2$	Gabbro Diabase Andesite Basalt Diorite Mica	Olivine and augite form insoluble Mg and Ca salts while biotite gives soluble K salts. Hornblende is intermediary in character.
Limestone[3]	Calcite – $CaCO_3$ Dolomite – $CaMg(CO_3)_2$	Limestone Chalk Dolomite	Generally good adherends but are friable. Undergo strong acid-base and electrostatic interactions with bitumen. Some have soluble salts.
Feldspar[4]	Albite – $NaAlSi_3O_8$ Orthoclase – $KAlSi_3O_8$ Anorthite – $CaAl_2Si_2O_8$	Rhyolite Granite Quartzite Gneiss Sandstone Diabase Gabbro	Some strip due to Na and K soluble salt formation. Anorthite forms insoluble Ca salts that are resistant to stripping.
Clays[5]	Illite Kaolinite Montmorillonite	Dust Baghouse fines	Fine coatings (< 4μ) and readily take up water. Form stable bonds lime.

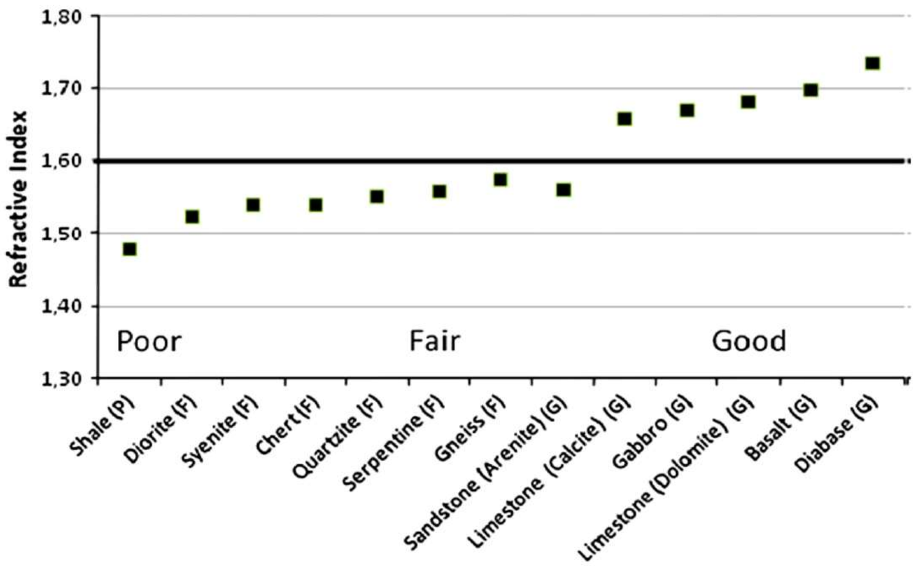
1. Rice (1959), Majidzadeh and Brovold (1968), Stuart (1990).
2. Rice (1959), Majidzadeh and Brovold (1968), Stuart (1990).
3. Curtis et al. (1990), Stuart (1990).
4. Scott (1978), Stuart (1990).
5. Clough and Martinez (1961), Ishai and Craus (1977), Balghunaim (1991), Kandhal et al. (1998).

Source: Bagampadde et al. (2004).

Most of the studies show that the choice of aggregate type is a more dominant factor than bitumen type affecting moisture sensitivity of the resulting asphalt mixes. The asphalt mixes having aggregates comprising Ca-feldspars and ferromagnesian minerals show improved resistance to moisture damage. However, the asphalt mixes with the presence of a high concentration of acid-insoluble minerals (SiO_2 and Al_2O_3) in the aggregates are sensitive to moisture damage. The surface texture of an aggregate (i.e. porous and rough surfaces) promote adhesion by providing mechanical interlock between the bitumen and surface of the aggregate (American Society for Testing and Materials (ASTM) 1959; Majidzadeh & Brovold 1968).

Soenen et al. (2020) reported that aggregates and minerals that have a reflective index higher than a cut-off value of around 1.6 are expected to be less susceptible to stripping, as represented in Figure 3.2.

Figure 3.2: High refractive index value for aggregate and minerals classified according to their degree of resistance to stripping



Key: P: Poor, F: Fair, G: Good.

Source: Soenen et al. (2020).

Figure 3.2 indicates that mixes composed of aggregates from source rocks such as basalt, diabase, limestone and dolomite are expected to be less susceptible to stripping. Conversely, mixes comprising aggregates of shale, diorite, syenite, chert and quartzite tend to be highly susceptibility to stripping. Mixes manufactured with aggregates from rocks and minerals such as serpentine, gneiss and sandstone are expected to have poor to fair stripping resistance.

The association between the refractive index and stripping resistance is also supported by other studies. The reflective index of a material describes how fast light travels through the material. It is a value calculated from the ratio of the speed of light in a vacuum to that in the material. For instance, Lyne (2014) used the Hamaker constant as a practical way to calculate Van der Waals interaction by using the dielectric properties of 2 interacting bodies and an intervening medium. Basalt has the highest Hamaker constant and granite has the lowest (Table 3.4). Each Hamaker constant value has been compared with resistance to stripping according to Cordon (1979). The performance of the aggregates and minerals correlated well with the Hamaker constant where resistance to stripping data was available.

Table 3.4: Hamaker constant calculated by Lyne (2014)

Rocks and minerals	Hamaker constant (A_{total}(.10 20 J)[(1)]	Resistance to stripping[(2)]
Basalt	11.06	Good
Limestone (dolomite)	10.33	Good
Limestone (calcite)	10.07	Good
Granite (kaolinite)	8.88	Fair
Quartz	8.74	Fair
Albite	8.56	–
Microcline	8.54	–
Granite (endelite, allophane, hyalite)	7.33–8.42	Fair

1. Hamaker constant calculated according to Israelachvili (2011).
2. Resistance to stripping calculated according to Cordon (1979).

Source: Lyne (2014).

The mineral composition of the aggregate is an important factor in decreasing the asphalt stripping potential. According to Tarrer and Wagh (1991) the aggregate's mineral composition could be determined by X-ray Fluorescence Spectroscopy (XRF).

Antistripping agents help in reducing the stripping potential in asphalt mixes. Common antistripping agents are fatty amines (solid or liquid), iron naphthenate and hydrated lime. There is considerable evidence that acids in bitumen migrate to the bitumen-aggregate surface forming salts with sodium and potassium minerals frequently associated with stripping-prone aggregates. The sodium and potassium salts are much more soluble than are the calcium salts of the same acids. Therefore, hydrated lime treatment of aggregate results in the formation of calcium salts at the bitumen-aggregate interface, giving a material that is more resistant to stripping.

Zhang, Airey and Grenfell et al. (2015a) experimentally evaluated the cohesive and adhesive bond strength and fracture energy of bitumen-aggregate systems. The results indicated that the limestone aggregate tended to have more surface area to bond with bitumen while the granite aggregate contained more mineral phases known to provide good adhesion with bitumen. Tensile strength of the bitumen film and the bitumen-aggregate interface measured with the pneumatic adhesion tensile testing instrument (PATTI) test was shown to change from cohesive failure to mixed cohesive and adhesive failure as the test temperature decreased from 40 °C to −10 °C. Moreover, mineral composition seems more important than morphology in terms of bonding with bitumen and the peel test is a suitable method to characterise the fracture energy of bitumen films.

3.4 Testing Procedures

In general, it is still not fully clear which mechanism or combination of mechanisms induce moisture damage, and how these depend on conditions such as temperature, mix design, binder aging, traffic, water exposure time and the presence of other pavement failures. Though continuous improvement on moisture susceptibility tests for asphalt mixes has been made in clarifying and understanding the mechanism of moisture damage, a reliable and practical laboratory method that can simulate moisture damage in the field considering the effects of loading and environmental conditions in real pavement is still needed for Australian road agencies (Diab & You 2013; Hand 2015).

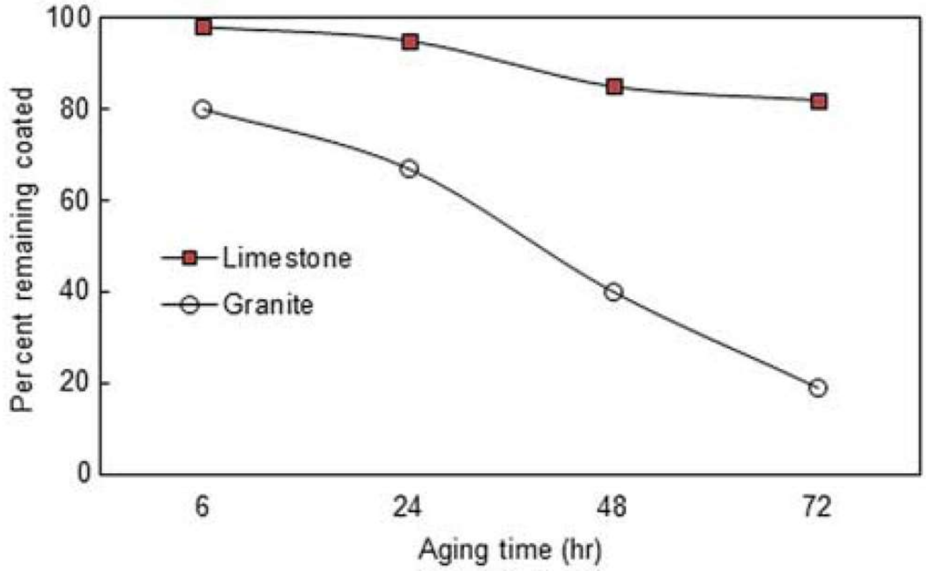
Moisture sensitivity has been studied using many test procedures, including the following (Soenen et al. 2020):

- individual components comprising binder, aggregate and additives
- loose asphalt mixes
- compacted asphalt mixes
- mixes in a pavement under field conditions.

Austroads (2019) describes the resistance to stripping assessed using the T230 test method (Roads and Maritime Services (RMS) 2012b) which is used to test single-sized unprecoated aggregate where an adhesion agent is not added to the binder prior to testing. Currently Main Roads does not specify any resistance to stripping of aggregate and binder tests.

Grenfell et al. (2014) reported the laboratory tests conducted on loose mix which included the static immersion test (AASHTO T182, ASTM D1664), boiling water test (ASTM D3625/D3625M), total water immersion test (TWIT), ultrasonic water bath technique and the rolling bottle method (BS EN 12697-11). Tests on compacted mixtures included the resistance of compacted asphalt mixtures to moisture-induced damage test (AASHTO T283) and the saturation ageing tensile stiffness (SATS) test. Results from the rolling bottle (RBT) and SATS tests indicate superior moisture resistance of the mixture made with limestone aggregate as shown in Figure 3.3.

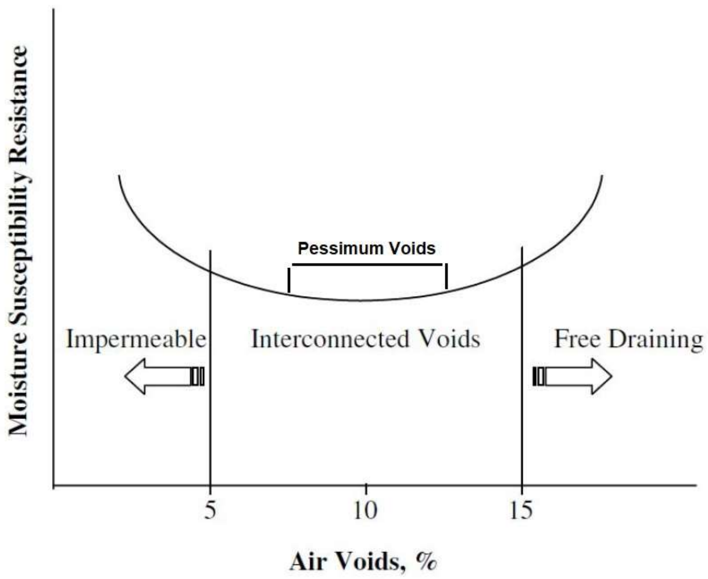
Figure 3.3: A plot of the percentage of aggregate that remains coated with bitumen as a function of conditioning time during the rolling bottle test



Source: Grenfell et al. (2014).

Terrel and Shute (1989) investigated the water sensitivity of asphalt mixes with respect to performance including fatigue, permanent deformation, and low-temperature cracking, as well as the effect of ageing and laboratory testing procedure development that will predict field performance. The relationship of air voids and the relative strength of mixes, following water conditioning, showed a region of pessimum voids as presented in Figure 3.4.

Figure 3.4: Relationship of air voids and relative strength of mixes following water conditioning



Source: Terrel and Shute (1989).

The relationship shows that moisture susceptibility resistance is higher for lower air voids and free draining pavement and lower in the zone of pessimum air voids where pavements are usually constructed. Permeability is controlled by the interconnected air voids. Nominal maximum particle size has an impact on permeability as an increase in nominal maximum particle size can enhance permeability at lower air voids.

Taylor and Khosla (1983) reported the test procedures developed at that time in an effort to determine the susceptibility of asphalt mixtures, as follows:

- Static immersion tests

  - ASTM D1664, Lee test, Holmes water displacement, Oberbach test, German U-37 test
- Dynamic immersion tests
  - Nicholson test, Dow or Tyler Wash test
- Boiling tests
  - ASTM D3625/D3625M, Reidel and Weber test
- Chemical immersion test
  - Reidel and Weber test
- Quantitative coating evaluation tests
  - Dye adsorption test, Mechanical integration method, Radioactive isotope tracer, Tracer-salt with flame photometer analysis, Light-reflection method
- Abrasion tests
  - Cold water abrasion test, Abrasion-displacement test, Surface water abrasion test
- Simulated traffic tests
  - English trafficking tests, test tracks
- Immersion-mechanical tests
  - Immersion-compression test (ASTM D1075 or AASHTO T165), Indirect tension (diametral compression) test, Water susceptibility test, Moisture vapor susceptibility test, Marshall immersion test
- Non-destructive tests
  - Sonic test, Resilient modulus test
- Miscellaneous tests
  - Detachment tests, Briquet soaking test, Swell test, Stripping coefficient measurement test, Peeling test, Texas Freeze-thaw pedestal test.

Some of these tests are no longer being used.

3.5 Road Design and Construction Considerations

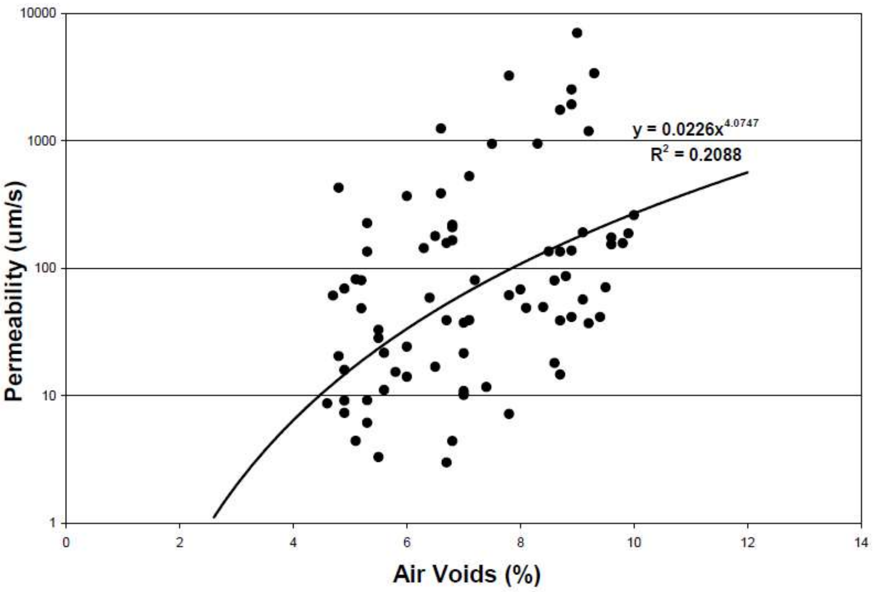
Good construction practices should be adopted to place and compact asphalt layers to inhibit moisture damage. The air voids after compaction are very important as the permeability characteristics of asphalt mixes are related to the air voids. Generally, the higher the air voids the higher the asphalt permeability which in turn results in the asphalt layer being more susceptible to moisture damage.

Inadequate compaction of asphalt mix is probably the most common construction-related factor leading to premature stripping. Studies have shown that less than 4 to 5% air voids content are generally not interconnected and thus almost impervious to water. Most of the dense graded asphalt mixes are designed to have 3 to 5% air voids.

US state road agencies specify a maximum in situ air void content of 8%. It is assumed that the pavement will be densified to the design air void content under 2 to 3 years traffic (Kandhal 1992). Poor compaction control results in higher than 8% air voids content at the time of construction. This may cause premature surface ravelling due to lack of cohesion.

Vardanega et al. (2008) investigated the factors that influence permeability of dense graded asphalt pavements. The findings show (Figure 3.5) that the major factors affecting asphalt permeability are air voids, lift thickness, mix type and grading. Generally, the permeability is reduced for finer mixes and increased for coarser mixes.

Figure 3.5: Relationship between permeability and air voids



Source: Vardanega et al. (2008).

Figure 3.5 shows an asphalt mix having low permeability at low air voids (i.e. around 8 x 10^{-6} m/s at 4.5% air voids). Permeability increases with the increase in air voids and the asphalt mix becomes moderately free draining at 8% air voids with a corresponding permeability of 1 x 10^{-4} m/s. Therefore, it is crucial to adopt best practice during construction to achieve the desired air voids content, particularly at construction joints.

Taylor and Khosla (1983) recommended the following for both design and construction of asphalt pavements to reduce the potential for stripping and other moisture-related failures:

- Incorporate the immersion-mechanical test into the mix design procedure in order to consider the loss of pavement strength and stiffness due to exposure to moisture and reduce the potential for stripping.
- Consider full depth asphalt (deep strength) design with well-compacted dense graded asphalt mixes for intermediate and base courses.
- Provide appropriate surface and subsurface drainage as well as maintain a well-sealed surface of the wearing course to minimise infiltration of surface water.
- Select bitumen grades that wet the aggregate thoroughly during mixing but will have viscosity in service that is as high as practical for other mix consideration.
- Use as high a bitumen content as is practical to meet stability and flow requirements. Thick bitumen films can best be accommodated by selecting an aggregate grading that provides a high percentage of voids in the mineral aggregate (VMA) after compaction.
- Avoid hydrophilic aggregates and use hydrated lime or a heat-stable surface-active agent in an amount determined by mix design.

Hydrated lime can be used to modify the aggregate surfaces by adding the lime directly to wet aggregate to rectify adverse adhesive properties of aggregates towards bitumen e.g. acidic aggregate (Bagampadde et al. 2004). The reason for this is that both anionic and cationic surfactants naturally present in the bitumen strongly bond with calcium ions while only cationic surfactants strongly bond with silica atoms (Curtis et al. 1993). Consequently, anionic surfactants are easily displaced by water on siliceous aggregate. Moreover, hydrated lime is highly effective in improving the resistance to moisture damage of aggregate with clay contamination (Lesueur et al. 2013).

4 Comparison Between Australian State Road and Transport Agencies Practices

A full depth asphalt (FDA) pavement typically comprises an asphalt basecourse, intermediate course and wearing course placed and compacted in layers on an unbound subbase or improved, natural or fill subgrade. Engineering Road Note 9 (Main Roads 2013a) does not allow the use of asphalt base mixes under the intermediate asphalt course to prevent moisture accumulation in the base layer due to concerns that this layer may be impermeable and may inhibit moisture from draining from the asphalt intermediate course. Therefore, FDA pavements in WA generally comprise asphalt intermediate and wearing courses over unbound granular basecourses, whereas Transport for New South Wales (TfNSW) full depth asphalt pavement comprises an asphalt basecourse overlying selected material zone over the subgrade. Most Australian jurisdictions have their own specifications for asphalt supply and pavement construction. Austroads also published a specification framework for asphalt mixes in the 2014 version of the *Guide to Pavement Technology Part 4B: Asphalt* (Austroads 2014). Asphalt pavement design details are covered in the *Guide to Pavement Technology Part 2: Pavement Structural Design* (Austroads 2017) and SRTA design supplements.

FDA pavements are mainly composed of dense graded asphalt (DGA) in the structural base layer with dense graded asphalt (DGA), stone mastic asphalt (SMA), open graded asphalt (OGA) and gap graded asphalt (GGA) as wearing courses depending upon performance requirements. High modulus asphalt (EME2) may be used as the base layer of heavy-duty pavements. The use of crumb rubber modified (CRM) binder in asphalt in the base layer is a recent development although Main Roads does not use CRM binder asphalt in the base layer. OGA can also be used in the basecourse as a drainage layer if required.

The specifications and guidelines for DGA and other mixes are discussed in detail below.

4.1 Dense Graded Asphalt

DGA mixes are the most widely used form of asphalt in structural layers due to their high load carrying capacity. DGA has a continuous distribution of aggregate particle sizes and fillers and a low design air voids content generally in the range of 3 to 7% (Austroads 2014).

The following DGA specifications and guidelines were reviewed and compared to those of Main Roads to gain a better understanding of the mix design, manufacturing and construction practices in Australia:

- Main Roads Specification 510 – *Asphalt Intermediate Course* (2022a)
- Main Roads Engineering Road Note 9 – *Procedure for the Design of Road Pavements* (2013a)
- Transport for New South Wales QA Specification R116 – *Heavy Duty Dense Graded Asphalt* (2021)
- Transport for New South Wales QA Specification R117 – *Light Duty Dense Graded Asphalt* (2022)
- RMS *Supplement to Austroads Guide to Pavement Technology – Part 2 (2018)*
- VicRoads Code of Practice RC 500.01 – *Registration of Bituminous Mix Designs* (2021)
- Department of Transport Victoria (DoT Vic) Specification 407 – *Hot Mix Asphalt* (2021)
- VicRoads Specification 409 – *Warm Mix Asphalt* (2012)
- VicRoads Technical Note 3 – *Dense Graded Asphalt* (2006)
- VicRoads Code of Practice RC 500.22 – *Selection and Design of Pavements and Surfacings* (2018c)
- Queensland Department of Transport and Main Roads (TMR): MRTS30, *Asphalt Pavements* (2022a)
- TMR *Supplement to 'Part 2: Pavement Structural Design' of the Austroads Guide to Pavement Technology* (2021)
- DIT – Master Specification: RD-BP-S2 *Supply of Asphalt* (2022)
- DIT – Master Specification: RD-BP-C3 *Construction of Asphalt Pavement* (2021)

- DPTI *Supplement to the Austroads Guide to Pavement Technology Part 2* (2018).

4.1.1 Mix Nominal Sizes

DGA mixes are differentiated based on different nominal aggregate sizes of the mix. The sizes specified by Australian SRTAs, Austroads and IPWEA are shown in Table 4.1.

Table 4.1: Australian DGA nominal aggregate sizes

Jurisdiction	Nominal aggregate size
Main Roads(1)	5 mm, 10 mm, 14 mm and 20 mm
TfNSW(2)	5 mm, 7 mm, 10 mm, 14 mm, 20 mm and 28 mm
DoT Vic(3)	14 mm and 20 mm
TMR(4)	7 mm, 10 mm, 14 mm and 20 mm
DIT(5)	10 mm, 14 mm and 14 mm high binder (HB)
IPWEA(6)	10 mm, 14 mm and 20 mm

1. Source: Main Roads (2022a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: IPWEA (2016).

4.1.2 Mix Components

Specification requirements for the main DGA mix components are presented below.

Aggregate

Aggregates constitute around 96% of a typical asphalt mix (Austroads 2014) and are a major factor in the mix stability, shear strength and resistance to deformation.

Main Roads specifies that the aggregate, including source rock, shall meet the requirements of Main Roads Specification 511, *Materials for Bituminous Treatments* (Main Roads 2021) and shall consist of crushed material sources. Similarly, other agencies specify that the aggregate must comply with their respective specifications (e.g. QA Specification 3152 (TfNSW 2020a), Technical Specification MRTS101 (TMR 2020)). Main Roads commonly uses granite aggregate in the Perth Metropolitan area for asphalt mixes.

More details on particle size distribution or combined aggregate grading for DGA mixes are presented in Table 4.4.

Binder

Table 4.2 summarises the binder requirements for the main DGA mixes in Australia.

Table 4.2: Australian binder requirements for DGA

Jurisdiction	MIX SIZE	BINDER TYPE	BINDER CONTENT (% BY MASS)	MAXIMUM BINDER DRAIN-OFF (%)
Main Roads(1)	14 mm	A15E	10.5 v	0.3
	20 mm	C320, C600, A15E	10.0 v	
TfNSW(2)	5 mm	Not specified	5.6–6.8	0.3
	7 mm		5.4–6.6	
	10 mm		5.1–6.4	
	14 mm		4.8–6.2	
	20 mm		4.6–6.1	

Jurisdiction	MIX SIZE	BINDER TYPE	BINDER CONTENT (% BY MASS)	MAXIMUM BINDER DRAIN-OFF (%)
	28 mm		4.0–5.8	
DoT Vic(3)	14 mm	C170, C320, C600, M500, A10E,	4.5	0.3
	20 mm		4.0	
TMR(4)	7 mm	C320, M1000, A15E	≥ 11.5 v	0.3
	10 mm		≥ 11.0 v	
	14 mm		≥ 10.5 v	
	20 mm		≥ 10.0 v	
DIT(5)	10 mm	A15E, A5E, C320, C170	AS2150	0.3
	14 mm			
	14 mm HB			
IPWEA(6)	10 mm	C320(7), C170	5.0–7.0	0.3
	14 mm		4.5–6.5	
	20 mm		4.0–6.0	

1. Source: Main Roads (2022a). Binder content is expressed as % by volume of the whole mass.
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a). Binder content is expressed as % by volume of the whole mass.
5. Source: DIT (2022).
6. Source: IPWEA (2016).
7. For heavy truck traffic and greater than 2,000,000 equivalent standard axle (ESA).

Key: v: by volume of total mix.

The typical binder content for 14 mm and 20 mm DGA varies between 4.5 to 5.5% by mass or 10.0 to 11.0% by volume of total mix. A maximum binder drain-off limit of 0.3% is also specified by all road agencies, when tested in accordance with Austroads test method AGPT/T235-06 *Asphalt Binder Drain-off* (Austroads 2006a). As seen from Table 4.2, the binder content requirements specified by Main Roads are generally in line with those specified by other SRTAs in Australia. However, given the wide range of allowable binder contents, it is not known whether the approved mixes used by Main Roads have similar binder contents to those used by other SRTAs.

Mineral filler

Austroads defines mineral fillers as the proportion of mineral matter that passes the 0.075 mm sieve. This can include a portion of the coarse and fine aggregate grading, recycling of the dust produced during the manufacturing processes (baghouse fines), or added material (Austroads 2014).

SRTAs in Australia typically specify a combined filler content of between 2 and 8% for DGA mixes, however, Main Roads allows a maximum of 5.5%.

Hydrated lime

Hydrated lime is often added to asphalt as a filler to reduce the moisture susceptibility (i.e. stripping potential) of a mixture. However, hydrated lime also has a stiffening effect on binders and may increase the risk of poor mix workability and low field compaction during construction. Therefore, the hydrated lime content is usually limited to between 1.0 to 1.5% of the aggregate mass (Austroads 2013). The specification limits for hydrated lime in DGA mixes used by Australian SRTAs are summarised in Table 4.3.

It is noted that TfNSW had significant issues with stripping more than 20 years ago (AAPA 2005, Kandhal & Rickards 2001). TfNSW specifies a minimum of 1.5% hydrated lime, whereas Main Roads specifies 1.5% lime must be used.

Table 4.3: Australian hydrated lime requirements for DGA

Jurisdiction	Hydrated lime content (% by mass of total aggregate)
Main Roads[(1)]	1.5%
TfNSW[(2)]	1.5 (minimum)
DoT Vic[(3)]	No reference
TMR[(4)]	1.0 (minimum)
DIT[(5)]	1.0 (minimum)

1. Source: Main Roads (2022a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a).
5. Source: DIT (2022).

Fibre additives

Fibres are typically added to DGA to control binder drain-off. All the SRTA specifications that were reviewed specify a minimum fibre content of 0.3%. It is a standard requirement that cellulose fibres are used. DIT also allows for the use of rock wool, glass fibre and other organic sources.

An allowance is made in some specifications for the contractors to propose and use alternative fibre additives, subject to a technical review which includes the submission of documented evidence of successful use or trials undertaken.

Adhesion agent

Adhesion agents can be used to increase the physio-chemical bond between the binder and aggregate, resulting in reduced moisture sensitivity of asphalt mixes (Austroads 2014). The following observations were made regarding the use of adhesion agents in the Australian DGA specifications:

- Main Roads specifies that adhesion agents must meet the requirements in Specification 511, *Materials for Bituminous Treatments* (Main Roads 2021). The adhesion agent in asphalt mixes is typically hydrated lime. However, an approved liquid adhesion agent can be used in applications where the use of hydrated lime is not practical (such as rural regions).
- The current DoT Vic Code of Practice RC 500.01 (VicRoads 2021) does not make any reference to adhesion agents.
- TfNSW, TMR and DIT allow for the use of adhesion agents in DGA mixes.

4.1.3 Particle Size Distribution

Table 4.4 summarises the combined aggregate grading specification requirements for DGA mix components.

TfNSW and TMR have similar particle size distribution (PSD) for 14 mm and 20 mm DGA. TfNSW is the only jurisdiction that has a 28 mm specification and TfNSW and TMR are the only jurisdictions that have specifications for 7 mm DGA; 7 mm and 10 mm DGA mixes are used in dense graded wearing courses whereas basecourses and intermediate courses are generally composed of 14 mm and 20 mm DGA. As Main Roads only uses intermediate course asphalt, size 28 mm mixes are not used. Gradings specified by different jurisdictions are in similar ranges.

Table 4.4: DGA grading requirements in Australia

AS sieve size (mm)	Main Roads(1)		TfNSW(2)						DoT Vic(3)				TMR(4)				DIT(5)		IPWEA(6)	
	Percentage passing sieve size (%)																			
	14 mm	20 mm	5 mm	7 mm	10 mm	14 mm	20 mm	28 mm	7 mm	10 mm	14 mm	20 mm	7 mm	10 mm	14 mm	20 mm	10 mm	14 mm	14 mm	20 mm
37.5	–	–	–	–	–	–	–	100	–	–	–	–	–	–	–	–	–	–	–	–
26.5	100	100	–	–	–	–	100	80–98	–	–	–	–	–	–	–	100	–	–	–	100
19.0	100	report(8)	–	–	–	100	80–98	#	–	–	100	90–100	–	–	100	80–100	–	100	100	90–100
13.2	report	report	–	–	100	80–89	65–93	50–80	–	100	85–100	75–88	–	100	80–100	65–93	100	80–92	85–100	75–90
9.5	report	report	–	100	80–98	#(7)	#	#	100	90–100	70–84	61–75	100	80–100	#	#	80–92	67–83	70–85	60–80
6.7	report	report	100	80–98	65–90	55–80	45–70	35–60	80–100	70–86	59–73	49–64	80–100	65–90	55–80	45–70	66–82	54–70	62–75	50–70
4.75	report	report	80–98	#	#	#	#	#	70–88	58–73	48–65	41–55	#	#	40–65	#	52–70	43–60	53–70	40–60
2.36	report	report	50–80	45–65	35–65	25–45	20–40	15–40	46–65	38–55	32–48	27–41	45–65	35–65	25–45	20–40	34–48	28–42	35–52	25–43
1.18	report	report	#	#	#	#	#	#	31–51	25–44	22–37	18–33	#	#	#	#	21–34	19–30	24–40	18–35
0.600	report	report	15–45	15–40	15–35	10–30	5–25	5–25	20–40	16–34	14–28	12–25	15–40	15–35	10–30	5–25	14–24	12–21	15–30	14–27
0.300	report	report	#	#	#	#	#	#	13–29	10–24	10–22	8–19	#	#	#	#	8–17	7–16	10–24	9–21
0.150	report	report	#	#	#	#	#	#	8–17	6–16	6–14	5–13	#	#	#	#	5–11	6–10	7–16	6–15
0.075	Max. 5.5	Max. 5.5	3–11	3–11	3–11	2–8	2–8	2–7	5–8	4–7	4–7	3–6	3–11	3–11	2–8	2–8	4–7	3–6	4–7	3–7

1. Source: Main Roads (2022a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: IPWEA (2016).
7. Where PSD is shown as #, state the value of the PSD limits in the mix design.
8. Report denotes the test results report.

4.1.4 DGA Volumetric Properties

The DGA mix design requirements vary between the various specifications reviewed. The laboratory compaction and volumetric requirements specified for DGA mixes in Australia are summarised in Table 4.5.

Table 4.5: Australian laboratory compaction and volumetric requirements for DGA mixes

Jurisdiction	Mix size	Laboratory compaction level	Laboratory compacted air voids (%)		VMA (%)	FBR	BFI
			Min.	Max.	Min.		
Main Roads[1]	14 mm	75 blows (Marshall)	3.5	5.5	14	–	≥ 8.0
	20 mm						
TfNSW[2]	14 mm	120 cycles (Gyratory)	3	6	15	0.8–1.2	> 7.5
		350 cycles (Gyratory)	2.0	N/A			
	20 mm	120 cycles (Gyratory)	3	6	14		
		350 cycles (Gyratory)	2	N/A			
DoT Vic[3]	14 mm	50 blows (Marshall)	4.9	5.3	15	–	–
	20 mm				14		9.5
	20 mm	80 cycles (Gyratory)	2	N/A	N/A		–
TMR[4]	14 mm	50 blows (Marshall) or (120 and 350 cycles (Gyratory))	3.0 (2.0)	6.0 (N/A)	N/A	1.0–1.3	> 7.5
	20 mm	50 blows (Marshall) or (120 and 350 cycles (Gyratory))			N/A		
DIT[5]	10 mm	50 and 80 cycles (Gyratory)	4.0		N/A	–	9.5, 8.5
	14 mm	50 and 80 cycles (Gyratory)	4.0		N/A		–
IPWEA[6]	14 mm	50 blows (Marshall)	4.0	6.0	14	–	≥ 7.0
	20 mm	80 blows (Marshall)					

1. Source: Main Roads (2022a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: IPWEA (2016).

Key: FBR: Filler/binder ratio, BFI: Binder film index.

TfNSW and DIT specify gyratory compaction for the design of their DGA mixes, whereas all the other road agencies specify a Marshall compaction level. The air voids limits for a 14 mm and 20 mm DGA specified by IPWEA and DIT have a lower limit of 4.0% and an upper limit of 6.0% whereas these limits for Main Roads, TMR and TfNSW vary between a lower limit of 3.0 to 3.5% and an upper limit of 5.5 to 6.0%.

4.1.5 DGA Mix Performance-related Requirements

Table 4.6 summarises the performance-related requirements for DGA mixes specified by Australian agencies.

Table 4.6: Australian performance-related requirements for DGA

Jurisdiction	Property	Test method	Requirements
Main Roads[1]	Stability (kN) Flow (mm)	WA 731.1-2018 (Main Roads 2018a)	8 (min.) 2–4
	Deformation resistance (mm)	AGPT/T231-06 (Austroads 2006b)	4 (max.)
	Tensile strength ratio (TSR, %)	AGPT-T232-07 (Austroads 2007a)	80% (min.)
TfNSW[2]	Tensile strength ratio (TSR, %)	RMS T640 (RMS 2012c)	80% (min.)
DoT Vic[3]	Tensile strength ratio (TSR, %) Indirect tensile modulus (MPa) Deformation resistance (mm)	AG:PT/T232-07 (Austroads 2007a) AS/NZS 2891.13.1 AGPT-T231-06 (Austroads 2006b)	80% (min.) 3,000–7,000 4 (max.)
TMR[4]	Stability (kN) Flow (mm) Stiffness Tensile strength ratio (TSR, %) Resilient modulus (MPa) Deformation resistance (mm)	Q305 (TMR 2022c) Q305 (TMR 2022c) Q305 (TMR 2022c) Q315 (TMR 2022d) or AGPT/T232-07 (Austroads 2007a) AS/NZS 2891.13.1:2013 AGPT-T231-06 (Austroads 2006b)	7.5 (min.) 2 (min.) 2.0 kN/mm 80% (min.) Report only 4.5 mm (max.) 3.5 (max.) for PMB
DIT[5]	Indirect tensile strength (kPa) Resilient modulus (MPa) Deformation resistance (mm) Flexural fatigue (min. microstrain at 1 million cycles)	DPTI:TP460-2013 (DPTI 2013) AS/NZS 2891.13.1:2013 AGPT-T231-06 (Austroads 2006b) DPTI:TP477-2015 (DPTI 2015)	Report only 2,400–6,600 3.0–6.0 170 (AC14M320) 150 (AC20M320)
IPWEA[6]	Stability (kN)	N/A	6.5 (50 blows) 8.0 (80 blows)
	Flow (mm)	N/A	2–4

1. Source: Main Roads (2022a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: IPWEA (2016).

There is not currently a harmonised approach to specifying performance-related properties for DGA mixes in Australia. Deformation resistance is, however, the most commonly specified property.

Moisture sensitivity tests are adopted in the mix design by different SRTAs. Technical Note 3 (VicRoads 2006) specifies moisture sensitivity tests used to measure the sensitivity of a mix to damage by stripping of binder from aggregates under saturated conditions. The test requires indirect tensile strength of both dry and moisture-conditioned test specimens to be determined. The moisture sensitivity is expressed as the wet to dry tensile strength ratio (TSR) as a percentage. The higher the TSR the less moisture sensitive the mix should be. The test is generally only applied to heavy duty wearing and intermediate course mixes where moisture damage is a risk.

Main Roads, TMR and TfNSW also specify TSR tests for moisture sensitivity as follows:

- Main Roads Specification 510 – *Asphalt Intermediate Course* (Main Roads 2022a) using test method AGPT/T232 (including freeze/thaw effect) (Austroads 2007a).
- TMR Specification MRTS30 – *Asphalt Pavements* (TMR 2022a) using test methods TMR Q315 or AGPT/T232 (including freeze/thaw effect) (Austroads 2007a).
- TfNSW QA Specification R116 – *Heavy Duty Dense Graded Asphalt* (TfNSW 2021) using RMS test method T640 (moulded asphalt) (RMS 2012c) and T649 (cores) (RMS 2012a) to assess the propensity for moisture damage in asphalt (including freeze/thaw effect).

Importantly, TfNSW specifications are evaluated using plant-mixed asphalt whereas Main Roads specifies laboratory-manufactured mixes.

4.1.6 DGA Field Compaction Requirements

Table 4.7 summarises the field compaction requirements for DGA specified in Australia.

Table 4.7: Australian field compaction requirements for DGA

Jurisdiction	Mix size	In situ air voids (%)[1]		Density ratio (%)[2]
		Min.	Max.	Min.
Main Roads[3]	14 mm and 20 mm	3	6	Not specified
TfNSW[4]	14 mm and 20 mm	3	7	Not specified
DoT Vic[5]	14 mm and 20 mm	Not specified	Not specified	96% characteristic value of Marshall density or 97% mean value of density ratio [9]
TMR[6]	14 mm and 20 mm surface layer Covered by 2 AC layers Covered by 3 AC layers	 3 2.5 2	 7 7 7	Not specified
DIT[7]	10 mm	4	8	Not specified
	14 mm	2.5	7	Not specified
IPWEA[8]	14 mm and 20 mm	1.5	7	Not Specified

1. Based on characteristic values.
2. Ratio between the bulk density of field cores and Marshall density.
3. Source: Main Roads (2022a). Maximum characteristic value for in situ air voids is relaxed to 7% to allow for asphalt suppliers to implement new asphalt mix designs and construction practices.
4. Source: TfNSW (2021). Upper limit for characteristic values of in situ air voids is 8% for specified layer thickness of > 30 mm and < 50 mm.
5. Source: VicRoads (2021).
6. Source: TMR (2022a).
7. Source: DIT (2021).
8. Source: IPWEA (2016).
9. Characteristic value used where 6 or more tests are available.

A review of the compaction requirements indicates that Main Roads, TfNSW, TMR and IPWEA specify a minimum and maximum limit for the characteristic value of in situ air voids of the compacted DGA layer. DoT Vic and Main Roads are the only jurisdictions that specify a density ratio which is defined as the ratio between the compacted in situ field density and the Marshall bulk density of an asphalt specimen determined in the laboratory. This ratio can, however, not be directly related to an in situ air void content without first determining the maximum theoretical density of the DGA mix in the laboratory. The maximum in situ air void content specified by Main Roads, TfNSW, TMR, DIT and IPWEA varies between 6 to 7%.

Main Roads is the only jurisdiction that tests for moisture content of asphalt cores to determine the in situ moisture content.

4.1.7 Pavement Design and Construction Considerations

Design considerations for DGA pavements for jurisdictions in Australia were reviewed. Table 4.8 summarises the specified ranges of layer thicknesses for intermediate and basecourses for DGA and Table 4.9 the asphalt design modulus values.

Table 4.8: Australian DGA allowable layer thickness ranges (intermediate and basecourse)

Jurisdiction	Allowable layer thickness for different nominal asphalt sizes (mm)	
	Size 14 mm mixes	Size 20 mm mixes
Main Roads[2]	50[1]	60–100
TfNSW[3]	42–70	60–100
DoT Vic[4]	35–50	50–100
TMR[5]	50–70	60–100

Jurisdiction	Allowable layer thickness for different nominal asphalt sizes (mm)	
	Size 14 mm mixes	Size 20 mm mixes
DIT[6]	50–80	–
Austroads[7]	35–50	50–100
IPWEA[8]	Minimum of 3 to a maximum of 5 times the maximum nominal aggregate size	

1. Asphalt nominal total thickness.
2. Source: Main Roads (2013a).
3. Source: RMS (2018).
4. Source: VicRoads (2018a).
5. Source: TMR (2022a).
6. Source: DIT (2021).
7. Source: Austroads (2017).
8. Source: IPWEA (2016).

Table 4.9: Australian DGA asphalt design modulus requirements

Jurisdiction	Method to determine asphalt design modulus	Asphalt modulus determination test method
Main Roads[5]	Lower than the lower of [1,2 & 3]	AS/NZS 2891.13.1-2013
TfNSW[6]	Derived from Shell nomographs or laboratory determined, calculated based on weighted mean annual pavement temperature (WMAPT), heavy vehicle (HV) design speed and in-service air voids[4]	AS/NZS 2891.13.1-2013 or AGPT-274-16 (Austroads 2016)
DoT Vic[7]	DoT Vic Code of Practice RC 500.22 (2018c) – Appendix E	
TMR[8]	TMR Pavement Design Supplement (2021) – Table 6.5.7 (a) A New Approach to Asphalt Pavement Design	AS/NZS 2891.13.1-1995
DIT[9]	DPTI Supplement to AGPT Part 2 Table 6.18	Equation – Jameson (2005)

1. DGA modulus values in Table 6.13 of the *Guide to Pavement Technology Part 2* (Austroads 2012).
2. When 3 to 9 tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the lowest individual result obtained; or when 10 or more tests are undertaken on the mix to be used at the combination of particle size distribution and binder content that results in the greatest pavement thickness, the value that 85% of test results are higher than.
3. For 10 mm Perth DGA with C170 binder the indirect tensile asphalt modulus used must exceed 5,000 MPa and for 14 mm Perth intersection mix with C320 binder the asphalt modulus used must exceed 5,500 MPa.
4. Minimum 1,000 MPa, maximum 4,000 MPa.
5. Source: Main Roads (2013a).
6. Source: RMS (2018).
7. Source: VicRoads (2018a).
8. Source: TMR (2022a & 2021).
9. Source: DPTI (2018).

It is noted that:

- Main Roads, TfNSW, TMR and DIT supplements to the *Guide to Pavement Technology Part 2* (Austroads 2017) require the heavy vehicle design speeds to be used in the mechanistic-empirical design procedure to determine asphalt modulus for various posted speed limits and longitudinal grades. VicRoads (2018a) specifies reduced pavement design speeds based on posted speed limits and the asphalt design modulus at WMAPT of 24 °C.
- TfNSW, DoT Vic and TMR specify a minimum thickness of 175 mm for DGA over a cemented layer which is in line with the *Guide to Pavement Technology Part 2*. DIT specifies a minimum total asphalt thickness equivalent to 100 to 175 mm over cemented subbases.
- Main Roads, TMR and TfNSW specify a 10 mm construction tolerance in its mechanistic-empirical design procedure in order to minimise the adverse effects of as-constructed variance levels on asphalt fatigue life.

Table 4.10 summarises the asphalt manufacturing and compaction temperatures specified in Australia.

Table 4.10: **Australian asphalt manufacturing and field compaction temperature requirements**

Jurisdiction	Asphalt temperatures (°C)	
	Manufacturing temperature (°C)	Field compaction temperature (°C)
Main Roads[1]	170 (C320, Sasobit) 175 (C600) 185 (PMB) 130 warm mix asphalt (WMA)	–
TfNSW[2]	Max. 175	As per project quality plan (PQP)
VicRoads[3]	Max. 175	
TMR[4]	175 (bitumen and multigrade binders) 185 (PMB)	(≤ 30 mm) DGA 115 OGA 120
DIT[5]	180 (C170) 185 (C320) 195 (C600)	≥ 120

1. Source: Main Roads (2013a).
2. Source: TfNSW (2021).
3. Source: VicRoads (2021).
4. Source: TMR (2021).
5. Source: DIT (2022) & DPTI (2018).

All the agencies specify pavement temperatures and weather conditions based on wind speed and nominal aggregate size for asphalt placement. Main Roads (2022a) specifies asphalt delivery temperatures measured at the discharge point for 14 mm and 20 mm nominal size asphalt mixes, namely:

- 140–170 °C (C320)
- 150–175 °C (C600)
- 160–185 °C (A15E)
- 125–155 °C (WMA, Sasobit).

DIT also specifies delivery temperatures as provided in AS 2150-Table 12.

4.1.8 Tack Coat and Waterproofing Seal

Main Roads specifies:

- A sprayed bituminous seal on the uppermost layer of the size 14 mm asphalt intermediate course and a tack coat of the diluted emulsion on the prepared surface for the placement of overlying asphalt layers.
- A 300 mm wide tack coat on longitudinal joints and at the end of the previous run (before the paving of the next run proceeds) for transverse joints.
- CRS/170-60 or CSS/170-60 for the tack coat during the preparation of the surface prior to the laying of asphalt with an application rate of 0.6 litres/m^2 of the diluted emulsion.
- A prime or sprayed seal between the thin (≤ 50 mm) asphalt surfacing and the granular base to waterproof the granular base and improve adhesion of the thin asphalt layer to the base.

TfNSW specifies:

- A prime or sprayed seal between the thin (≤ 50 mm) asphalt surfacing and the granular base to waterproof the granular base and improve adhesion of the thin asphalt layer to the base.
- A low cutter seal for the thicker (> 50 mm) asphalt layers over granular base.
- An application rate between 0.15 to 0.30 litres/m^2 for the tack coat.

TMR specifies an application rate between 0.10 to 0.30 litre/m^2 of residual binder for the tack coat.

TfNSW, TMR and DoT Vic specify CRS/170-60 for the tack coat and double the application rate on joints and chases. DIT specifies CRS grade emulsion for the tack coat at vertical edges between old and new asphalt pavements, on the top of existing asphalt layers and on the top of new asphalt not placed on the same day to ensure an adequate bond between pavement layers.

4.1.9 High Modulus Asphalt (EME2)

EME2 is a high modulus asphalt for use in heavy duty pavements and high stress locations such as at traffic lights, climbing lanes and roundabouts.

Among Australian road agencies, only TfNSW, DoT Vic and TMR have documented specifications for EME2 mixes. The following specifications have been reviewed:

- Transport for New South Wales QA Specification R126 – *High Modulus Asphalt (EME2)* (2020b)
- DoT Vic Specification 418 – *High Modulus Asphalt (EME2)* (2016)
- TMR: MRTS32, *High Modulus Asphalt (EME2)* (2022b).

EME2 asphalt properties and requirements specified by the 3 agencies are summarised in Table 4.11.

Table 4.11: EME2 asphalt properties and requirements

Properties	TfNSW(1)	VicRoads(2)	TMR(3)
Nominal size (mm)	14	Not specified	14
Binder type	15/25 pen	15/25 or 10/20 pen	15/25 or 10/20 pen
Binder volume (%)	13.5	13.3	13.5
Hydrated lime (%)	Optional	Not specified	2,000–4,200
Adhesion agent (%)	Max. 1.0	Not specified	Not specified
Air voids in laboratory compacted mix, at 100 cycles (%)	Max. 6.0	Max. 6.0	Max. 6.0
Layer thickness (mm)	70–130	70–130	70–130
Modulus (MPa)	6,000	2,800–6,000	Report
Flexural stiffness (MPa)	Min. 14,000	Min. 14,000	Min. 14,000
Fatigue resistance (με)	Min. 150	Min. 150	Min. 150
Deformation resistance (mm) At 60°C and 30,000 cycles At 60°C and 5,000 cycles	 Max. 4 Max. 2	Max. 6	 Max. 4 Max. 2
Moisture sensitivity – tensile strength ratio (TSR, %)	Min. 80	Not specified	
Richness modulus	Min. 3.4	Min. 3.4	Min. 3.4
Characteristic value of density ratio (6 tests), (%)	Not specified	Min. 98.0	Not specified
Characteristic value of in situ air voids (%)	Max. 5.5	Not specified	Max. 5.5

1. Source: TfNSW (2020b) & RMS (2018).
2. Source: VicRoads (2016 & 2018a).
3. Source: TMR (2022b).

As shown in Table 4.11, the agencies have almost the same specification requirements for EME2 mixes. Most of the specifications are based on the *EME2 Model Specification* (AAPA 2018).

4.2 Stone Mastic Asphalt

Stone mastic asphalt (SMA) is a coarse gap-graded asphalt mix with stone-on-stone contact between the coarse aggregate as a primary contributor to the stability of the compacted layer (Austroads 2014). It has around 50% extra coarse aggregate than a typical DGA mix (Main Roads 2016). The voids in the coarse aggregate skeleton are filled with a mastic of binder, filler and fine aggregate that improves the durability of the asphalt mix because of increased cohesion and reduced moisture sensitivity and improved fatigue characteristics (Kreide et al. 2003).

The following SMA specifications and guidelines were reviewed and compared to Main Roads specifications:

- Main Roads Specification 502 – *Stone Mastic Asphalt (SMA)* (2022b)
- Main Roads Engineering Road Note 10 – *Stone Mastic Asphalt* (2016)
- Transport for New South Wales QA Specification R121 – *Stone Mastic Asphalt* (2020c)
- DoT Vic Specification 404 – *Stone Mastic Asphalt* (2018a)
- TMR: MRTS30, *Asphalt Pavements* (2022a)
- DIT – Master Specification: RD-BP-S2 *Supply of Asphalt* (2022)
- DIT – Master Specification: RD-BP-C3 *Construction of Asphalt Pavements* (2021)
- DPTI *Supplement to Austroads Guide to Pavement Technology Part 2*, 2018.
- Austroads *Guide to Pavement Technology Part 4B (2007b) (superseded)*

4.2.1 Mix Nominal Sizes

SMA mixes are differentiated based on the nominal aggregate size of the mix. The different sizes specified by SRTAs, Austroads and IPWEA are shown in Table 4.12.

Table 4.12: Australian SMA mix sizes

Jurisdiction	Mix sizes
Main Roads(1)	7 mm and 10 mm
TfNSW(2)	10 mm and 14 mm
VicRoads(3)	10 mm
TMR(4)	10 mm and 14 mm
DIT(5)	7 mm and 10 mm
Austroads(6)	7 mm, 10 mm and 14 mm
IPWEA(7)	5 mm, 7 mm, 10 mm and 14 mm

1. Source: Main Roads (2022b).
2. Source: TfNSW (2020c).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: Austroads (2007b).
7. Source: IPWEA (2016).

4.2.2 Mix Components

Specification requirements for the main SMA mix components are presented below.

Aggregate

Requirements for the aggregate of SMA mixes are the same as described in Section 4.1.2 in relation to DGA mixes for Main Roads and other SRTAs.

More details on particle size distribution or combined aggregate grading for SMA mixes are presented in Table 4.15.

Binder

Table 4.13 summarises the binder requirements for the main SMA mixes used in Australia.

Table 4.13: Australian binder requirements for SMA mixes

Jurisdiction	Mix size	Binder type	Binder content (% by mass)	Maximum binder drain-off (%)
Main Roads(1)	7 mm	A20E	6.5–7.5	0.3
	10 mm		6.0–7.0	
TfNSW(2)	10 mm	PMB or multigrade binder	6.2–7.2	0.3

Jurisdiction	Mix size	Binder type	Binder content (% by mass)	Maximum binder drain-off (%)
	14 mm		6.0–7.0	
DoT Vic(3)	10 mm (normal)	A15E, A20E or A25E	6.5–7.5	0.3
	10 mm (heavy duty)	A10E	6.0–7.0	0.3
TMR(4)	10 mm	A15E	≥ 14.5	0.3
	14 mm		≥ 13.5	
DIT(5)	7 mm	A15E or A5E	7.0 (target)	0.3
	10 mm		6.5 (target)	
Austroads(6)	7 mm	N/A	6.0–7.3	0.3
	10 mm		6.0–7.0	
	14 mm		5.8–6.8	
IPWEA(7)	5 mm	C320	6.0–8.0	0.3
	7 mm		6.0–8.0	
	10 mm		6.0–8.0	
	14 mm		5.5–7.5	

1. Source: Main Roads (2022b).
2. Source: TfNSW (2020c).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: Austroads (2007b).
7. Source: IPWEA (2016).

The binder type specified by each jurisdiction is as follows:

- Main Roads, DoT Vic, TMR and DIT specify a polymer modified binder (PMB).
- TfNSW specifies that either a PMB or multigrade binder must be used.

The typical binder content for 7 mm and 10 mm SMA varies between 6.0 to 8.0%. A maximum binder drain-off limit of 0.3% is also specified by all road agencies, when tested in accordance with Austroads test method AGPT/T235-06 *Asphalt Binder Drain-off* (Austroads 2006a). Table 4.13 shows that the binder content requirements specified by Main Roads are generally in line with those specified by the other SRTAs.

Mineral filler

SRTAs typically specify a combined filler content of between 8% and 12% for SMA mixes, except for TMR that specifies between 6.5% and 12.5%.

Hydrated lime

Hydrated lime is often added. The specification limits for hydrated lime in SMA mixes are summarised in Table 4.14.

Table 4.14: Australian lime content requirements for SMA mixes

Jurisdiction	Hydrated lime content (% by mass of total aggregate)
Main Roads(3)	1.5%(1)
TfNSW(4)	1.5%(2) (Minimum)
DoT Vic(5)	No reference
TMR(6)	1% (Minimum)
DIT(7)	1% (Minimum)

1. For both 7 mm and 10 mm SMA mixes used in Perth.
2. Only if the combined filler (excluding hydrated lime) has a methylene blue value of between 10 mg/g and 18 mg/g. The methylene blue value of the filler is determined in accordance with AS 1141.66-2012 and is an indication of the amount and type of clay in the filler component that could be detrimental to the moisture resistance of asphalt mixes.
3. Source: Main Roads (2022b).
4. Source: TfNSW (2020c).
5. Source: VicRoads (2018b).
6. Source: TMR (2022a).
7. Source: DIT (2022).

Fibre additives

The requirements for the fibres that are typically added to SMA to control binder drain-off are the same for Main Roads and the other SRTAs as described in Section 4.1.2 for DGA mixes.

Adhesion agent

The observations made regarding the use of adhesion agents in the Australian SMA specifications reviewed are the same as described in Section 4.1.2 for DGA mixes.

4.2.3 Particle Size Distribution

Table 4.15 summarises the combined aggregate grading specification requirements for SMA mix components used in Australia.

Table 4.15: Australian SMA particle size distribution requirements

AS sieve size (mm)	Main Roads(1)		tfNSW(2)		DoT Vic(3)		TMR(4)		DiT(5)		Austroads(6)			IPWEA(7)			
	Percentage passing sieve size (%)																
	7 mm	10 mm	10 mm	14 mm	10 mm (normal)	10 mm (HD)	10 mm	14 mm	7 mm	10 mm	7 mm	10 mm	14 mm	5 mm	7 mm	10 mm	14 mm
19.0	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
13.2	100	100	100	76–100	100	100	100	84–100	100	100	100	100	90–100	100	100	100	90–100
9.5	100	90–100	80–100	31–64	90–100	90–100	85–100	40–65	100	90–100	100	90–100	30–40	100	100	90–100	30–40
6.7	90–100	25–40	31–64	16–44	45–65	25–45	40–62	25–45	85–100	30–55	90–100	25–40	20–30	100	90–100	25–40	20–30
4.75	25–40	18–30	16–44	14–36	30–50	18–32	25–45	18–32	30–62	20–40	25–45	18–30	18–30	90–100	25–45	18–30	18–30
2.36	15–28	15–28	13–31	13–31	21–31	15–30	18–31	14–28	20–35	15–28	15–28	15–28	15–28	25–40	15–28	15–28	15–28
1.18	13–24	13–24	11–27	11–27	16–25	13–24	14–28	12–24	16–28	13–24	13–24	13–24	13–24	13–24	13–24	13–24	13–24
0.600	12–21	12–21	8–24	8–24	14–22	12–21	12–24	10–20	14–24	12–21	12–21	12–21	12–21	12–21	12–21	12–21	12–21
0.300	10–18	10–18	7–21	7–21	12–19	10–18	10–20	9–17	12–20	10–18	10–18	10–18	10–18	10–18	10–18	10–18	10–18
0.150	9–14	9–14	8.5–16	8.5–16	9–15	9–15	8–17	7.5–14.5	10–16	9–14	10–16	9–24	9–14	9–14	9–14	9–14	9–14
0.075	8–12	8–12	7.5–12.5	7.5–12.5	8–12	8–12	6.5–12.5	6.5–12.5	8–12	8–12	8–12	8–12	8–12	8–12	8–12	8–12	8–12

1. Source: Main Roads (2022b).
2. Source: TfNSW (2020c).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: Austroads (2007b).
7. Source: IPWEA (2016).

Main Roads, DIT and IPWEA are the only jurisdictions that have a 7 mm SMA specification. The grading specified by DIT for a 7 mm SMA mix is finer than the gradings adopted by Main Roads and IPWEA. There is a marked difference between the grading envelopes for a 10 mm SMA mix, especially on the 4.75 mm and 6.70 mm sieve sizes.

The grading specified by Main Roads and DoT Vic (HD application) are typically coarser on the intermediate sieve sizes (i.e. 6.70 mm, 4.75 mm and 2.36 mm) compared to the grading specified by TMR, TfNSW and DIT for a 10 mm SMA mix. The grading specified by TMR and DoT Vic (normal duty application) are typically finer across all the sieve sizes compared to the other road agencies.

4.2.4 SMA Volumetric Properties

The SMA mix design requirements vary between the specifications reviewed. It is worth noting that Main Roads uses Marshall compaction for the design of its SMA mixes, whereas TfNSW and DIT use gyratory compaction. Therefore, the volumetric requirements specified by them are not necessarily comparable to Main Roads values.

Table 4.16 summarises the laboratory compaction and volumetric requirements specified for SMA mixes.

The air void limits for a 7 mm SMA specified by Main Roads, DIT, IPWEA and Austroads vary between a lower limit of 3.0 to 3.5% and an upper limit of 4.5 to 5.5%.

The air voids specified by Main Roads for a 10 mm SMA range between 3.5 to 5.5%, which is similar to the limits specified by DoT Vic (normal duty mix), DIT and IPWEA. TfNSW specifies a 1.0% higher air void content for its 10 mm SMA compared to Main Roads. TMR has the lowest minimum air void requirement (i.e. 2%) compared to the other agencies.

The minimum voids in mineral aggregate (VMA) limits specified are similar across all jurisdictions. TfNSW and TMR are the only jurisdictions that do not specify a minimum VMA value. They do however specify a maximum mix volume ratio, which is a function of the volume of air voids in the coarse aggregate of the mix.

Table 4.16: Australian laboratory compaction and volumetric requirements for SMA mixes

<table>
<tr><th rowspan="2">Jurisdiction</th><th rowspan="2">Mix size</th><th rowspan="2">Laboratory compaction level</th><th colspan="2">Laboratory compacted air voids (%)</th><th>VMA (%)</th></tr>
<tr><th>Min.</th><th>Max.</th><th>Min</th></tr>
<tr><td rowspan="2">Main Roads[(1)]</td><td>7 mm</td><td rowspan="2">50 blows (Marshall)</td><td>3.0</td><td>5.0</td><td>19</td></tr>
<tr><td>10 mm</td><td>3.5</td><td>5.5</td><td>18</td></tr>
<tr><td rowspan="4">TfNSW[(2)]</td><td rowspan="2">10 mm</td><td>120 cycles (Gyratory)</td><td>3.5</td><td>6.5</td><td rowspan="2">N/A</td></tr>
<tr><td>350 cycles (Gyratory)</td><td>2.0</td><td>N/A</td></tr>
<tr><td rowspan="2">14 mm</td><td>120 cycles (Gyratory)</td><td>3.5</td><td>6.5</td><td rowspan="2">N/A</td></tr>
<tr><td>350 cycles (Gyratory)</td><td>2.5</td><td>N/A</td></tr>
<tr><td rowspan="2">DoT Vic[(3)]</td><td>10 mm (normal)</td><td rowspan="2">50 blows (Marshall)</td><td>3.5</td><td>5.0</td><td>18</td></tr>
<tr><td>10 mm (HD)</td><td>4.8</td><td>5.2</td><td>18</td></tr>
</table>

			Laboratory compacted air voids (%)		VMA (%)
Jurisdiction	**Mix size**	**Laboratory compaction level**	**Min.**	**Max.**	**Min**
TMR[4]	10 mm	50 blows (Marshall)	2	5	N/A
	14 mm	50 blows (Marshall)			N/A
DIT[5]	7 mm	80 cycles (Gyratory)	3.5		N/A
	10 mm	80 cycles (Gyratory)			N/A
Austroads[6]	7 mm	50 blows (Marshall – normal/medium duty) or 80 cycles Gyratory normal/medium duty) 75 blows (Marshall – HD) or 120 cycles Gyratory – HD	3.5	4.5	19
	10 mm		3.5	4.5	18
	14 mm		3.5	4.5	17
IPWEA[7]	7 mm, 10 mm and 14 mm	50 blows (Marshall) or 80 cycles (Gyratory)	3.5	3.5	N/A

1. Source: Main Roads (2022b).
2. Source: TfNSW (2020c).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: Austroads (2007b).
7. Source: IPWEA (2016).

4.2.5 SMA Mix Performance-related Requirements

Table 4.17 summarises the performance-related mix design requirements specified by the jurisdictions. Importantly, moisture sensitivity requirements are not specified for SMA, presumably because these mixes are perceived to be more impermeable than DGA.

Table 4.17: Australian performance-related mix design requirements for SMA mixes

Jurisdiction	Property	Test method	Requirements
Main Roads[1]	Stability (kN) Flow (mm)	WA 731.1-2018 (Main Roads 2018a)	6 (min.) 2–5
TfNSW[2]	Deformation resistance (mm)	AGPT/T231-06 (Austroads 2006b)	2.5 (max.)
DoT Vic[3]	Stability (kN) Resilient modulus (MPa)	AS/NZS 2891.5:2015 AS/NZS 2891.13.1:2013	5.5 (min.) Report only
TMR[4]	Resilient modulus (MPa) Deformation resistance (mm)	AS/NZS 2891.13.1:2013 AGPT/T231-06 (Austroads 2006b)	Report only 2.0 (max.)
DIT[5]	Indirect tensile strength Deformation resistance (mm) Flexural fatigue (min. microstrain at 1 million cycles)	DPTI:TP460-2013 (DPTI 2013) AGPT/T231-06 (Austroads 2006b) DPTI:TP477-2015 (DPTI 2015)	Report only 3.0 (max.) 350 (SMA 10M15E) 250 (SMA 10M5EP)
	Resilient modulus (MPa)	AS/NZS 2891.13.1:2013	1,000–3,000
			4,000–6,000
IPWEA[6]	Cantabro abrasion loss (%)	Not specified	25 (unconditioned) 35 (conditioned)

1. Source: Main Roads (2022b).
2. Source: TfNSW (2020c).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Source: IPWEA (2016).

There is not currently a harmonised approach to specifying performance-related properties for SMA mixes. Deformation resistance is the most specified performance property.

4.2.6 SMA Field Compaction Requirements

Table 4.18 summarises the SMA field compaction requirements specified by the jurisdictions.

A review of the requirements indicates that there is not currently a harmonised approach to specifying field compaction of SMA mixes:

- TfNSW, TMR and DIT specify minimum and maximum limits for the in situ air voids of the compacted SMA layer.
- Main Roads, DoT Vic and IPWEA specify a minimum density ratio based on Marshall compaction. The density ratio is defined as the ratio between the compacted in situ field density and the Marshall density of an asphalt specimen determined in the laboratory. This ratio cannot, however, be directly related to an in situ air void content without first determining the maximum theoretical density of the SMA mix in the laboratory.

The maximum in situ air void content specified by TfNSW, TMR and DIT varies between 6.0 to 7.0%. This is the upper desirable limit for SMA. An upper air void content of 10% specified by IPWEA is considered very high and could potentially lead to permeable SMA layers in the field.

Table 4.18: Australian field compaction requirements for SMA mixes

Jurisdiction	Mix size	In situ air voids (%)(1)		Density ratio (%)(2)
		Min.	Max.	Min.
Main Roads(3)	7 mm and 10 mm	Not specified	Not specified	95% characteristic value of Marshall density
TfNSW(4)	10 mm and 14 mm	3	7	Not specified
DoT Vic(5)	10 mm (normal or heavy duty)	Not specified	Not specified	96% characteristic value of Marshall density or 97.5% mean value of density ratio(9)
TMR(6)	10 mm	2	7	Not specified
	14 mm	2	6	
DIT(7)	7 mm and 10 mm	1	5	Not specified
IPWEA(8)	7 mm, 10 mm and 14 mm	3.5	10	Not specified

1. Based on characteristic values.
2. Ratio between the bulk density of field cores and Marshall density.
3. Source: Main Roads (2022b).
4. Source: TfNSW (2020c).
5. Source: VicRoads (2018b).
6. Source: TMR (2022a).
7. Source: DIT (2021).
8. Source: IPWEA (2016).
9. Characteristic value used where 6 or more tests are available.

4.2.7 Pavement Design and Construction Considerations

Pavement design considerations for pavements with SMA layers for the jurisdictions were reviewed. Table 4.19 summarises the specified range of layer thicknesses for SMA mixes.

Table 4.19: SMA specified layer thickness range

Jurisdiction	Allowable layer thickness for different SMA mix sizes (mm)		
	7 mm	10 mm	14 mm
TfNSW(1)	–	30–50	42–70
DoT Vic(2)	–	35	
TMR(3)	–	35–40	50–60
Austroads(4)	20–30	–	–
DIT(5)	–	40(7)	–
IPWEA(6)	Minimum of 4 to a maximum of 6 times the maximum nominal aggregate size		

1. Source: TfNSW (2020c).
2. Source: VicRoads (2018b).
3. Source: TMR (2022a).
4. Source: Austroads (2017).
5. Source: DPTI (2018).
6. Source: IPWEA (2016).
7. This minimum thickness applies when a waterproofing spray seal interlayer is placed below the SMA. Where no spray seal is provided, a minimum SMA thickness of 45 mm shall apply.

SMA manufacturing, spreading and compaction temperature requirements are provided in the relevant specifications and are generally similar to those for DGA as outlined in Section 4.1.7.

4.2.8 Tack Coat and Waterproofing Seal

Tack coat and waterproofing seal requirements for SMA mixes are the same as described in Section 4.1.8, except TfNSW specifies an application rate between 0.15 to 0.40 litres/m^2 for the tack coat for SMA mixes.

4.3 Open Graded Asphalt

Open graded asphalt (OGA) mixes have relatively high air voids, generally in the range of 18.0 to 25.0% and their PSD is characterised by a large proportion of coarse aggregate and only a small amount of fine aggregate and filler. The stability of OGA relies largely on the mechanical interlock of aggregate particles (Austroads 2014). Due to high permeability, OGA mixes are placed on a layer that is waterproof and free draining to minimise vertical movement of moisture into the pavement and provide lateral drainage to the edges of the pavement.

OGA is mainly used as surfacing on high-speed multi-lane roads in the urban areas for lower tyre noise and greater safety by preventing aquaplaning. The following specifications and guidelines were reviewed and compared to Main Roads specifications:

- Main Roads Specification 504 – *Asphalt Wearing Course* (2022c)
- Transport for New South Wales QA Specification R119 – *Open Graded Asphalt* (2020d)
- Transport for New South Wales QA Specification R123 – *Thin Open Graded Asphalt Surfacing* (2020e)
- Roads and Maritime Services *Supplement to Austroads Guide to Pavement Technology – Part 2* (2018)
- DoT Vic Specification 417 – *Open Graded Asphalt* (2018b)
- DoT Vic Technical Note 4 – *Open Graded Asphalt* (2004)
- TMR: MRTS30, *Asphalt Pavements* (2022a)
- TMR *Supplement to 'Part 2: Pavement Structural Design' of the Austroads Guide to Pavement Technology Part 2* (2021)
- DIT – Master Specification: RD-BP-S2 *Supply of Asphalt* (2022)
- DIT – Master Specification: RD-BP-C3 *Construction of Asphalt Pavement* (2021)
- DPTI *Supplement to the Austroads Guide to Pavement Technology Part 2* (2018)

4.3.1 Mix Nominal Sizes

OGA mixes are differentiated based on the nominal aggregate size of the mix. Table 4.20 summarises the sizes specified.

Table 4.20: OGA nominal aggregate size

Jurisdiction	Nominal aggregate size
Main Roads(1)	10 mm
TfNSW(2)	10 mm and 14 mm
DoT Vic(3)	10 mm
TMR(4)	10 mm and 14 mm
DIT(5)	10 mm and 14 mm

1. Source: Main Roads (2022c).
2. Source: TfNSW (2020d).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).

4.3.2 Mix Components

Specification requirements for the main OGA mix components are presented below.

Aggregate

Requirements for the aggregate in OGA mixes are the same as described in Section 4.1.2 in relation to DGA mixes for Main Roads and the other SRTAs.

More details on particle size distribution or combined aggregate grading for OGA mixes are presented in Table 4.23.

Binder

Table 4.21 summarises the binder requirements for the OGA mixes.

Table 4.21: Australian binder requirements for OGA mixes

Jurisdiction	Mix size	Binder type	Binder content (% by mass)	Maximum binder drain-off (%)
Main Roads(1)	10 mm	A20E	4.5 ± 0.3	0.3
TfNSW(2)	10 mm	A15E	3.8–5.7	0.3
	14 mm		3.4–5.2	
DoT Vic(3)	10 mm	A10E, A15E, A20E, A30P or A35P	6.5 ± 0.3	0.3
TMR(4)	10 mm	A15E	≥ 9.0	0.3
	14 mm		≥ 8.0	
DIT(5)	10 mm	A15E	5.6 (target)	0.3
	14 mm		5.3 (target)	

1. Source: Main Roads (2022c).
2. Source: TfNSW (2020d).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).

The use of polymer modified binder (PMB) in OGA is specified by all the jurisdictions. The typical binder content for 10 mm and 14 mm OGA for Main Roads, TfNSW and DIT varies between 3.4 to 5.7%. DoT Vic specifies 6.5% binder content and TMR specifies greater than 8.0% binder content in OGA mixes. A maximum binder drain-off limit of 0.3% is also specified by all jurisdictions, when tested in accordance with

Austroads test method AGPT/T235-06 *Asphalt Binder Drain-off* (Austroads 2006a). Table 3.21 shows that the binder content requirements specified by Main Roads are generally in line with those specified by the other SRTAs.

Hydrated lime

Table 4.22 lists the hydrated lime contents specified for OGA mixes.

Table 4.22: Australian lime content requirements for OGA mixes

Jurisdiction	Hydrated lime content (% by mass of total aggregate)
Main Roads[1]	1.5%
TfNSW[2]	1% (minimum)
DoT Vic[3]	1% (minimum)
TMR[4]	1% (minimum)
DIT[5]	1% (minimum)

1. Source: Main Roads (2022c).
2. Source: TfNSW (2020d).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).

Fibre additives

The requirements for the fibres that are typically added to OGA to control binder drain-off are the same for Main Roads and the other SRTAs as described in Section 4.1.2 for DGA mixes.

Adhesion agent

The observations made regarding the use of adhesion agents in the OGA specifications reviewed are the same as described in Section 4.1.2 for DGA mixes.

4.3.3 Particle Size Distribution

Table 4.23 summarises the PSD envelopes specified for OGA mixes.

Main Roads and DoT Vic use 10 mm OGA mixes because layer thicknesses of 30 mm OGA are placed, whereas as the other SRTAs allow 10 mm and 14 mm and thicker OGA layers.

The PSD specified by Main Roads is similar to the grading adopted by TMR and DIT; however, the grading specified by TfNSW for 10 mm and 14 mm OGA mixes is finer than the gradings adopted by Main Roads and the other SRTAs especially for a 2.36 mm sieve size.

Table 4.23: **Australian OGA particle size distribution requirements**

AS sieve size (mm)	Main Roads[1]	Tfnsw[2]		DoT Vic[3]	TMR[4]		DIT[5]	
	Percentage passing sieve size (%)							
	10 mm	10 mm	14 mm	10 mm	10 mm	14 mm	10 mm	14 mm
19.0	–	–	100	–	–	100	–	100
13.2	100	100	85–100	100	100	85–100	100	85–100
9.5	90–100	85–100	65–95	90–100	85–100	40–76	85–100	45–70
6.7	#[6]	50–80	35–75	50–65	40–75	20–47	35–65	25–45
4.75	30–40	25–55	15–45	25–35	20–46	9–30	20–45	10–25
2.36	10–16	10–35	3–25	10–20	4–20	3–17	10–20	7–15
1.18	8–14	0–19	0–20	6–12	2–16	1–14	6–14	6–12
0.600	#	#	#	5–10	#	#	5–10	5–10
0.300	4–10	#	#	4–8	0–10	0–9	4–8	4–8
0.150	#	#	#	3–6	#	#	3–7	3–7
0.075	2–4	0–4	0–4	3–5	0.5–5.5	0.5–5.5	2–5	2–5

1. Source: Main Roads (2022c).
2. Source: TfNSW (2020d).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).
6. Where PSD is shown as #, state the value of the PSD limits in the mix design.

4.3.4 OGA Volumetric Properties

The OGA mix design requirements vary between the various specifications reviewed. It is worth noting that Main Roads uses Marshall compaction for the design of its mixes, whereas TfNSW and DIT use gyratory compaction. Therefore, the volumetric requirements specified by them are not necessarily comparable to Main Roads values.

Table 4.24 summarises the laboratory compaction and volumetric requirements specified for OGA mixes. Main Roads air void limits for 10 mm and 14 mm OGA are between 16 to 21%. TfNSW and DoT Vic have a range of air voids between 18 to 25%, the upper limit being higher than other jurisdictions.

Main Roads and DIT do not specify minimum binder film index (BFI) requirements. TfNSW, TMR and DoT Vic specify minimum BFI of 15, 16 and 20 microns respectively.

Table 4.24: Australian laboratory compaction method and volumetric requirements for OGA mixes

Jurisdiction	Mix size	Laboratory compaction level	Laboratory compacted air voids (%)		BFI
			Min.	Max.	(microns)
Main Roads[1]	10 mm	75 blows (Marshall)	16.0	21.0	Not specified
TfNSW[2]	10 mm	80 cycles (Gyratory)	20.0	25.0	> 15
	14 mm				
DoT Vic[3]	10 mm	50–80 cycles (Gyratory)	18.0	25.0	> 20
TMR[4]	10 mm	50 blows (Marshall)	20.0	N/A	> 16
	14 mm	50 blows (Marshall)			
DIT[5]	10 mm	80 cycles (Gyratory)	18	20.0	Not specified
	14 mm	80 cycles (Gyratory)			

1. Source: Main Roads (2022c).
2. Source: TfNSW (2020d).
3. Source: VicRoads (2018b).
4. Source: TMR (2022a).
5. Source: DIT (2022).

4.3.5 OGA Performance-related Requirements

There is not currently a harmonised approach to specifying performance-related properties for OGA mixes. None of the SRTAs has specified any compaction criteria such as in situ air voids and/or density ratio requirements. OGA mixes are however required to be placed and compacted as per the project quality plan requirements.

4.3.6 Pavement Design and Construction Considerations

Pavement design considerations for flexible pavements with OGA surfacings were reviewed.

Table 4.25 summarises the allowable layer thickness ranges for OGA mixes, and the asphalt design modulus methods are listed in Table 4.26.

Table 4.25: OGA allowable layer thickness ranges

Jurisdiction	Allowable layer thickness for different nominal asphalt sizes (mm)				
	5 mm	7 mm	10 mm	14 mm	20 mm
Main Roads[2]	–	–	30[1]	–	–
TfNSW[3]	–	–	25–40	35–56	–
DoT Vic[4]	–	–	30	–	–
TMR[5]			25–35	35–45	

Jurisdiction	Allowable layer thickness for different nominal asphalt sizes (mm)				
	5 mm	7 mm	10 mm	14 mm	20 mm
DIT(6)	–	–	–	–	–
IPWEA(7)	Minimum of 4 to a maximum of 6 times the maximum nominal aggregate size				

1. Asphalt nominal total thickness.
2. Source: Main Roads (2013a).
3. Source: RMS (2018).
4. Source: VicRoads (2018b).
5. Source: TMR (2022a).
6. Source: DIT (2021).
7. Source: IPWEA (2016).

Table 4.26: OGA asphalt design modulus methods

Jurisdiction	Method to determine asphalt design modulus	Modulus test method
Main Roads(1)	For OGA (C320 binder) nominal thickness of 60 mm or less refer to Engineering Road Note 9 (ERN 9) Table 7 – the modulus varies between 1,500 MPa to 2,500 MPa based on speed limits For OGA thickness greater than 60 mm, the modulus used must be less than 800 MPa	AS/NZS 2891.13.1-2013
TfNSW(2)	Maximum 300 MPa (wearing surface if taken into account in pavement design) and maximum 500 MPa (as a drainage interlayer)	–
TMR(3)	800 MPa (for all WMAPTs and design speeds)	AS/NZS 2891.13.1-2013

1. Source: Main Roads (2013a).
2. Source: RMS (2018).
3. Source: TMR (2021).

OGA manufacturing, spreading and compaction temperature requirements are specified in relevant SRTA specifications and are generally like those for DGA as described in Section 4.1.7

4.3.7 Tack Coat and Waterproofing Seal

Tack coat and waterproofing seal requirements for OGA mixes are the same for all the agencies as described in Section 4.1.8. In addition, TMR specifies a waterproofing seal (WP-A) between the asphalt surfacing and the intermediate layer in pavements with multiple asphalt layers and where the surfacing is OGA. The Main Roads 30 mm OGA mix is placed over a 30 mm DGA which is on a waterproof seal.

4.4 Crumb Rubber Asphalt

Crumb rubber asphalt (CRA) contains a specified proportion of crumb rubber incorporated in it and is intended to reduce the risk of reflection cracking through the pavement. Only TfNSW and DoT Vic have standard specifications for CRA.

The following specifications were reviewed:

- Transport for New South Wales QA Specification R118 – *Crumb Rubber Asphalt* (2020f)
- DoT Vic Specification 421 – *High Binder Crumb Rubber Asphalt* (2020).

Table 4.27 lists the CRA properties and requirements specified by TfNSW and DoT Vic.

Table 4.27: CRA properties and requirements

Properties	TfNSW(1)	DoT Vic(2)
Nominal size (mm)	10 & 14	10, 14 & 20
Binder type	AR450	C320
Binder volume (%)	7.3–8.3 by mass	Not specified

Properties	TfNSW(1)	DoT Vic(2)
Crumb rubber content (%)	Min. 2%	Not specified
Hydrated lime (%)	Min. 1.5	Not specified
Adhesion agent (%)	Min. 1.0	
Air voids in laboratory compacted mix, at 120 cycles (%)	3.0–6.0	5.0–6.5
Layer thickness (mm) Size 10 mm Size 14 mm	 35 45	Not specified
Modulus (MPa)	Not specified	Report
Deformation resistance (mm) At 60°C and 30,000 cycles At 60°C and 5,000 cycles		Report
Characteristic value of in situ air voids (%)	3.0–7.0(3)	Not specified

1. Source: TfNSW (2020f).
2. Source: VicRoads (2020).
3. For layer thickness 50 mm or greater. The upper limit will be 8.0% for layer thickness 30 to 50 mm.

5 International Road Agencies Practices

This section reviews asphalt specifications, guidelines and practices used by the overseas road agencies.

5.1 New Zealand

NZ Transport Authority (NZTA) *Specification for Dense Graded Asphalt Concrete* (NZTA 2020) details its mix design requirements.

Table 5.1 summarises the asphalt mix requirements. Note that the minimum tensile strength ratio of 75% is lower than the minimum 80% commonly used in Australia. The effect of moisture on asphalt mixes is tested in accordance with ASTM D4867:2014.

Table 5.1: NZTA asphalt mix properties and requirements

Properties	DGA[1, 2]			SMA[1]			OGPA (HV)[3]			
Nominal sizes (mm)	10, 14, 20, 28			7, 10, 14			7, 10, 14, 20			
Nominal size (mm)	10 mm	14 mm	20 mm	7 mm	10 mm	14 mm	7 mm	10 mm	14 mm	20 mm
Maximum layer thickness (mm)	35	50	70	30	40	55	20	25	30	50
Binder content (% by mass)	4.5–6.5	4.0–6.0	3.8–5.8	6.0–7.3	6.0–7.0	5.8–6.8	4.5	4.0	4.0	4.0
Air voids in laboratory compacted mix, heavy traffic category	120 cycles		80/120 cycles	80 cycles			–			
	4.0		3.0	4.0			25–30			
Air voids – Marshall method, heavy traffic category	75 blows		50/75 blows	50 blows			–			
	4.0		3.0	4.0						
Stability (kN) – minimum	6.5			–			–			
Flow (mm)	2–4.5			–			–			
VMA (%) – minimum	12–16[4]			16–18[4]			–			
Tensile strength ratio (%) – minimum	75			–			75			

1. Source: NZTA (2020).
2. DGA for medium, heavy and very heavy traffic, heavy wearing course and all basecourse mixes.
3. Source: Transit NZ (2007).
4. Based on mix nominal size and design air voids requirement.

The *Guide to Pavement Structural Design* (NZTA 2018) recommends using epoxy modified open graded porous asphalt (i.e. 25% epoxy bitumen with standard 80/100 penetration grade bitumen (equivalent to C170 Australian binder)) to enhance resistance to ravelling; however, no rheological improvements are generated. Main Roads specifies higher air voids content than NZTA in a laboratory compacted mix. Similarly, Main Roads requires higher minimum TSR than NZTA (i.e. 80% for DGA).

5.2 South Africa

The *Asphalt Mix Design Manual for South Africa* (Southern African Bitumen Association (SABITA) 2014) categorises asphalt mixes into sand-skeleton or stone-skeleton types based on aggregate packing. Sand-skeleton mixes include semi gap graded and gap graded asphalt, and medium fine continuously graded asphalt. Stone-skeleton mixes include coarse continuously graded asphalt, stone mastic asphalt, ultra-thin friction courses, and open graded porous asphalt.

Workability of asphalt mix (i.e. ease of handling, placing and compacting) for a given aggregate grading can be improved by increasing binder content, decreasing binder viscosity, less angular aggregate, limiting the maximum particle size to less than a third of the layer thickness and construction control that ensures the mix is compacted at the proper temperature. Durability of the mix can be controlled by appropriate binder in relatively thick film, low air voids (dense graded packing), sound, durable and strip-resistant aggregates, and the use of anti-strip agents or hydrated lime.

Table 5.2 summarises SABITA mix requirements.

Table 5.2: SABITA mix properties and requirements

Properties(1)	Sand skeleton					Stone skeleton			
Nominal sizes (mm)	7, 10, 14, 20, 25					10, 14, 20, 25			
Preferred layer thickness (mm)	7	10	14	20	25	10	14	20	25
Maximum layer thickness (mm)	25	35	50	90	110	35	50	90	110
Air voids in laboratory compacted mix (25 gyratory cycles)	> 0 and < 2					> 0 and < 2			
VMA (%) – minimum	11–16(2,3)								
Tensile strength ratio – minimum (4)	0.60, 0.65, 0.70, 0.75, 0.80								
Richness modulus (minimum)	2.9					3.4			

1. Source: SABITA (2014).
2. Based on mix nominal size and design air voids requirement.
3. Only values for continuously graded mixes are available and presented in this table.
4. Based on permeability (low, medium, high) and climatic condition (dry, medium, wet).

The *User Guide for the Design of Hot Mix Asphalt* (SABITA 2005) specifies the modified Lottman test for the determination of moisture sensitivity of the asphalt mix. Indirect tensile strength (ITS) measurements are taken before and after a sample is subjected to an environmental stress regime (water submergence and freeze-thaw) and readings obtained are expressed as a tensile strength ratio (TSR). The TSR represents a mix's ability to prevent moisture penetration and tests are to be conducted at 7% air voids at a minimum compaction of 93% of the maximum theoretical relative density (MTRD).

Air voids content specified by SABITA (< 2%) is not comparable with the Main Roads specification (3.5–5.5%) as SABITA uses 25 gyratory cycles, however, Main Roads uses 75 Marshall blows for laboratory compaction.

5.3 United States of America

Several agencies in the USA use the wet-to-dry TSR to evaluate a compacted mix with some agencies using wheel tracking devices. In Australia, only TfNSW, DoT Vic and TMR use TSR to assess the asphalt mix.

The Hamburg wheel tracking (HWT) device is commonly used by transportation agencies in the USA to estimate the rutting and moisture resistance of asphalt mixes. The Texas boiling test is simple, quick, and easy to perform for measuring the moisture susceptibility of asphalt binder qualitatively (Transportation Consortium of South-Central States 2018).

5.3.1 Federal Highway Administration

The Federal Highway Administration (FHWA 2020) specifies mix design requirements for asphalt pavements as shown in Table 5.3.

Table 5.3: FHWA mix design requirements

Design ESAL (million)	Gyratory compaction level (% theoretical maximum specific gravity, G_{mm})			Min. VMA (%)(1) Nominal size(2)					VFA (%)	Dust-to-binder ratio(3)	Min. TSR
	$N_{initial}$	N_{design}	N_{max}	25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm			
< 0.3	6 (≤ 91.5)	50 (96.0)	75 (≤ 98.0)	12–15	13–16	14–17	15–18	–	70–80	0.8–1.6	0.80
0.3 to < 3	7 (≤ 90.5)	75 (96.0)	115 (≤ 98.0)						65–78		
3 to 30	8 (≤ 89.0)	100 (96.0)	160 (≤ 98.0)						65–78		
–	6 (≤ 91.5)	50 (96.0)	75 (≤ 98.0)	–	–	–	–	16–19	76–80	0.6–2.0	

1. When mineral filler or hydrated lime is used, include in the calculation for compliance with the VMA.
2. The nominal maximum size aggregate is one size greater than the first sieve to retain more than 10% of the combined aggregate.
3. The dust-to-binder ratio is the effective asphalt content divided by the total percent of material passing a number 200 (75-um) sieve. Dust includes lime, bag house fines and other mineral matter.

In addition to Table 5.3, FHWA (2020) specifies following requirements:

- The target air voids content is 3 to 5%.
- The maximum allowed temperature to heat asphalt is 185 °C.
- The minimum air temperature for asphalt placement is 1.7 °C and rising.
- In case of using lime as the antistripping agent, the aggregate moisture is to be adjusted to at least 4% by mass of the aggregate.
- Compaction is to be carried out with at least 3 rollers, with at least one being a pneumatic-tyre roller.

Table 5.4 summarises the asphalt mix placement temperature requirements.

Table 5.4: FHWA asphalt mix placement temperature requirements

Road surface temperature (°C)	Minimum lay-down temperature (°C) (1)		
	< 50 mm	50–75 mm	> 75 mm
< 1.7	(2)	(2)	(2)
1.7–4.3	(2)	(2)	138
4.4–4.9	(2)	141	135
10.0–15.5	146	138	132
15.6–21.1	141	135	129
21.2–26.6	138	132	129
26.7–32.2	132	129	127
≥ 32.2	129	127	124

1. Do not heat asphalt concrete mix above the temperature specified in the approved asphalt concrete mix design.
2. Do not pave.

5.3.2 Texas Department of Transportation

Texas Department of Transportation (TxDOT 2014) specifies the following requirements related to DGA mixes:

- The minimum VMA is based on aggregate size depending upon different aggregate categories such as:
  - 11.5% and 13.0% for coarse aggregates

  - 12.5%, 14.5% and 15.5% for fine aggregates.

- The laboratory mix design properties are:
  - target laboratory-moulded density 96.5% (can be increased to 97.0% or 97.5%)
  - design gyrations 50 (varies 35 to 100 gyrations based on requirements)
  - indirect tensile strength (IDT dry) 85–200 psi (586–1,378 kPa) (IDT strength can exceed 200 psi (1,378 kPa if the corresponding HWT rut depth is greater than 3.0 mm and less than 12.5 mm).

- The HWTT requirements (minimum number of passes at 12.5 mm ruth depth, tested at 50 °C) are:
  - PG 64 or lower: 10,000 (can be decreased to no less than 5,000 passes)
  - PG 70: 15,000 (can be decreased to no less than 10,000 passes)
  - PG 76 or higher: 20,000.

- The temperature at the time of shipping (at discharge point) is 100 to175 °C.
- A hand-held thermal camera or infrared thermometer is used to measure and record the internal temperature of the mixture as discharged from the truck for placement.
- The road surface temperature for placement of the asphalt mix should be at or above 15.5 °C.
- In situ air voids in the compacted mix should be between 3.8% and 8.5%.

The TxDOT upper limit for in situ air voids content (8.5%) is higher than the Main Roads upper limit (6%). In fact, all SRTAs have an upper maximum limit of 7%. The reason for US road agencies having a higher upper limit for in situ air voids as constructed is allowance for asphalt secondary compaction (densification under traffic in 2 to 3 years after construction).

5.4 Europe

Pereira and Pais (2017) evaluated asphalt mix design methods to identify challenges for the development of a European method. In Europe, asphalt test specimens are generally compacted using the Marshall method as specified in the European standard BS EN 13108-1 for both stability and flow in addition to the ratio between the stability and flow and volumetric parameters of the asphalt mix. France uses a different approach based on gyratory compaction of test specimens. European Standard BS EN12697-34 defines the procedure to perform Marshall tests and interpret results.

European mix design processes include water sensitivity and resistance to permanent deformation assessed using the indirect TSR and HWTT.

In France, asphalt mix design with gyratory compaction is complemented with water sensitivity testing through the Duriez test, permanent deformation with the laboratoire central des ponts et chaussees (LCPC) wheel tracking test, and stiffness assessment with the two-point trapezoidal bending beam. In the French method, asphalt mixes are designed based on their use, the type of mix and traffic volume by implementing design levels from zero to 4 based on traffic loading. Testing requirements increase with the increase in the traffic level incorporating water resistance testing with the gyratory test. Lower levels consist of volumetric design with the verification of the effect of water in the asphalt mix. Higher levels include stiffness modulus and fatigue resistance testing as well.

In the United Kingdom (UK), complementary tests are used to assess water sensitivity, rutting, stiffness and fatigue resistance.

The determination of fatigue resistance for an asphalt mix requires testing of a large number of specimens. European Standard BS EN 12697-34 requires the testing of at least a total of 18 specimens at 3 different strain levels. ASTM standard requires only 6 specimens and the AASHTO standard does not define the number of specimens tested. The rutting resistance, as per the European standard, tests the susceptibility of asphalt mixes to deform under load and temperature.

6 Stripping Occurrences in Western Australia

Main Roads has observed the effects of moisture in thick lift asphalt (TLA) and full depth asphalt (FDA) on a number of sites. The following sections provide the details of stripping issues identified on the Main Roads network.

6.1 New Perth Bunbury Highway

The section of New Perth Bunbury Highway between Safety Bay Road to Mandijigoordap Road was constructed in 2008 as an FDA by the Southern Gateway Alliance. The asphalt mix designs included:

- 30 mm OGA10, C320
- 30 mm OGA10, C170
- 130 mm DGA20, C320
- 60 mm high bitumen content DGA20, C320.

A cherry-flavoured asphalt additive was added to mask the odours from the mobile asphalt plant located in the vicinity of residential properties. The chemical amyl acetate was thought to be a constituent of the asphalt additive, however, its effect on the long-term performance of asphalt is unclear.

The high bitumen content DGA20 was flagged for non-conformance (low refusal air voids, VMA > 13%, PSD) and these were argued to be beneficial deviations. During the field trials it was observed that emulsion flowed into the asphalt where it was voided and discharged at the base of the layer. This observation illustrated the need to improve edge compaction and it further raised doubts about the degree of compaction and void contents along the cold construction joints within the pavement. Figure 6.1 shows stripping observations from the New Perth Bunbury Highway.

Figure 6.1: Stripping observations on New Perth Bunbury Highway

Photo 1: Water seepage from underneath 20 mm intermediate mix after rain shower.

Photo 2: View of the wet core showing some of the voids at the layer interface.

Source: Main Roads.

6.2 Great Eastern Highway, Greenmount

Early stages of stripping were observed in the TLA on the Great Eastern Highway between Stuart Street and the Bullara Road intersection. The asphalt mix was perceived as poor quality and it was thought that poor construction practice was adopted. The project location is known for fluctuating water levels, above and below ground level, which may exacerbate the stripping potential of poorly constructed asphalt. Figure 6.2 shows stripping in the TLA on the Great Eastern Highway.

Figure 6.2: Stripping observations on Great Eastern Highway

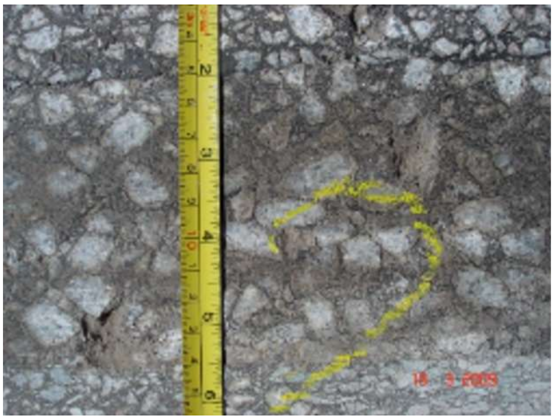


Photo 1: Pavement with 3 layers of asphalt approximately 200 mm thick.

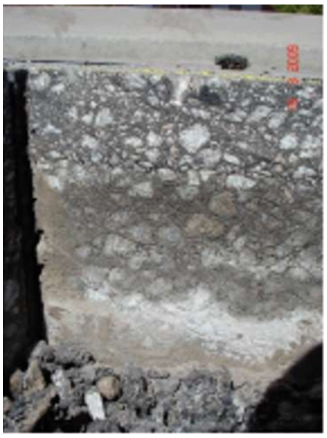


Photo 2: A view of the face of the hole.

Source: Main Roads.

6.3 Great Eastern Highway and Roe Highway Interchange

The Great Eastern Highway and Roe Highway Interchange (GERI) project was constructed in 2012. Asphalt mixes used were JM X (20 mm mix) and JM Y (14 mm mix). Pavement types used were pavement type E (PTE), pavement type F (PTF) and pavement type D (PTD) as shown in Figure 6.3.

Figure 6.3: Pavement structure for the GERI project

Pavement Type D (PTD)	
DGA 14 mm (A15E PMB)	50 mm
DGA 14 mm (A35P PMB)	40 mm
DGA 20 mm (A35P PMB)	60 mm
DGA 20 mm Class 320 bitumen – 260 mm	260 mm
Crushed Limestone sub-base	200 mm
Select fill subbase	–
Sandy clay layer	–

Pavement Type E (PTE)	
DGA 14 mm (Class 320 Bitumen)	40 mm
DGA 20 mm (C320)	205 mm
Crushed Limestone sub-base	200 mm
Select fill subbase	–
Sandy clay layer	–

Pavement Type F (PTF)	
DGA 14 mm (Class 320 Bitumen)	40 mm
DGA 20 mm (C320)	180 mm
Crushed Limestone sub-base	200 mm
Select fill subbase	–
Sandy clay layer	–

Source: Main Roads.

Rutting developed in the pavement soon after opening to traffic. An investigation was launched to determine whether deformation was in the uppermost layer of the intermediate asphalt or the next layer down due to inadequate PMB usage. No seal was placed to waterproof the top of the intermediate asphalt layer. The investigation concluded that stripping and collapse of the second top layer DGA20 or DGA14 resulted in rutting. Figure 6.4 shows signs of stripping in the cores.

Figure 6.4: **Signs of stripping in the cores**

Photo 1: Stripping leading to voided areas in the core.

Photo 2: Stripping leading to voided areas in the core.

Source: Main Roads.

6.4 Bunbury Port Access Project, Stage 2, Picton Roundabouts

The pavement was constructed in 2012 with JM L (20 mm DGA A35P basalt) and JM M (14 mm DGA A35P basalt and 14 mm DGA A15E) asphalt mixes. Reportedly trucks with either a local basalt mix or a granite mix tipped randomly into the paver as it arrived, with no clear distinction between basalt or granite mixes. It was noted that when cores were split into separate layers, free water was visible on the broken layer interfaces.

6.5 Great Eastern Highway, Belmont

The Great Eastern Highway Upgrade (Kyoong Road to Tonkin Highway) was constructed as an FDA in 2013 with JM N (20 mm DGA) intermediate layers and 14 mm DGA with a A15E wearing course. The mixes included RAP and warm mix additives. The high in situ moisture content during construction was likely to be due to excessive median irrigation, not rainfall. Moisture in asphalt was notably present at the layer interface (Figure 6.5).

Figure 6.5: **Moisture at the layer interface**

Photo 1: Moisture at the layer interface.

Photo 2: Moisture at the layer interface.

Source: Main Roads.

6.6 Leach Highway Pavement Investigations

The following investigations were carried out on Leach Highway:

- Investigations at Shelley Bridge indicated that the asphalt thickness of lanes 1 and 2 was in the range of 70 mm to 110 mm. The original asphalt had been overlaid and the lower layer of asphalt was generally badly stripped. The entire bridge was resurfaced in 2017 and a 70 to 100 mm thickness of asphalt was replaced.
- Leach Highway between Winnacott Street and Stock Road was investigated in 2011 as this section was showing extensive cracking and pumping of fines. The results indicated that the lower layer of asphalt in the wheel paths of lane R2 was poor. There was also poor bond between the layers of asphalt.
- Leach Highway between Norma Road and North Lake Road was investigated in 2010 as there were signs of failure in lane R2 of the westbound carriageway in the form of rutting, cracking and pumping of fines and lane R3 was showing signs of cracking in both wheel paths. The investigation showed that the underlying asphalt layer had failed. The exact cause of failure needs to be established.
- Leach Highway between Stock Road and Carrington Street showed signs of failure in the form of crocodile cracking and pumping of fines. A detailed geotechnical investigation initiated in April 2012 showed heavily voided and pitted asphalt in both lanes (i.e. R2 and R3).
- Leach Highway between Corinthian Road and Vahland Avenue on the eastbound carriageway was investigated in April 2012 as a result of distress appearing in L2 in the form of cracking, pumping of fines and rutting in both wheel paths. The section comprised 14 mm and 10 mm DGA in addition to a 14 mm SAM seal wearing course. The 14 mm DGA layer had significant voids and pitting and the 10 mm DGA crumbled when light pressure was applied.

Figure 6.6 shows stripping in asphalt at different places.

Figure 6.6: Stripping in Leach Highway

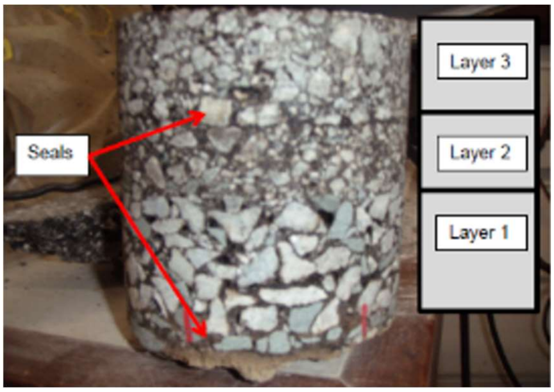


Photo 1: Core from Leach Highway between Stock Road and Carrington Street.

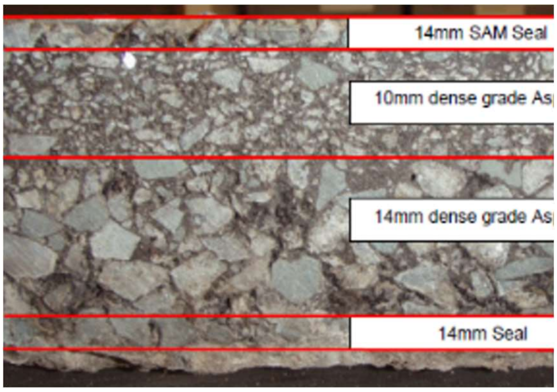


Photo 2: Core from Leach Highway between Corinthian Road and Vahland Avenue.

Source: Main Roads.

7 Virtual Workshop

In June 2021 a virtual workshop was held to discuss the findings from the literature review with selected asphalt experts to capture their views on the effect of moisture on asphalt pavement performance to highlight potential areas that Main Roads needs to investigate and potential changes to consider for improving its current practice.

The workshop covered the mix design, aggregate, use of hydrated lime and liquid adhesion agents, and binder content and testing procedures to assess the moisture susceptibility of the asphalt mixes. The stripping cases investigated in other Australian jurisdictions were discussed as well.

7.1 Arrangements

A list of the potential workshop attendees was prepared jointly by the Main Roads project manager and the ARRB project leader. An invitation was sent to the invitees on Friday 21 May 2021 for the workshop to be held on Friday 4 June 2021.

The draft version of the literature review section was circulated to all invitees a week prior to the workshop to allow familiarisation with the findings of the literature review.

7.2 Participants

The invitees were nominated based on expertise in asphalt material and pavements with a balanced representation from Main Roads, ARRB and the industry. 21 people were invited and 14 people including 6 from the industry attended the virtual workshop.

7.3 Structure of the Workshop

The workshop duration was 1 hour and 45 minutes comprising:

- 45 minutes – presentation
- 1 hour – discussion.

The presentation was given by the ARRB project leader and the Main Roads project manager to report the key findings of the literature review.

The workshop covered the following areas:

- asphalt stripping mechanisms
- aggregate type and mineralogy
- testing procedures
- road design and construction considerations
- comparison between Australian state road agency practices
- international road agency practices
- key findings of the literature review.

7.4 Workshop Discussion

This section summarises the main points discussed:

- Perth asphalt mixes consist mostly of granite aggregate. If granite aggregate is prone to stripping due to its inherent hydrophilic characteristics as indicated in the literature review findings, the stripping potential needs to be investigated to inform a strategy to manage moisture susceptibility of the relevant asphalt mixes.

- Petrographic test results from the quarry are required as a part of the Main Roads mix design process. However, currently, no additional testing is carried out related to the moisture sensitivity of granite aggregate in the asphalt mix.
- Some of the participants were keen to discuss which aggregate (e.g. granite aggregate) characteristics related to stripping could be assessed using XRF.
  - The purpose of XRF is to determine silica content, the mineral assemblage of the source rock and the molecular structure or content. It was argued that research, indicating threshold values based on XRF data that indicates which components are critical to stripping, must be available.
  - It was highlighted by one of the asphalt suppliers that a petrographic and secondary mineral report is provided to meet the criteria of the asphalt mix design approval process of Main Roads.
- Further to the above discussion about XRF, in UK source rock geological properties have been investigated using a mineral liberation analyser (MLA) device instead of XRF. This apparatus uses a scanning electron microscope to understand the geology of the aggregate, grain sizes and percentages of different geological components of the aggregate source rock to find some correlation between certain components of granites forming the granite aggregate.
  - Certain phases within granite were problematic if there are high percentages of those phases in aggregate. Also, there were issues if there are large grain sizes in the problematic components.
  - Main Roads highlighted that petrographic results of the approved mixes can be used to assess whether stripping potential is related to petrographic results.
- It was identified that granite aggregate has been used in Victoria in some of the mixes, but no stripping issues have been reported. The use of hydrated lime in the mix seemed to solve the stripping problem in Victoria for the last 30 years.
- A concern was raised that the 'granite we have is the granite we have'. Given the volume of granite that has been laid through WA, 'are we able to identify what percentage of granite is specific to the stripping issue?'
- Currently the project is aimed at investigating if the granite aggregate is a real cause of stripping in WA through analysing Main Roads data related to stripping including actual cases (considering mix design, construction and maintenance history of the pavements) as a part of the data analysis phase of the project. If it is confirmed that the granite is contributing to asphalt stripping in WA, potential technical solutions for this problem will be sought. Otherwise, the project will investigate further to pinpoint the actual problem and recommend possible technical solutions Main Roads may consider adopting.

- A question was raised regarding past investigations to identify the extent to which stripping is limiting the life of WA pavements.
  - Main Roads highlighted that a few investigations were done on selective sites which shows stripping in thick lift asphalt or full depth asphalt pavements. Detailed investigation is required to establish the causes of stripping.
  - Main Roads is placing more and more FDA pavements and wants to be prepared to address stripping issues at the planning, mix design and construction phases rather than responding after stripping has occurred. This research project is aimed at providing this preparedness.
- The next stage of the project may include laboratory testing on WA asphalt mixes (asphalt mixes constituting granitic and non-granitic aggregate such as basalt, limestone, etc.) and comparing the susceptibility to moisture damage.

  - This idea was well supported by the participants. One of them highlighted that South Africa previously used modified Lottman testing for assessing moisture resistance of an asphalt mix and moved towards more performance-based tests. The literature review investigated performance-based tests including the Hamburg wheel tracking test (HWTT).
  - The asphalt mix performance can be tested and if such testing is undertaken as a priority, it will provide early insight as to whether stripping characteristics can be improved by better mix design. HWTT can be used not only for the mix deformation resistance assessment but also for permeability and stripping potential.

- Further to the use of the HWTT, it was highlighted that one of the issues with the inflection point in the measured deformations is what is causing it. The inflection point may be caused by cracking due to rutting, which leads to increased moisture susceptibility, whereas without cracking, the mix may have been performing well in terms of moisture resistance. Therefore, in analysing the test results, careful consideration needs to be given as to whether the occurrence of the inflection point is due to moisture susceptibility or deformation. More research is required to understand the inflection point reached in HWTT.
- Quarry management of source rock and the aggregate crushing and blending process is important. Quarry faces may vary due to the effects of weathering, which affects aggregate quality. This variability should be recognised and managed accordingly.

  - One of the asphalt suppliers highlighted that it has been exploring a situation where some of the basalt aggregates from quarries are becoming borderline in terms of stripping due to moisture susceptibility properties. The reason for having problematic aggregate is variability in the source rock on some of the quarry faces, although these quarries historically did not have this problem. The supplier noted that sometimes the hydrated lime has not worked significantly to improve moisture susceptibility whereas liquid adhesion agents have worked well. It is now investigating the occurrence of problematic aggregate on quarry faces and how best to select adhesion agents for its mixes.

- Main Roads specifies 1.5% hydrated lime in the asphalt mix design.

  - It is similar to Victorian specifications where stripping is not a major cause of distress. However, the mineralogy and geology of Victorian aggregates may differ from those in WA. It would be beneficial to investigate whether Victorian granites are different from those in WA in order to assess the applicability of the Victorian experience and the design and construction standards.
  - WA hydrated lime content in the mix is in line with other Australian state road agencies. It should be noted that increasing the hydrated lime content in the mix may lead to more brittleness making the mix more susceptible to cracking. These cracks will open up avenues for moisture ingress into the pavement making it more prone to stripping.
  - Hydrated lime is also used as a filler. Currently Main Roads does not allow use of any filler other than the hydrated lime. Other sources of filler are incorporated by chance from the rock dust. Fillers are used to achieve a range of requirements for asphalt mixes. For instance, filler filled voids to reduce the optimum binder content, help to meet specified aggregate grading requirements, and enhance strength by stiffening the binder mastic but may also decrease workability.
  - Main Roads uses the hydrated lime as both a filler and an adhesion agent. On the other hand, local governments do not include hydrated lime and their mixes start losing sand from the surface with the passage of time and then ravel. However, they do not lose coarse aggregates. Main Roads has 40 years of experience related to mixes, which include hydrated lime. If a decision is taken to move away from hydrated lime, the suitability of using wet adhesion agents will need to be investigated.
  - It is thought that the use of hydrated lime in WA mixes began in the 1980s. Two instances of catastrophic stripping have been observed; one was in the intermediate course, and other in the wearing course. Both were constructed in winter and failed mainly due to poor workmanship.

- Regarding using the Bailey method to achieve more packed mixes, it was the opinion of one of the asphalt suppliers that there is no need to improve on the current procedures for the mix structure. Implementing the Bailey method requires considerable work by Main Roads and industry and this supplier's view was that the method did not need to be investigated. One of the industry experts mentioned that it had compared its mixes with the results from using the Bailey method and that they had aligned well with the results from using the current procedure.
- An investigation is required to evaluate if permeability testing (e.g. Marvil) is required for guidance as a part of the construction process.
  - It was highlighted by Main Roads that the Marvil test may be a bit variable, but other options can be looked at (e.g. constant head) in the laboratory at least.
- An experience was shared related to surface drainage where flushout risers are used to maintain subsurface drains. It has been found that soil grains blocked these drains leading to ponding in the pavement. This highlights the need to maintain the subsurface drainage to ensure it is operating as intended.
- The RMS T230 test method was previously investigated by Main Roads and several issues were identified. The test method states that the plate needs to be clamped to the bench but this may lead to blisters. When Victorian aggregates were tested, they did not pass the RMS test requirement when no adhesion agent was used. Australian Standard AS 1141.50 is a national version of the RMS T230, however South Australia and WA did not agree that this test was appropriate to use. For WA this test is specified for the properties of aggregate mainly for asphalt and sprayed seal. It is used in regions where liquid adhesion agents are used (hydrated lime in the bitumen is not used for the test).
- It would be useful to investigate if there is a database of the AS 1141.50 test results for Perth granite mixes and regional mixes.
- The possibility of increasing binder content to have higher binder film thickness and accommodating air voids through enhanced VMA in the aggregate was discussed.
  - In regional WA, it is challenging to control VMA because it is difficult to get suitable sand sizes within the aggregate grading. As a result, VMA in regional WA tends to be higher and more bitumen content could be accommodated but in Perth it will be more challenging to have higher VMA.
  - In Perth, VMA is generally problematic especially in 10 mm and 20 mm intermediate course mixes, which tend toward the bottom limit of the VMA specification. A lower VMA specification will lead to voids filled with binder (VFB) values of 75 to 80%, which may lead to inadequate rut-resistance.
  - Binder properties that enhance adhesion could also be investigated.
  - Nevertheless, there is evidence that a thicker binder film improves asphalt mix resistance to moisture damage. However, it may initiate other issues such as resistance to permanent deformation as mentioned above. In the UK, research has been carried out to understand which binder is more appropriate for which aggregate type. However, as binders are generally imported, there may be limited opportunities to specify binders with optimum adhesion characteristics. Still, the effect on adhesion of using additives to bitumen (e.g. CRM binder) could be investigated.
- An investigation by east-coast road agencies about 20 years ago, looked at the impact of compactive effort, binder content and air void content on the stripping potential of the asphalt mixes. It was found that Victoria had 0.3% more binder content than NSW and that it had 0.3% more than Queensland. It is important to consider binder content, binder film thickness, compactive effort and compaction temperature to establish mix design.
- The majority of the asphalt pavements in WA have an open graded asphalt surfacing. There can be a problem when the rate of infiltration from rainfall or other sources of water results in the accumulation of water in open graded asphalt particularly as its permeability is reduced by the build-up of road debris. In such situations, where there is an increased risk of moisture infiltrating the underlying dense

graded asphalt, there is need for a robust waterproofing seal on the dense graded asphalt surface. Therefore, suitability of current seals needs to be investigated.

- Vertically interconnected voids are associated with permeability; however, horizontally interconnected voids are associated with stripping. It is worth investigating interconnected voids during the next stage of the project to assess which interconnected voids, vertical or horizontal, are more detrimental in terms of stripping.

7.5 Key Findings

The following key findings were noted:

- An investigation is required to establish the extent of the stripping issue in WA. In particular, the stripping potential of the asphalt mix manufactured using granite aggregates needs to be investigated. This investigation could be carried out through analysing Main Roads data including real-world cases (mix design, construction and maintenance history of the pavement). If stripping is significantly affecting the life of Main roads pavements, potential technical solutions will need to be proposed or recommended for Main Roads consideration.
- The next stage of the project may include laboratory testing on WA asphalt mixes and comparison of the moisture susceptibility of granitic and non-granitic mixes (non-granite mixes from other regions can be used for analysis if Perth mixes are only based on granite aggregate). It is also important to assess how stripping increases with the proportion of granite used in an asphalt mix.
- Petrographic results submitted by the suppliers to Main Roads need to be investigated particularly in relation to whether WA granites are similar to those used in Victoria where stripping has not been an issue. In addition to XRF, the use of alternative tools for aggregate mineralogy and geology such as MLA (mineral liberation analyser) needs to be investigated.
- The HWTT could be considered in the mix design process to assess moisture resistance of the asphalt mix as a performance-based test. It should be noted that the inflection point is not an indicator of moisture susceptibility only. It can happen due to low performance of the mix as well.
- In relation to the use of the RMS test method T230, concerns were expressed about the requirement for plate clamping to the bench. Main Roads currently uses this test for regional mixes where liquid adhesion agents are used instead of hydrated lime. The existing test data needs to be collated and analysed.
- The effect of vertical and horizontal interconnected voids should be investigated.
- It is thought that the current approved asphalt mixes are optimally packed when compared to the results of using the Bailey method. This can be verified in the next stage of the project.
- Surface texture controls the water entering the pavement. Therefore, for dense graded asphalt a tight surface finish is a favourable condition.
- Hydrated lime seems to be the most widely used adhesion agent amongst road agencies, although it was noted that liquid adhesion agents might outperform hydrated lime given a different aggregate type.
- Waterproofing seal on the dense graded asphalt underneath the open graded asphalt needs to be robust to prevent moisture getting into the dense graded mix and ensure water drain-off through the overlying open graded wearing course.

8 Data Analysis and Linkages to Main Road Practice

8.1 Introduction

A key project task was to analyse Main Roads data and the linkages to its practice.

It was discussed and agreed during project proposal development that Main Roads would investigate data availability and provide the data for analysis. It identified test data related to the Gateway WA Alliance field trials on Tonkin Highway for analysis. This data mainly included asphalt mix designs (PSD, binder content, Marshall properties), TSR testing (wet and dry strength, TSR) and asphalt cores (moisture content, air voids, average Marshall density, wheel tracking tests and permeability).

The following data was proposed to be collected and provided by Main Roads for analysis and interpretations:

- TSR – test results, photographs, observations
- moisture monitoring data for asphalt pavements
- asphalt mix design information and results
- information related to asphalt manufacturing plants
- copies of results and reports of internal investigations related to asphalt stripping
- testing procedures and test methods
- examples of projects and sites having stripping issues
- other asphalt pavement stripping-related data deemed helpful for this research.

It should be noted that the data provided by Main Roads for analysis was not collected as a part of this project. Main Roads supplied the data from the Gateway field trials on Tonkin Highway for analysis. As the data supplied was from a single project, this limited the ability to draw general conclusions about the stripping potential of WA mixes.

8.2 Gateway Field Trials Overview

The purpose of the Gateway field trials was to construct a pavement that prevents moisture ingress or movement within the pavement. The field trials were constructed in April 2015 by the Gateway WA Alliance (GWA) on the northbound carriageway of Tonkin Highway over the full width of lanes R1, R2 and R3 and the right-hand shoulder between the off and on ramps of the Boud Avenue Bridge. The trial investigated the influence of moisture ingress on the mix design of the intermediate course and also the effect of seal placement.

Table 8.1 summarises the location of the 4 trial sections each about 150 m in length. Pavement structure and asphalt mix details are provided in Table 8.2 and Figure 8.1.

Table 8.1: Basic information related to Gateway field trials

Section	Chainage		Job mix	Mix type	Lift
	From	To			
Section 1	9,150	9,300	JM A	20 mm DGA	1
			JM A	20 mm DGA	2
			JM A	20 mm DGA	3
			JM B	14 mm DGA	4
Section 2	9,300	9,600	JM C	20 mm DGA	1

Section	Chainage		Job mix	Mix type	Lift
	From	To			
			JM C	20 mm DGA	2
			JM C	20 mm DGA	3
			JM D	14 mm DGA	4
Section 3	9,600	9,750	JM E	20 mm DGA	1
			JM E	20 mm DGA	2
			JM E	20 mm DGA	3
			JM E	14 mm DGA	4
Section 4	9,750	9,900	JM A	20 mm DGA	1
			JM A	20 mm DGA	2
			JM A	20 mm DGA	3
			JM B	14 mm DGA	4

Section 1 was constructed with a 2nd generation mix intermediate course for the seal trials. Sections 2 and 3 were constructed with a 3rd generation mix and AAPA mix intermediate courses respectively. Section 4 was constructed with a 2nd generation mix intermediate course as a control section. Refer to Section 8.3 for more details on the mixes.

Table 8.2: Pavement structure for Gateway trial sections

Course	Mix type	Nominal size	Binder class	Thickness
Wearing course	OGA	10 mm	Class 320	30 mm
Wearing course	DGA	14 mm	A15E	40 mm
Waterproofing seal	–	–	–	–
Intermediate course	DGA	14 mm	A15E	50 mm
Intermediate	DGA	60–65 mm	Class 600	60 mm

Figure 8.1 illustrates the design of the pavement for the Gateway trial sections at Tonkin Highway.

Figure 8.1: Pavement design for Gateway trial sections

Wearing Course – 10 mm Open Graded Asphalt (Class 320 Bitumen)	30 mm
Wearing Course – 14 mm Dense Graded Asphalt (A15E binder)	40 mm
Waterproof Seal	
Intermediate Course – 14mm Dense Graded Asphalt (A15E Binder)	50 mm
Intermediate Course – 20 mm Dense Graded Asphalt (Class 600 Bitumen)	185 mm
Crushed Rock base Subbase	200 mm
Sand Subgrade (Perth Sand) Compact requirement: 96% min.	300 mm

8.3 Asphalt Mix Designs

Apart from Section 3, the field trial sites were asphalt pavements constructed with Main Roads 2nd and 3rd generation asphalt mixes as shown in Table 8.3. Note that Section 3 was constructed with job mix E.

Table 8.3: Details of the different generations of asphalt mixes(1)

Mix generation	Job mix	Mix type	Nominal size	Marshall blows	Air voids (Marshall) (%)	VMA(2) (%)	VFB(3) (%)	BFI(4)	Air voids (350 gyratory cycles)
2nd generation(5)	JM A	DGA	20 mm	75	3.5–5.5	14 min	70 min	8.0 min	2.5 min
	JM B		14 mm		4.0–6.0				
3rd generation(6)	JM C	DGA	20 mm	75	3.5–5.5	14 min	70 min	8.0 min	2.5 min
	JM D		14 mm						

1. Asphalt mixes designed to meet the requirements of Specification 510 (Main Roads 2022a).
2. Voids in mineral aggregate.
3. Voids filled with bitumen.
4. Binder film index.
5. Composed of crushed rock and hydrated lime.
6. Composed of crushed rock, sand and hydrated lime.

It should be noted that the job mix E section was reported as a part of the trials; however, its performance was not evaluated due to a lack of information related to mix design. The PSD of the job mix E was similar to that of the 3rd generation mix JM C, but as the aggregate was sourced from different quarries the packing characteristics of the aggregate may vary.

Table 8.4 summarises the mix design details of the intermediate course asphalt mixes used in the trials. Mix design details of the job mix E were not provided.

Table 8.4: Mix design details of the Gateway field trials

JM #	JM A	JM B	JM C	JM D
Course	Intermediate course	Intermediate course	Intermediate course	Immediate course
Mix generation	2nd generation	2nd generation	3rd generation	3rd generation
Mix type	DGA	DGA	DGA	DGA
Nominal size	20 mm	14 mm	20 mm	14 mm
Aggregate rock type	Granite	Granite	Granite	Granite
Binder class	C600 bitumen	A15E bitumen	C600 bitumen	A15E bitumen
Binder content	4.6% ± 0.3%	4.8% ± 0.3%	4.6% ± 0.3%	5.0% ± 0.3%
Year mix approved?	2014	2015	2014	2016
Plant	Welshpool	Welshpool	Welshpool	Welshpool
Design target particle size distribution (PSD)				
AS sieve size (mm)	Percentage passing by mass (%)			
26.5	100	100	100	100
19.0	91–100	100	92–100	100
13.2	75–89	90–100	77–91	90–100
9.5	64–78	76–90	66–80	76–90
6.7	48–62	62–76	54–68	62–76
4.75	38–52	52–66	45–59	52–66
2.36	28–38	37–47	31–41	37–47
1.18	18–28	27–37	20–30	27–37
0.60	13–21	21–29	14–22	21–29
0.30	7–15	10–18	9–17	10–18
0.15	5–10	5–10	5–10	5–10
0.075	3.5–6.5	4–7	4–7	4–7

JM #	JM A	JM B	JM C	JM D
Marshall criteria				
Marshall air voids (%)	3.5–5.5	4.0–6.0	3.5–5.5	3.5–5.5
VMA(1) (%)	≥ 14	≥ 14	≥ 14	≥ 14
VFB(2) (%)	–	≥ 70	≥ 70	–
BFI(3)	≥ 8.0	≥ 8.0	≥ 8.0	≥ 8.0
Stability (kN)	≥ 8.0	≥ 8.0	≥ 8.0	≥ 8.0
Flow	2.0–4.0	2.0–4.0	2.0–4.0	2.0–4.0

1. VMA denotes voids in mineral aggregates.
2. Voids filled with bitumen.
3. Binder film index.

Based on the mix design data, it can be concluded that:

- Both generations of mixes are 75% blow mix designs with VMA minimum 14% and VFB minimum 70%, BFI minimum 8 and air voids after 350 gyratory cycles minimum 2.5%.
- The 3rd generation mixes are composed of crushed rock, hydrated lime and sand. However, the 2nd generation mixes contain crushed rock and hydrated lime only.
- The 20 mm 3rd generation mix has finer PSD from 6.7 mm to 2.36 mm in comparison to the 2nd generation mix, including use of natural sand and dust fraction.
- The 14 mm 3rd generation mix has the same aggregate components and PSD as the 2nd generation mix.
- Binder content for the 20 mm mixes is the same for 2nd and 3rd generation mixes; however, the binder content for the 14 mm 3rd generation mix is higher than the 2nd generation mix.
- Design air voids of 20 mm for the 2nd and 3rd generation mixes and the 14 mm 3rd generation mix are same (3.5–5.5%). However, design air voids of 14 mm for the 2nd generation mix were higher (i.e. 4.0–6.0). This results in an increase in binder content of 0.2%.
- The 20 mm and 14 mm intermediate courses were prepared using C600 and A15E binders respectively. The 14 mm and 10 mm wearing courses were prepared using A15E and C320 binders respectively.

Table 8.5 summarises the characteristics of the aggregate for the asphalt mixes used in the trials.

Table 8.5: Characteristics of aggregate for the Gateway asphalt mixes

Job mix #	Aggregate source (quarry)	Typical aggregate proportion (%)
JM A		
Aggregate type		
20 mm	Boral Quarry, Orange Grove	26.0
10 mm	Boral Quarry, Orange Grove	20.0
7 mm	Boral Quarry, Orange Grove	12.0
5 mm	Boral Quarry, Orange Grove	5.0
Quarry sand	Boral Quarry, Orange Grove	35.5
Filler (hydrated lime)	N/A	1.5
JM C		
Aggregate type		
20 mm	Boral Quarry, Orange Grove	26.0
10 mm	Boral Quarry, Orange Grove	14.0
7 mm	Boral Quarry, Orange Grove	10.0
5 mm	Boral Quarry, Orange Grove	10.0
Quarry sand	Boral Quarry, Orange Grove	28.5
Natural yellow sand	WA Limestone Mandogalup Sand Quarry	10

Job mix #	Aggregate source (quarry)	Typical aggregate proportion (%)
Filler (hydrated lime)	N/A	1.5
JM D		
Aggregate type		
14 mm	Boral Quarry, Orange Grove	20.0
10 mm	Boral Quarry, Orange Grove	12.0
7 mm	Boral Quarry, Orange Grove	10.0
5 mm	Boral Quarry, Orange Grove	13.0
Quarry sand	Boral Quarry, Orange Grove	33.5
Natural yellow sand	WA Limestone Mandogalup Sand Quarry	10
Filler (hydrated lime)	N/A	1.5
JM B		
Aggregate type		
14 mm	Boral Quarry, Orange Grove	20.0
10 mm	Boral Quarry, Orange Grove	12.0
7 mm	Boral Quarry, Orange Grove	10.0
5 mm	Boral Quarry, Orange Grove	13.0
Quarry sand	Boral Quarry, Orange Grove	33.5
Natural yellow sand	WA Limestone Mandogalup Sand Quarry	10.0
Filler (hydrated lime)	N/A	1.5

8.4 Testing During Construction

8.4.1 Subgrade and Subbase Compaction

The subgrade under the trial pavements was Perth sand. The subbase overlying sand subgrade was crushed rock. The minimum compaction for the subgrade and subbase was 96% and 94% respectively. All the sections in the subgrade and subbase are above those criteria. Table 8.6 provides the compaction results for the subgrade and subbase.

Table 8.6: Construction details

Construction element	Start chainage (m)	End chainage (m)	Lane width	Dry density ratio (%)
Subgrade	9,150	9,400	Full width	100.0
	9,400	9,600		101.5
	9,600	1,000		101.6
Crushed rock subbase	9,050	9,250	Full width	98.6
	9,250	9,500		99.2
	9,500	9,700		94.1
	9,700	9,900		95.7

8.4.2 Asphalt Compaction

Table 8.7 summarises the asphalt compaction data.

Table 8.7: **Asphalt compaction details**

Layer – mix	Core or thin layer density gauge (TLG)	Lower characteristic voids (%)	Upper characteristic voids (%)	Average air voids (%)	Standard deviation of air voids (%)
Section 1 Summary of compaction performance					
1-JMA	Main Roads cores	7.1	7.9	6.3	0.83
	Boral cores	5.0	7.0	6.0	1.06
	Main Roads TLG	6.3	8.5	7.4	1.11
2-JMA	Boral cores (R1, R2)	4.7	7.0	5.9	1.17
	Boral cores (R3)	5.0	7.0	6.0	1.08
	Main Roads TLG (R1, R2)	4.9	7.2	6.1	1.15
	Main Roads TLG (R3)	4.9	9.0	7.0	2.28
3-JMA	Boral cores (R1, R2)	4.9	6.5	5.7	0.91
	Boral cores (R3)	5.0	6.1	5.5	0.61
	Main Roads TLG (R1, R2)	6.3	9.8	8.1	1.80
	Main Roads TLG (R3)	6.7	9.0	7.9	1.28
4-JMB	Boral cores (R1, R2)	5.9	7.9	6.9	1.02
	Boral cores (R3)	5.1	6.7	5.9	0.89
	Main Roads TLG (R1, R2)	7.4	10.1	8.7	1.37
	Main Roads TLG (R3)	4.6	8.3	6.5	2.06
Section 2 Summary of compaction performance					
1-JMC	Main Roads cores	4.4	7.7	6.1	1.66
	Boral cores	3.8	5.9	4.9	1.09
	Main Roads TLG	4.8	7.4	6.1	1.33
2-JMC	Main Roads cores	4.0	7.3	5.7	1.92
	Boral cores	3.8	5.2	4.5	0.69
	Main Roads TLG	3.2	6.0	4.6	1.38
3-JMC	Boral cores	4.0	5.7	4.8	0.85
	Main Roads TLG	4.7	7.0	5.8	1.13
4-JMD	Main Roads cores (R3)	4.1	5.4	4.8	0.70
	Main Roads cores (R1, R2)	5.0	6.7	5.9	0.93
	Boral cores (R1, R2)	4.5	6.2	5.3	0.84
	Boral cores (R3)	5.1	6.5	5.8	0.76
	Main Roads TLG (R1, R2)	5.2	8.2	6.7	1.51
	Main Roads TLG (R3)	6.0	9.0	7.5	1.61
Section 3 Summary of compaction performance					
1-JME20	Main Roads cores	3.6	5.3	4.5	0.95
	Fulton Hogan cores	3.6	4.2	3.9	0.48
	Main Roads TLG	3.3	5.2	4.3	0.99
2-JME20	Main Roads cores	2.6	3.6	3.1	0.57
	Fulton Hogan cores	3.8	5.2	4.5	1.01
	Main Roads TLG	3.1	4.5	3.8	0.73
3-JME20	Main Roads cores	4.3	6.2	4.3	1.07
	Fulton Hogan cores	4.5	5.7	5.1	1.41
	Main Roads TLG	3.7	6.4	5.1	1.41
4-JME20	Main Roads cores	6.1	8.3	7.2	1.12
	Fulton Hogan cores	5.2	7.2	6.2	1.41
	Main Roads TLG	6.1	8.3	7.2	1.12

Layer – mix	Core or thin layer density gauge (TLG)	Lower characteristic voids (%)	Upper characteristic voids (%)	Average air voids (%)	Standard deviation of air voids (%)
Section 4 Summary of compaction performance					
1-JMA	Main Roads cores	5.6	7.5	6.5	0.97
	Boral cores	4.4	6.5	5.5	1.08
	Main Roads TLG (R1, R2, R3)	5.9	9.3	7.6	1.71
	Main Roads TLG (R3)	6.7	8.5	7.6	0.99
2-JMA	Main Roads cores (R2, R3)	6.1	7.4	6.8	0.83
	Boral cores (R2, R3)	3.2	4.8	4.0	0.84
	Boral cores (R1)	3.8	6.1	5.0	1.34
	Main Roads TLG (R1)	3.2	6.5	4.8	1.67
	Main Roads TLG (R2, R3)	4.5	7.0	5.8	1.30
3-JMA	Boral cores	5.5	6.6	6.0	0.92
	Main Roads TLG	3.6	6.3	5.0	1.38
4-JMB	Boral cores (R1)	4.4	6.8	5.6	1.22
	Boral cores (R2, R3)	6.2	7.9	7.0	0.92
	Main Roads TLG (R2, R3)	6.3	8.2	7.3	1.07
	Main Roads TLG (R1)	7.0	8.1	7.5	0.63

Wearing course compaction

Th wearing course comprised A15E 14 mm intersection mix overlayed with 10 mm open graded asphalt to Main Roads Specification 504. The wearing course was placed on top of the waterproofing seal using JM 45. Table 8.8 summarises the wearing course asphalt compaction conformance and Table 8.9 provides details on TLG testing of the intersection mix.

Table 8.8: Wearing course asphalt conformance

Start chainage (m)	End chainage (m)	Lane	Characteristic percentage Marshall density (%)
14 mm dense graded asphalt intersection mix			
9,150	9,900	R3	94.4
		R2	95.0
		R1	95.5
Open graded asphalt			
9,150	9,900	R2	93.4
		R2	94.1
		R3	94.2

Table 8.9: TLG testing results of intermediate mix

Layer	Average air voids (%)	Characteristic percentage Marshall density (%)	Standard deviation
Section 1	12.4	91.4	1.45
Section 2	12.6	91.8	0.89
Section 3	10.9	92.6	1.80
Section 4	10.5	94.1	0.80

Overall, the air voids test results in the intermediate mixes are high. The highest air voids are reported for Section 2 (3rd generation mix) and the lowest for Section 4 (2nd generation mix). It should be noted that TLG measurements are considered as a guide but not highly accurate. For example, the RMS technical guide for field density testing using a nuclear density gauge (RMS 2015) indicates that TLG measurements are not accepted for calculations and reporting. Considering this limitation of TLG, the measurements of air voids in the intermediate mix are taken as a guide and not used for discussion as a part of data analysis.

8.4.3 In Situ Moisture

Production of each mix type was tested on a daily basis for asphalt moisture content in accordance with AS/NZS 2891.10 and the degree of particle coating in accordance with AS/NZS 2891.11. None of the samples failed with all samples having moisture content of 0% and a particle coating of 100%.

Moisture content of the in situ asphalt was determined in each section to establish a baseline of in situ moisture. The cores were sampled on 11 May 2015 after the intermediate mix asphalts were constructed, prior to the application of seal. Moisture content is generally considered low in all samples tested. Table 8.10 summarises the in situ initial core moisture data for the Gateway trials.

Table 8.10: In situ core moisture data for Gateway trial sections

Trial section	Chainage (m)	Lane	Mix type	Layer	Moisture content (%)
1	9,175	R1	20 mm JM A	1	0.2
			20 mm JM A	2	0.1
			20 mm JM A	3	0.1
			14 mm JM B	4	0.1
	9,225	R1	20 mm JM A	1	0.3
			20 mm JM A	2	0.1
			20 mm JM A	3	0.1
			14 mm JM B	4	0.1
	9,275	R1	20 mm JM A	1	0.1
			20 mm JM A	2	0.2
			20 mm JM A	3	0.2
			14 mm JM B	4	0.1
2	9,450	R1	20 mm JM C	1	0.1
			20 mm JM C	2	0.1
			20 mm JM C	3	0.1
			14 mm JM D	4	0.1
	9,450	R3	20 mm JM C	1	0.2
			20 mm JM C	2	0.2
			20 mm JM C	3	0.1
			14 mm JM D	4	0.1
3	9,650	R1	20 mm JM E	1	0.2
			20 mm JM E	2	0.1
			20 mm JM E	3	0.1
			14 mm JM E	4	0.0
	9,700	R1	20 mm JM E	1	0.1
			20 mm JM E	2	0.1
			20 mm JM E	3	0.1
			14 mm JM E	4	0.0

Trial section	Chainage (m)	Lane	Mix type	Layer	Moisture content (%)
4	9,600	R1	20 mm JM A	1	0.1
			20 mm JM A	2	0.2
			20 mm JM A	3	0.1
			14 mm JM B	4	0.1
	9,850	R1	20 mm JM A	1	0.2
			20 mm JM A	2	0.1
			20 mm JM A	3	0.1
			14 mm JM B	4	0.1

8.4.4 Waterproofing Seal

A temporary waterproofing seal i.e. 10/5 mm CRS 170/60 emulsion 2 coat seal was placed on the 20 mm intermediate course due to inclement weather for part of the trial on 30 April 2015 covering work from 20, 22, 23, 28 and 30 April 2015 on trial Sections 1, 2 and 4. This temporary seal was cold planed prior to the continuation of asphalt placement on 06/05/2015. Table 8.11 summarises the seal application rates.

Table 8.11: Seal application rates

Start chainage (m)	End chainage (m)	Lane	Application rate (L/m²@15 °C)
S35E PMB seal			
9,300	9,877	R1	1.41
		R2	1.40
		R3	1.38
S20E PMB seal			
9,150	9,200	R1	1.83
9,050	9,250	R2	1.80
9,250	9,300	R3	1.70

8.5 Testing after Opening to Traffic

8.5.1 Coring of Trial Sections

The Gateway trial sections on Tonkin Highway were opened to traffic in September 2015.

To evaluate the trial mixes, 150 mm diameter cores were extracted from the trial sections in May 2015, October 2015, October 2016 and December 2018 from each lane. Dry coring was used to drill and extract core samples. Note that the May 2015 moisture data indicates the moisture content prior to opening the trial sections to traffic; however, it is presented with other moisture data for comparison and to show the fluctuation of moisture in asphalt over time.

8.5.2 In Situ Moisture Content

The core specimens were sampled from the Gateway field trial sites and tested in accordance with test method WA 705.1 (Main Roads 2013b). Table 8.12 summarises the core moisture content test results.

Comparison of moisture data shows that:

- The highest moisture content was observed in lift/layer 5 in all trial sections which is asphalt surfacing composed of 40 mm DGA and 30 mm OGA on the top. The highest moisture content values were recorded in October 2016 in Section 1 and December 2018 in Sections 2, 3 and 4.

- The lowest moisture contents occurred in May and October 2015 in all trial sections (within 6 months after construction). It can be deduced from the data that moisture content was steadily increasing with the passage of time with the highest values recorded in December 2018 in all sections. There was no moisture-related data reported after 2018.
- In terms of the intermediate course moistures, all values were low and stripping of any mix is currently unlikely. Section 3 (job mix E) and Section 4 (2nd generation mix) had generally higher moisture contents compared to the 3rd generation mix (Section 2). Most likely this is due to the higher air voids (4.0–6.0%) and lower binder content (4.8% ± 0.3%) in the 14 mm DGA 2nd generation mix as compared to the air voids (3.5–5.5%) and binder content (5.0% ± 0.3%) in the 14 mm DGA 3rd generation mix.
- Moisture data indicates that the temporary seal placed on the intermediate course after construction prior to surfacing performed well in terms of preventing moisture ingress into the pavement during construction.

Table 8.12: In situ core moisture content

Trial section	Lift	Mean moisture content (%) (May 2015)				Mean moisture content (%) (Oct 2015)				Mean moisture content (%) (Oct 2016)				Mean moisture content (%) (Dec 2018)			
		Lane				Lane				Lane				Lane			
		R3	R2	R1	0.2m FES	R3	R2	R1	0.2m FES	R3	R2	R1	0.2m FES	R3	R2	R1	0.2m FES
Section 1	1	–	–	0.2	–	0.5	0.2	0.3	0.6	1.0	0.7	0.5	0.8	0.9	0.9	0.7	0.9
	2	–	–	0.1	–	0.2	0.1	0.1	0.2	0.5	0.7	0.4	0.4	0.3	0.8	0.5	0.7
	3	–	–	0.1	–	0.1	0.1	0.0	0.1	0.4	0.3	0.2	0.2	0.3	0.3	0.2	0.2
	4	–	–	0.1	–	0.2	0.1	0.1	0.2	0.6	0.2	0.1	0.2	0.8	0.4	0.3	0.2
	5	–	–	–	–	1.4	2.4	1.5	1.0	2.9	2.6	2.1	1.3	2.7	2.6	2.1	1.0
Section 2	1	0.2	–	0.1	–	0.3	0.3	0.2	0.6	0.6	0.7	0.5	0.8	0.6	0.7	0.5	1.0
	2	0.2	–	0.1	–	0.3	0.1	0.1	0.3	0.6	0.4	0.4	0.3	0.5	0.4	0.3	0.3
	3	0.1	–	0.1	–	0.1	0.1	0.1	0.2	0.4	0.3	0.2	0.2	0.2	0.6	0.2	0.2
	4	0.1	–	0.1	–	0.1	0.2	0.1	0.1	0.2	0.3	0.3	0.1	0.5	0.6	0.5	0.1
	5	–	–	–	–	1.8	1.8	1.6	0.7	3.1	3.0	2.5	0.9	3.8	2.5	1.6	0.2
Section 3	1	–	–	0.2	–	0.2	0.1	0.1	0.4	0.3	0.2	0.2	0.5	0.3	0.3	0.3	0.5
	2	–	–	0.1	–	0.1	0.0	0.1	0.6	0.1	0.0	0.3	0.7	0.1	0.1	0.4	0.7
	3	–	–	0.1	–	0.2	0.1	0.1	0.2	0.4	0.2	0.3	0.2	0.4	0.3	0.2	0.4
	4	–	–	0	–	0.3	0.7	0.1	0.2	0.9	1.1	0.8	0.1	1.0	1.5	0.9	0.2
	5	–	–	–	–	1.9	1.4	1.6	0.6	3.5	2.6	2.9	0.7	4.3	2.9	3.3	0.3
Section 4	1	0.2	0.3	0.2	0.58	0.2	0.3	0.2	0.6	0.6	1.4	0.4	0.8	0.4	0.9	0.6	0.8
	2	0.2	0.1	0.1	0.25	0.2	0.1	0.1	0.3	0.6	0.7	0.2	0.3	0.6	0.7	0.7	0.4
	3	0.1	0.1	0.1	0.10	0.1	0.1	0.1	0.1	0.6	0.7	0.2	0.3	0.6	1.0	0.4	0.4
	4	0.1	0.1	0.1	0.10	0.1	0.1	0.1	0.1	0.5	0.6	0.2	0.3	0.5	1.2	0.4	0.5
	5	–	–	–	–	1.5	1.7	1.8	0.5	2.5	2.8	2.9	0.7	3.0	3.5	3.0	1.5

Notes:

- Sections 1 and 4 have 2nd generation asphalt mixes i.e. JM A (20 mm) and JM B (14 mm) whereas Section 2 has 3rd generation asphalt mixes i.e. JM C (20 mm) and JM D (14 mm).
- Section 3 has JM E asphalt mixes (20 mm and 14 mm).

Figure 8.2 to Figure 8.5 show in situ core moisture content for Sections 1 to 4.

Figure 8.2: In situ core moisture content for Section 1 for each lane

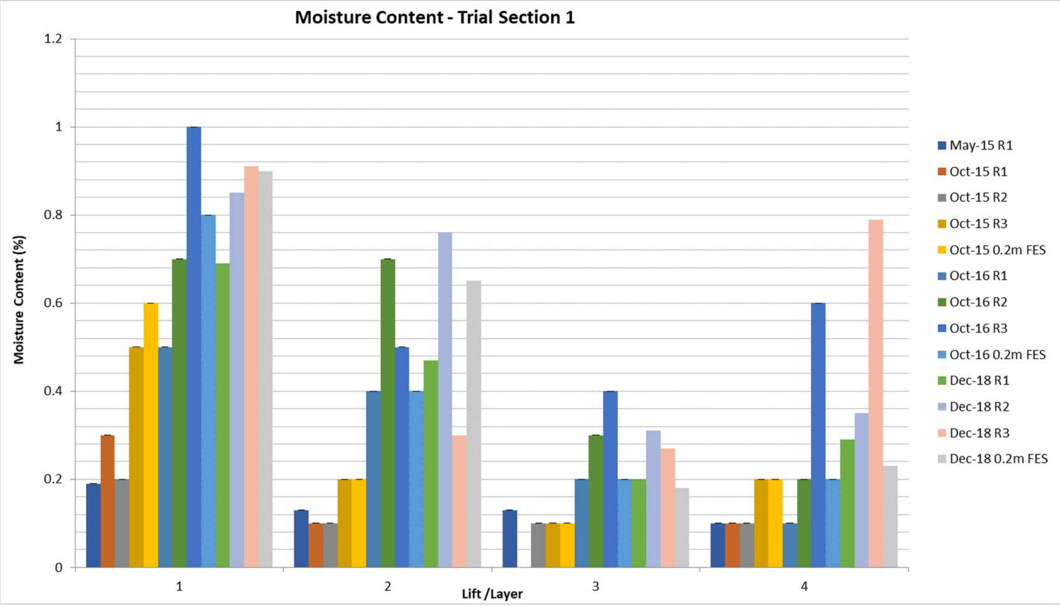


Figure 8.3: In situ core moisture content for Section 2 for each lane

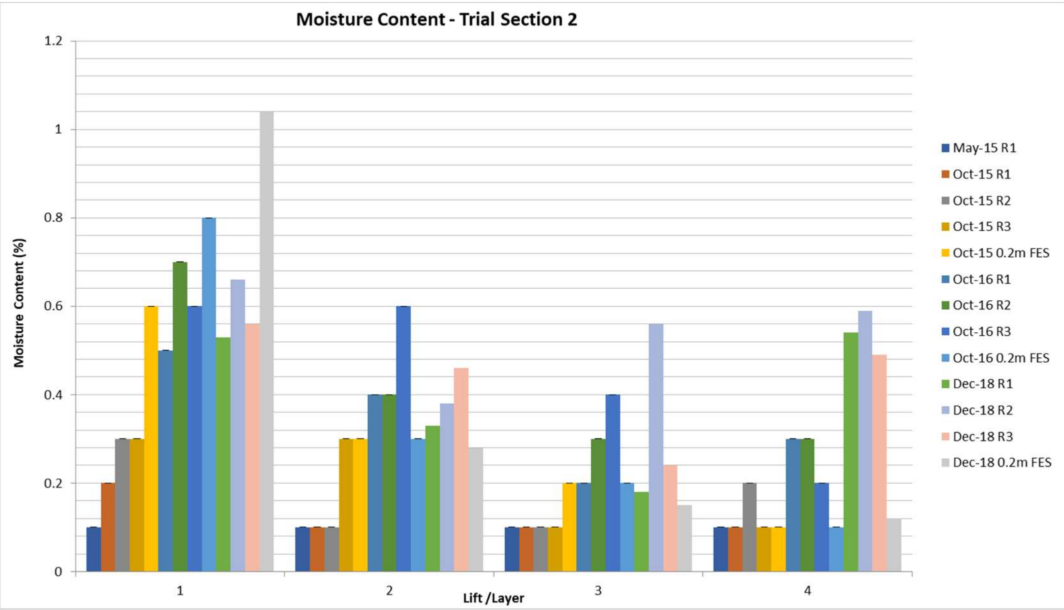


Figure 8.4: In situ core moisture content for Section 3 for each lane

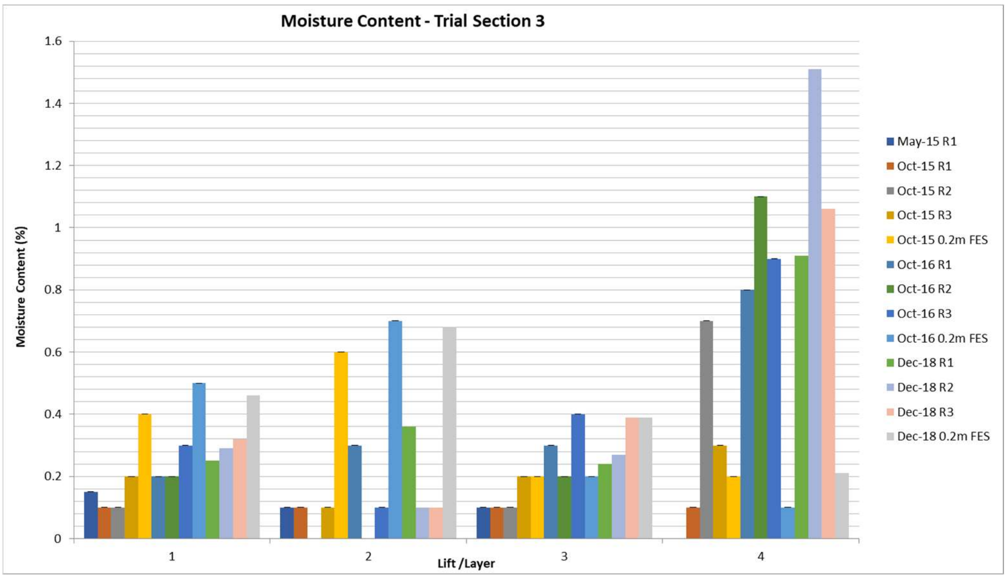
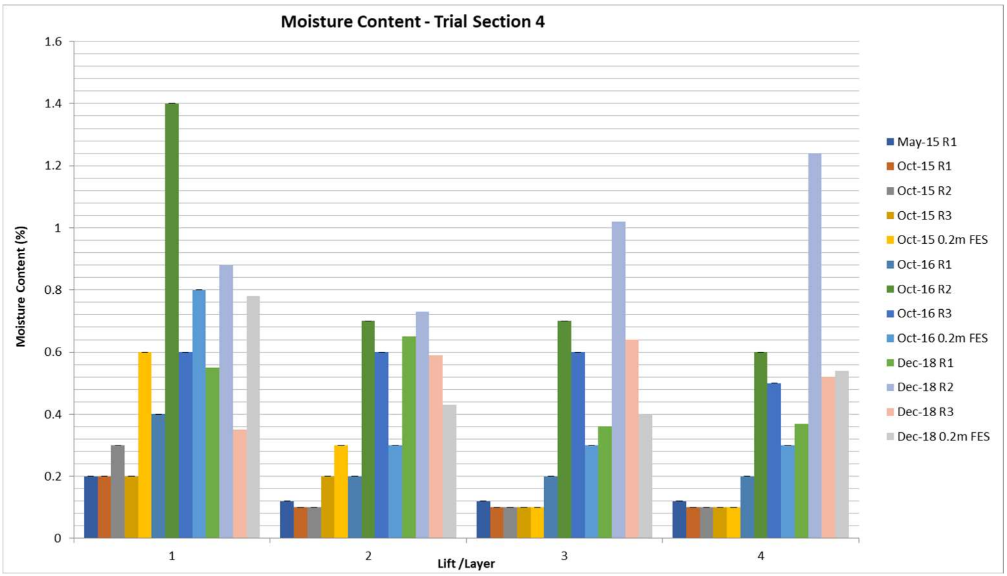


Figure 8.5: In situ core moisture content for Section 4 for each lane



8.5.3 Tensile Strength

Wet and dry tensile strength testing for 14 mm and 20 mm intermediate mixes was undertaken in accordance with test method AG:PT/T232 (Austroads 2007a). The ratios of the wet to dry strengths, and the TSR were also determined (Table 8.13).

Table 8.13: Tensile strength ratios of asphalt mixes

Section	Date	Sample number	Mix ID	Mix generation	Wet strength (kPa)	Dry strength (kPa)	TSR (%)
14 mm intermediate mix							
1	12/05/2015	S5248	JM B	2nd	958	979	98
2	11/05/2015	S5226	JM D	3rd	1,027	1,049	98
3	08/05/2015	S5208	JM E	–	984	974	101
4	06/05/2015	S5162	JM B	2nd	720	850	85
5	08/05/2015	S5201	JM B	2nd	832	918	91
20 mm intermediate mix							
1	14/04/2015	S5039	JM A	2nd	804	1,040	78

Section	Date	Sample number	Mix ID	Mix generation	Wet strength (kPa)	Dry strength (kPa)	TSR (%)
1	29/04/2015	S5126	JM A	2nd	941	976	97
2	30/04/2015	S5130	JM C	3rd	991	1,082	92
3	15/05/2015	S5186	JM E	–	962	994	97
4	20/04/2015	S5063	JM A	2nd	951	1,043	92
4	28/04/2015	S5092	JM A	2nd	775	918	85

Figure 8.6 and Figure 8.7 show a comparison of wet and dry strength and TSR for 14 mm and 20 mm job mixes respectively.

Figure 8.6: Comparison of wet and dry strength and TSR for 14 mm asphalt mixes

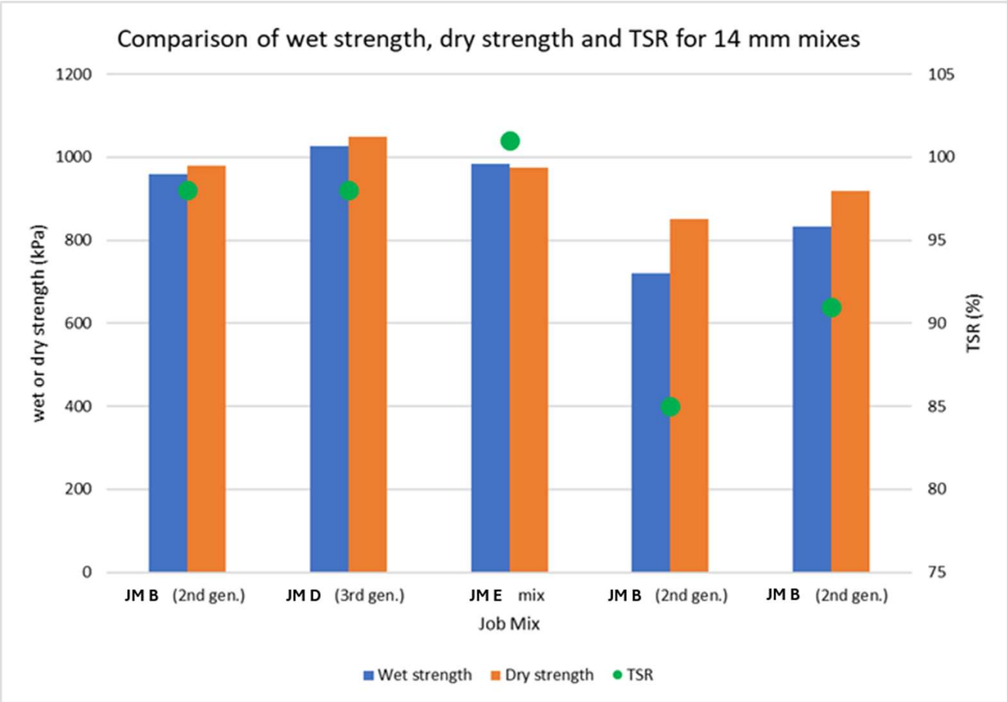
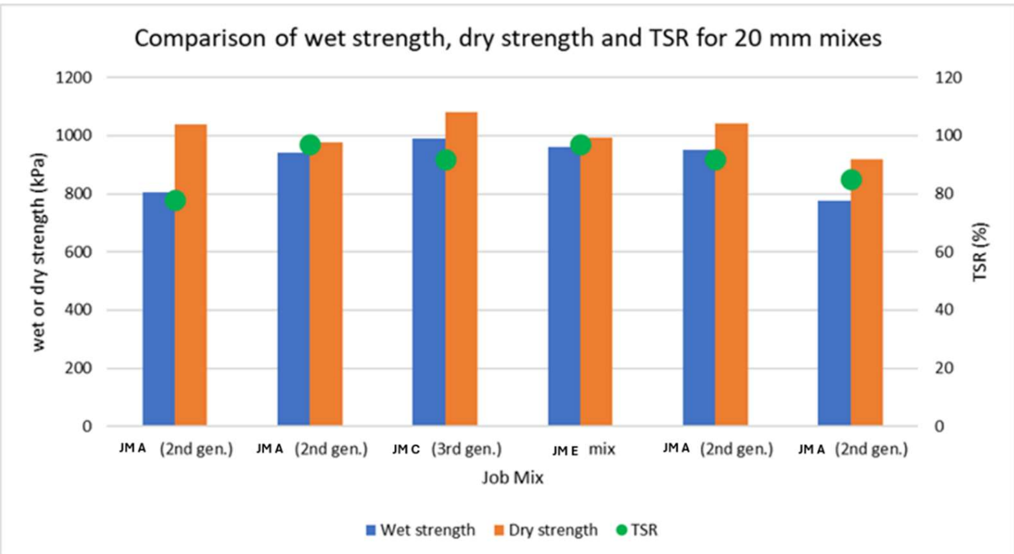


Figure 8.7: Comparison of wet and dry strength and TSR for 20 mm asphalt mixes



The 14 mm intermediate mix tensile strength shows that the ratios between wet and dry tensile strengths were similar, resulting in a good TSR. The JM B (Section 1) on 06 May 2015 was the lowest wet strength. JM D (Section 2) had the highest wet and dry tensile strengths. In comparison, JM D, with the highest binder content had the highest tensile strengths. Moreover, the 2nd generation 14 mm mix (JM B) had lower wet and dry strengths as compared to the 3rd generation mix (JM D); however, the TSRs are close to each other.

For the 20 mm intermediate mix, the ratio between wet and dry tensile strengths for the JM A mix (2nd generation mix in Sections 1 and 4) were variable with some lower wet strengths. The others have close TSRs. The JM A mix on 28 April 2015 had the lowest wet strength. The JM C (Section 2) had the highest wet and dry tensile strengths. In comparison, JM A, with the highest binder content had the highest tensile strength. Moreover, the 2nd generation 20 mm mix (JM A) had slightly lower wet and dry strengths as compared to the 3rd generation mix (JM C). However, the TSRs were in a similar range.

Overall, the tensile strength results indicate that the 3rd generation mixes have higher wet and dry strengths suggesting that there is a clear improvement in tensile strength compared to the 2nd generation mix designs.

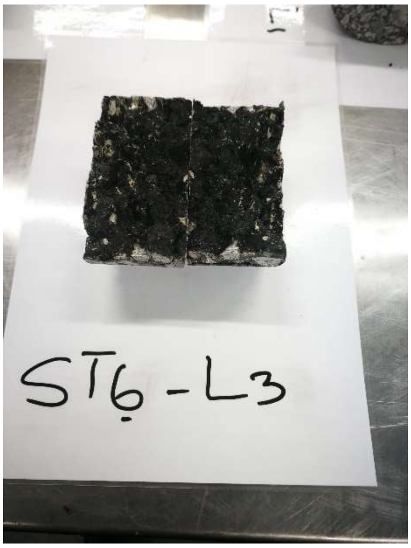
TSR values for the asphalt mixes tested were above the Main Roads specified limit of 80% (minimum) except for the 2nd generation 14 mm intermediate mix on 14 May 2015 which showed a TSR value of 78%. TfNSW, TMR and DoT Vic also specify an 80% TSR limit. However, DIT does not specify a TSR requirement.

Figure 8.8 shows photographs of the cores from TSR testing from the Main Roads 2018 investigation at the Gateway trials on Tonkin Highway.

Figure 8.8: Photographs of TSR cores

L1 – TSR core

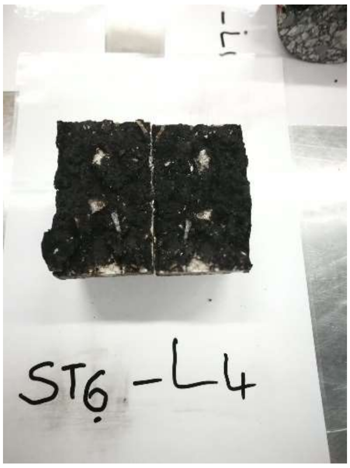
L1 – TSR core



L3 – TSR core

L3 – TSR core

L5 – TSR core



L4 – TSR core

Source: Main Roads.

8.5.4 Air Voids and Permeability

The air void content in asphalt is a critical factor in asphalt stripping. Asphalt mixes are generally considered impermeable below 4% and free draining above 9% air voids in compacted mix. Unfortunately, pavements are constructed at pessimum voids – the area where the asphalt mix is more prone to stripping (generally 4 to 7% air voids). If pavement is constructed above 8% air void content, there are less chances of stripping due to the pavement's free-draining nature; however, there is a high risk of premature surface ravelling due to lack of cohesion as reported by Kandhal (1992).

The permeability of asphalt was tested in accordance with test method Q304A (TMR 2022e). The air void content of asphalt was determined in accordance with test method WA 733.1 (Main Roads 2022d).

Table 8.14 summarises the intermediate mix air voids and permeability results as well as the assigned permeability level.

Table 8.14: Core air voids and permeabilities of intermediate course mixes

Mix ID	Air voids (%)	Permeability (μm/s)	Category[1]
14 mm intermediate mix			
JM B	6.0	7.24	Moderately permeable
JM B	7.1	161	Moderately free draining
JM B	6.3	1.40	Moderately permeable
JM B	5.5	< 0.01	Very low permeability
JM B	5.3	2.11	Moderately permeable
JM D	4.9	< 0.01	Very low permeability
JM D	4.6	0.15	Low permeability
JM E	6.8	5.94	Moderately permeable
JM E	6.7	3.35	Moderately permeable
20 mm intermediate mix			
JM A	6.8	3.44	Moderately permeable
JM A	5.8	< 0.01	Very low permeability
JM A	4.6	< 0.01	Very low permeability
JM A	6.6	0.55	Low permeability
JM A	7.3	231	Moderately free draining
JM A	6.7	19.7	Permeable
JM A	3.9	< 0.01	Very low permeability
JM A	5.8	1.16	Moderately permeable
JM A	7.3	17.7	Permeable
JM A	4.3	< 0.01	Very low permeability
JM A	5.0	3.99	Moderately permeable
JM A	5.0	7.31	Moderately permeable
JM A	6.2	3.40	Moderately permeable
JM A	3.0	0.02	Very low permeability
JM A	6.0	< 0.01	Very low permeability
JM C	3.1	0.08	Very low permeability
JM C	5.6	0.17	Very low permeability
JM C	6.2	1.37	Moderately permeable
JM C	5.3	2.19	Moderately permeable
JM C	4.8	0.10	Very low permeability
JM C	3.1	< 0.01	Very low permeability

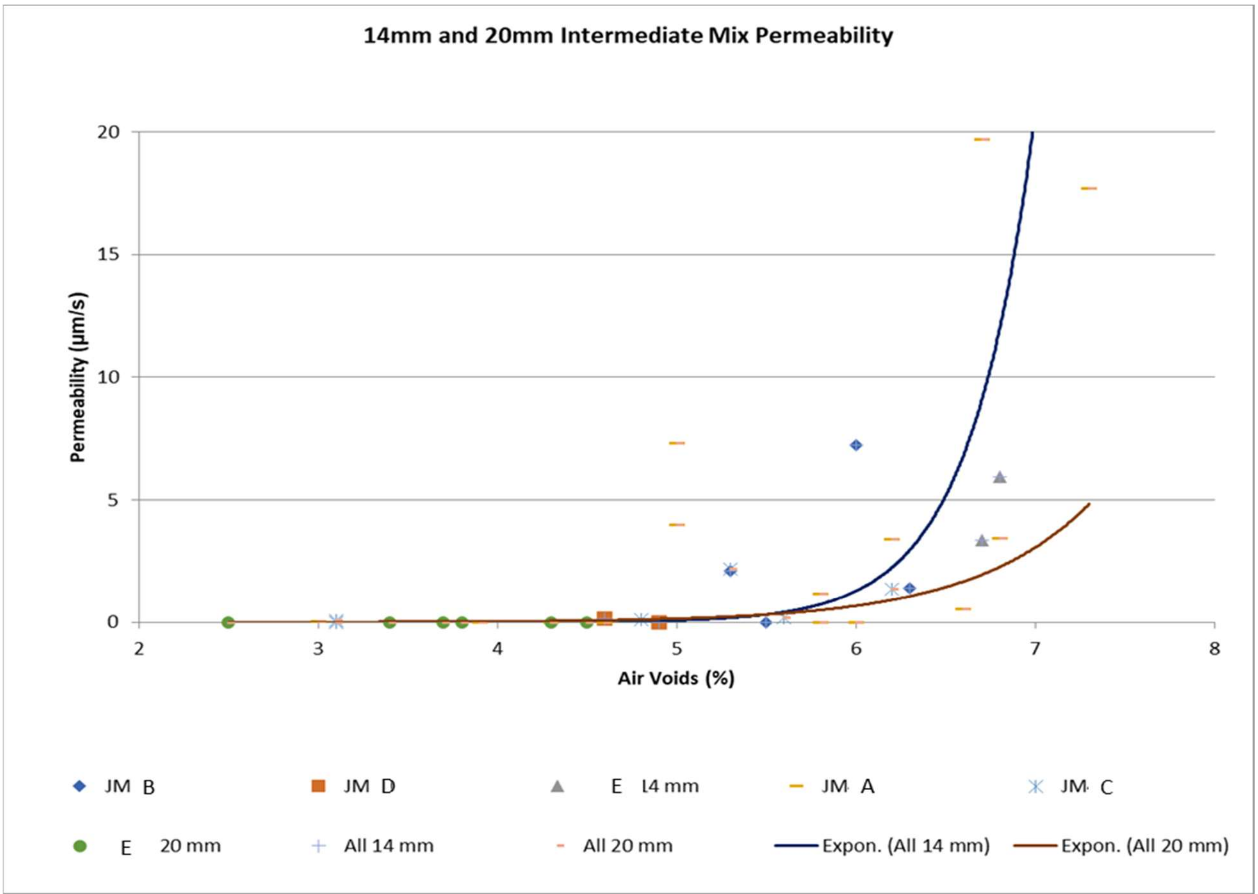
Mix ID	Air voids (%)	Permeability (µm/s)	Category(1)
JM E	3.7	< 0.01	Very low permeability
JM E	3.4	< 0.01	Very low permeability
JM E	4.5	< 0.01	Very low permeability
JM E	2.5	< 0.01	Very low permeability
JM E	4.3	< 0.01	Very low permeability
JM E	3.8	< 0.01	Very low permeability

1. The category of the permeability level is based on TMR Test Method Q304A (TMR 2022e).

In terms of the 14 mm mixes, the JM D mix (3rd generation, Section 2) showed the lowest permeability. For the 20 mm mixes, the job mix E (Section 3) showed the lowest and most consistent permeability. It also had the highest field compaction (Table 8.7). The 2nd generation mix (JM A) had the highest permeability of all the mixes.

Figure 8.9 shows the comparison between the air voids and permeability of each mix. Note that the 2 highest permeability values of 161 µm/s and 231 µ/ms are not shown graphically for scaling. As expected, permeability is related to air voids – the lower the voids, the lower the permeability.

Figure 8.9: 20 mm and 14 mm intermediate mix air voids and permeability



8.5.5 Deformation Resistance (Wheel Tracking Test)

The deformation resistance test was conducted on each mix in accordance with AG:PT/T231 (Austroads 2006b) at 10,000 passes as per the standard method and at 60,000 passes as a variation to the standard test. To reduce the number of slab specimens to be prepared, the specimens were tested to 60,000 passes and the data at 10,000 passes was extracted. In addition, 150 mm diameter core specimens were sampled from the field trial sites and tested in accordance with the test method at 10,000 passes.

Note that AG:PT/T231 requires a minimum core diameter of 200 mm; however, core specimens sampled from the sites had a 150 mm diameter.

Table 8.15 summarises the test results for the 14 mm and 20 mm intermediate mix asphalt tested at 10,000 passes, 14 mm and 20 mm intermediate mix asphalt core specimens tested at 10,000 passes and 14 mm and 20 mm intermediate mix asphalt tested at 60,000 passes.

Table 8.15: Summary of wheel tracking test results

Section	Date	Sample number	Mix ID	Mix generation	Air voids (%)	Maximum tracking depth (mm)	Mean maximum tracking depth (mm)
14 mm intermediate mix slabs – 10,000 passes							
2	11/05/2015	S5235	JM D	3rd	4.9	1.3	1.05
					4.5	0.8	
		S5242	JM D	3rd	4.6	1.6	1.35
					4.1	1.1	
3	08/05/2015	S5216	JM E	–	5.1	1.4	1.55
					5.9	1.7	
4	08/05/2015	S5214	JM B	2nd	5.5	1.6	1.40
					5.2	1.2	
14 mm intermediate mix cores – 10,000 passes							
2	08/05/2015	S5262	JM D	3rd	6.6	1.7	1.80
					3.2	1.9	
3	20/04/32015	S5217	JM E	–	8.5 7.1	3.0 3.0	3.00
4	11/05/2015	S5220	JM B	2nd	5.0	2.0	2.60
					5.8	3.2	
14 mm intermediate mix slabs – 60,000 passes							
2	11/05/2015	S5235	JM D	3rd	4.9	1.5	1.30
					4.5	1.1	
		S5242	JM D	3rd	4.6	2.1	1.80
					4.1	1.5	
3	08/05/2015	S5216	JM E	–	5.1	1.9	2.05
					5.9	2.2	
4	08/05/2015	S5214	JM B	2nd	5.5	2.0	1.75
					5.2	1.5	
20 mm intermediate mix slabs – 10,000 passes							
2	22/04/2015	S5085	JM C	3rd	4.9	2.5	2.50
					4.5	2.5	
3	07/05/2015	S5181	JM E	–	4.6	3.5	2.80
					4.5	2.1	
4	30/04/2015	S5137	JM A	2nd	5.1	2.6	2.25
					5.4	1.9	

Section	Date	Sample number	Mix ID	Mix generation	Air voids (%)	Maximum tracking depth (mm)	Mean maximum tracking depth (mm)
20 mm intermediate mix cores – 10,000 passes							
2	20/04/2015	S5262	JM C	3rd	7.1	2.3	2.30
					8.5	2.6	
	29/04/2015				4.2	2.5	
					4.5	1.9	
	30/04/2015				5.8	2.1	
					4.3	2.4	
3	20/04/2015	S5217	JM E	–	8.5	3.0	2.50
					7.1	3.0	
	20/04/2015				3.1	2.6	
					2.7	2.7	
	20/04/2015				4.2	1.9	
					3.9	2.0	
4	21/04/2015	S5220	JM A	2nd	4.1	2.2	2.50
					5.9	1.7	
	20/04/2015				10.0	2.8	
					7.1	2.7	
	30/04/2015				7.1	2.9	
					8.5	2.5	
20 mm intermediate mix slabs – 60,000 passes							
2	22/04/2015	S5085	JM C	3rd	4.9	3.5	3.60
					4.5	3.7	
3	07/05/2015	S5181	JM E	–	4.6	4.9	3.95
					4.5	3.0	
4	30/04/2015	S5137	JM A	2nd	5.1	3.8	3.30
					5.4	2.8	

Figure 8.10 shows the deformation resistance (maximum tracking depth) of the 14 mm mixes after 10,000 and 60,000 passes for standard slab specimens, while Figure 8.11 shows mix core results. Similarly, Figure 8.12 and Figure 8.13 show deformation resistance for the 20 mm mixes for mix slabs and mix cores respectively.

Figure 8.10: Maximum tracking depth for 14 mm intermediate asphalt mix slabs

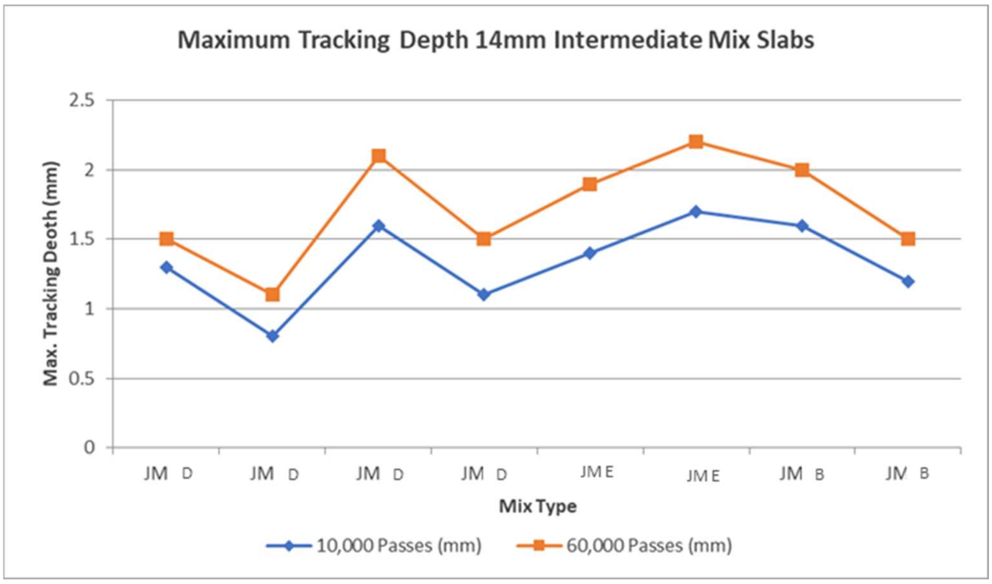


Figure 8.11: Maximum tracking depth for 14 mm intermediate mix cores

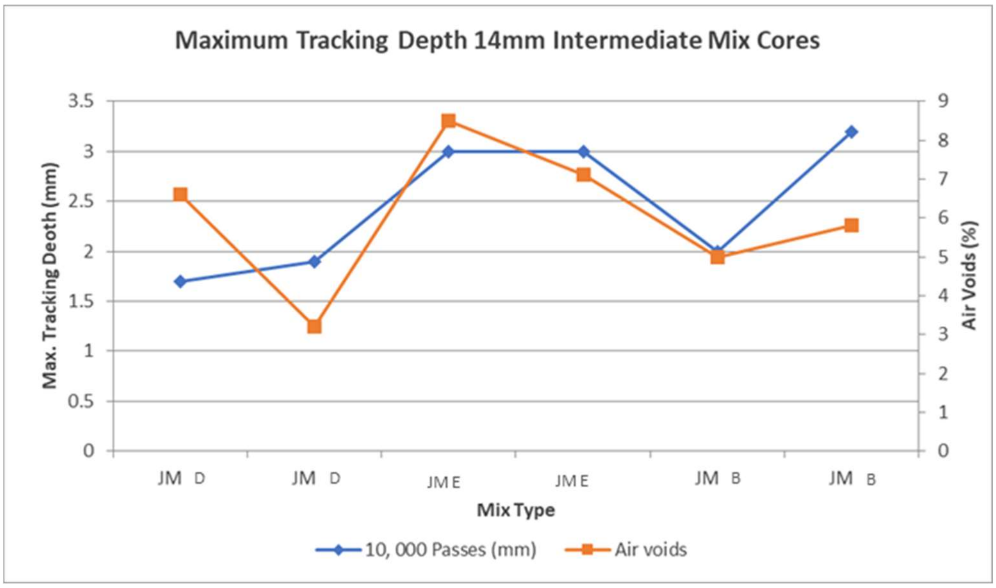


Figure 8.12: Maximum tracking depth for 20 mm intermediate asphalt mix slabs

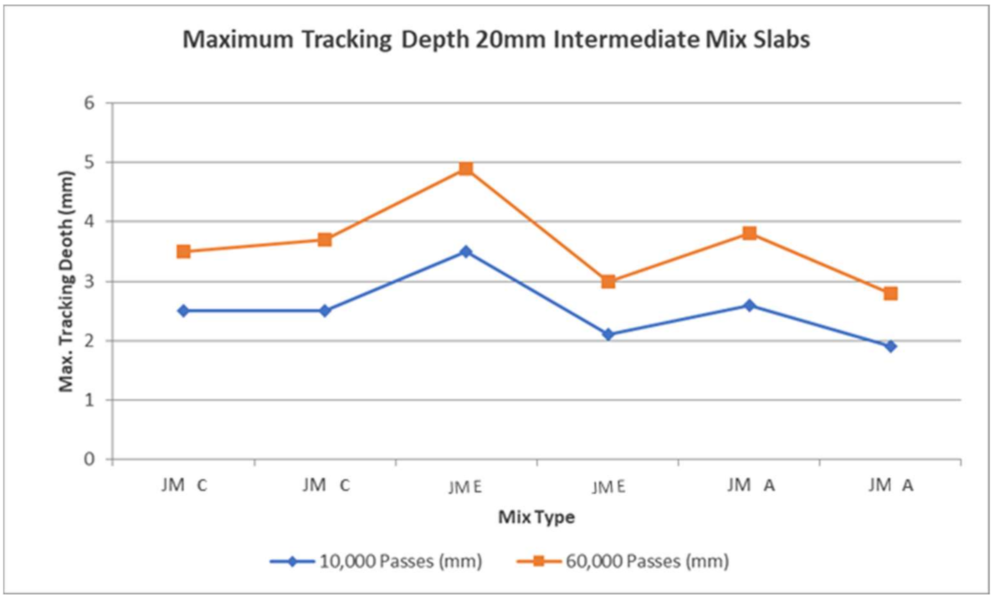
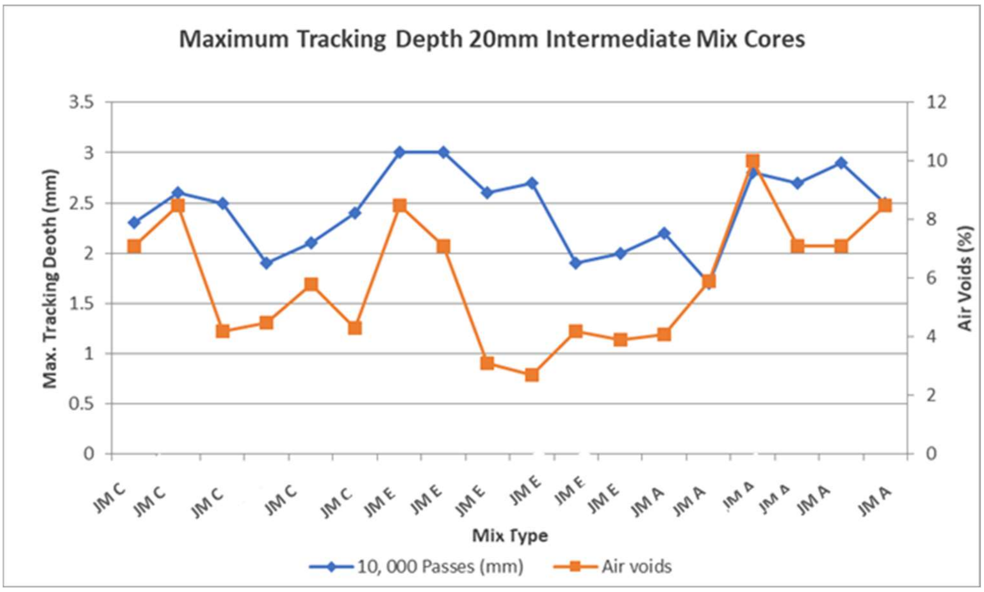


Figure 8.13: Maximum tracking depth for 20 mm intermediate asphalt mix cores



The deformation of the 14 mm intermediate mixes at 10,000 passes was similar. The core specimens from the field shown in Figure 8.11 were typically higher than the slab specimens. As expected, the maximum tracking depth varies with the air voids. Air voids were determined prior to the wheel tracking test.

Similarly, the deformation of all the 20 mm intermediate mixes at 10,000 passes was also close. Figure 8.12 shows that most of the deformation occurs on the first 10,000 passes, with a consistent increase around 1 mm at 60,000 passes.

Overall, the 14 mm mixes produced using A15E PMB were less prone to deformation than the 20 mm mixes produced with Class 600 bitumen.

Main Roads Specification 510 requires deformation resistance (AG:PT/T231) of a maximum 4 mm for the design target. Comparison of the wheel tracking test results for the 14 mm and 20 mm asphalt intermediate mixes shows that the maximum tracking depth reported for all samples tested is well below this limit. TMR and DoT Vic also specify 4 mm for the design target. DIT specifies 3 mm to 6 mm whereas TfNSW does not specify any limit for the wheel tracking test.

8.5.6 Hamburg Wheel Tracking Test Results

Main Roads engaged the Australian Road Research Board (ARRB) to perform Hamburg wheel tracking testing for the Gateway field trials. The testing was carried out in accordance with AASHTO T324 at the TMR laboratory in Brisbane on a series of field-cored samples and laboratory-manufactured slab specimens.

Fifteen pairs of cores sampled by the Main Roads Material Engineering Branch (MEB) and 14 slabs produced at ARRB were tested using the Hamburg wheel tracker device (HWTD). The test parameters were similar to AG:PT/T231, with inclusion of the slabs being immersed in a 60 °C water bath during testing. The slabs were compacted to 5 ± 1% air voids, using the maximum and bulk densities for each sample as supplied by Main Roads. Photographs of the HWTD are shown in Figure 8.14.

Figure 8.14: HWTD with sample covers (left) and with sample loaded (right)

Source: ARRB Contract Report Project – 010978.

Three sets of cores failed prematurely (before the test ended) including 2 sets of JM A (2nd generation mix) and one set of the AAPA 20 mm mix. Table 8.16 summarises the test results.

Table 8.16: Hamburg wheel tracking test results

Section	Mix Id	Mix generation	Average rut depth (mm)(1)		Average rut depth (mm)(2)	
			Cores	Slabs	Cores	Slabs
2	JM A (20 mm)	3rd	9.1	15.8	9.1	15.8
3	JM E (20 mm)	–	12.8	10.9	11.4	10.9
4	JM A (20 mm)	2nd	13.2	8.4	10.0	8.4
2	JM D (14 mm)	3rd	4.5	5.7	4.5	5.7
3	JM E (14 mm)	–	5.5	5.2	5.5	5.2
4	JM B (14 mm)	2nd	9.5	4.3	9.5	4.3

1. Including stripped samples.
2. Excluding stripped samples.

Figure 8.15 to Figure 8.18 show the results for 14 mm intermediate mix slabs, 14 mm intermediate mix cores, 20 mm intermediate mix slabs and 20 mm intermediate mix cores respectively.

Figure 8.15: 14 mm intermediate mix slabs

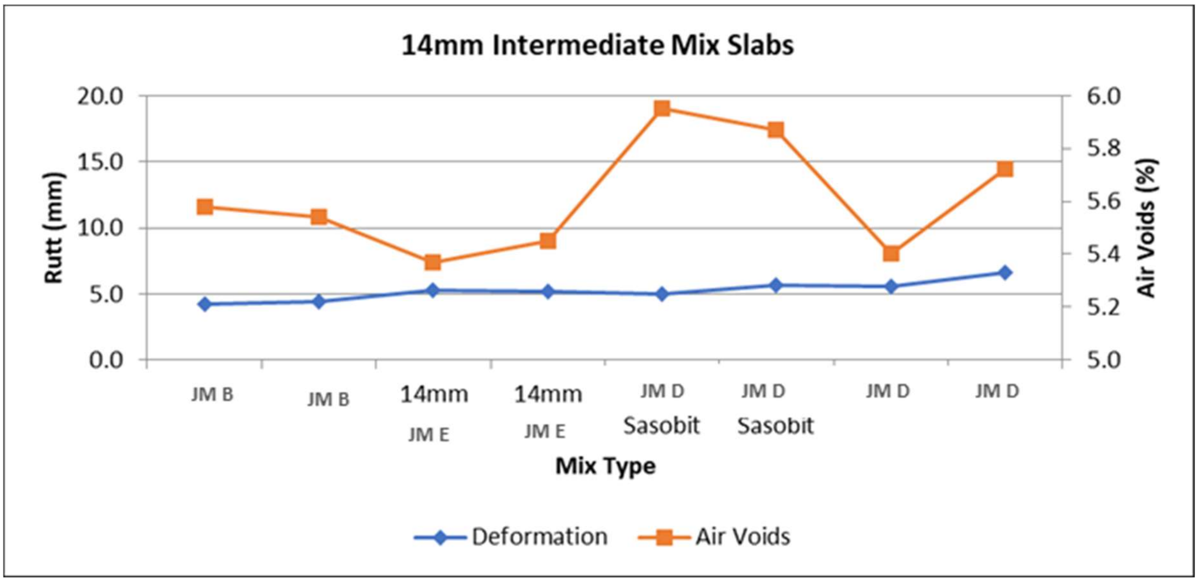


Figure 8.16: 14 mm intermediate mix cores

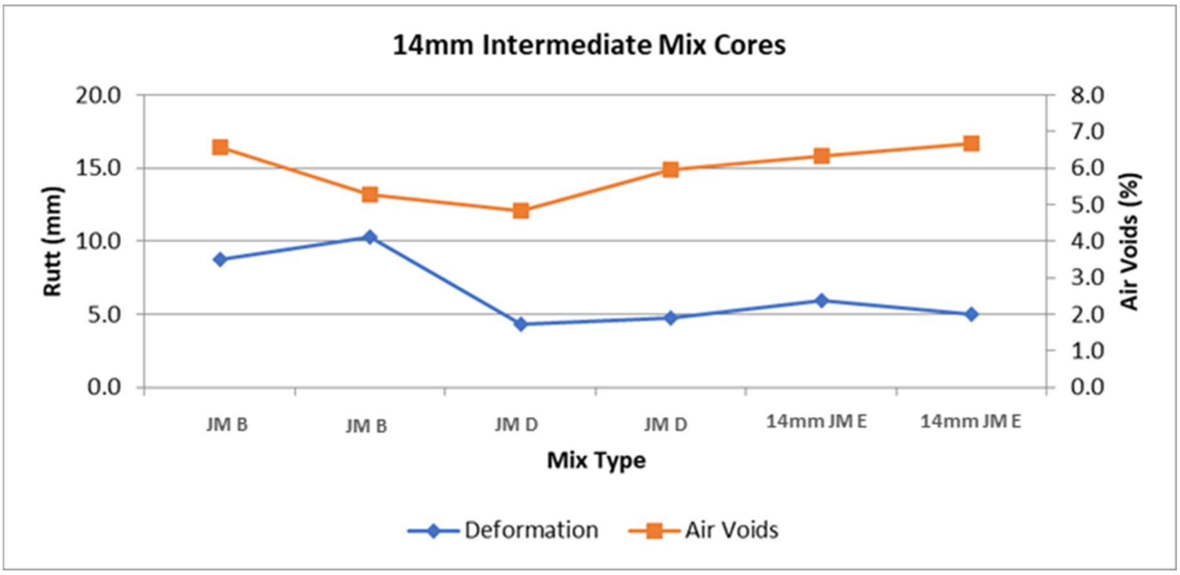


Figure 8.17: 20 mm intermediate mix slabs

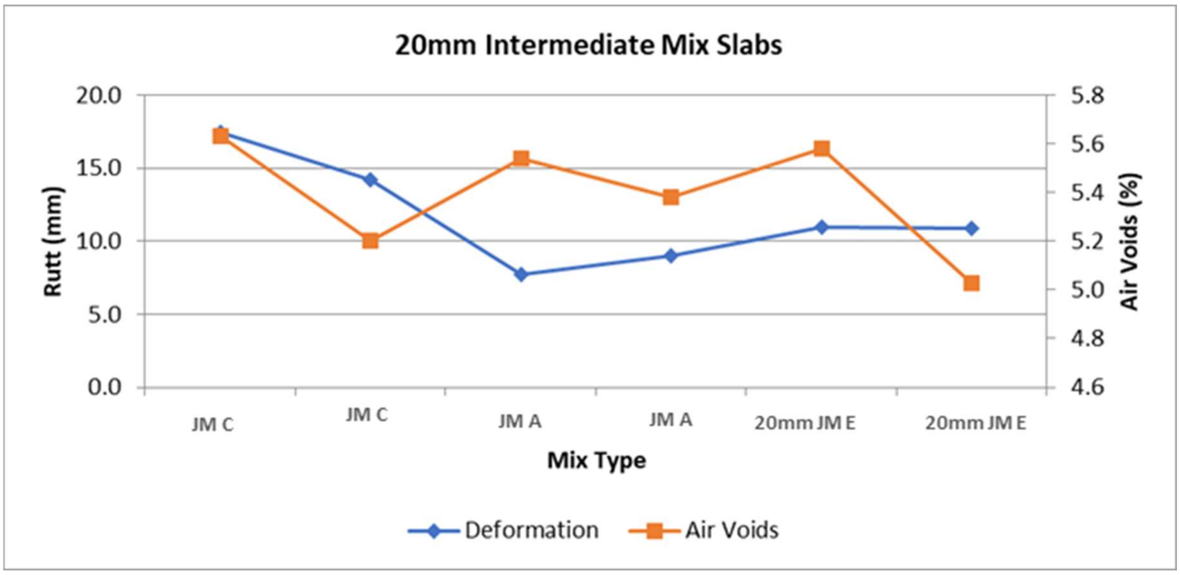
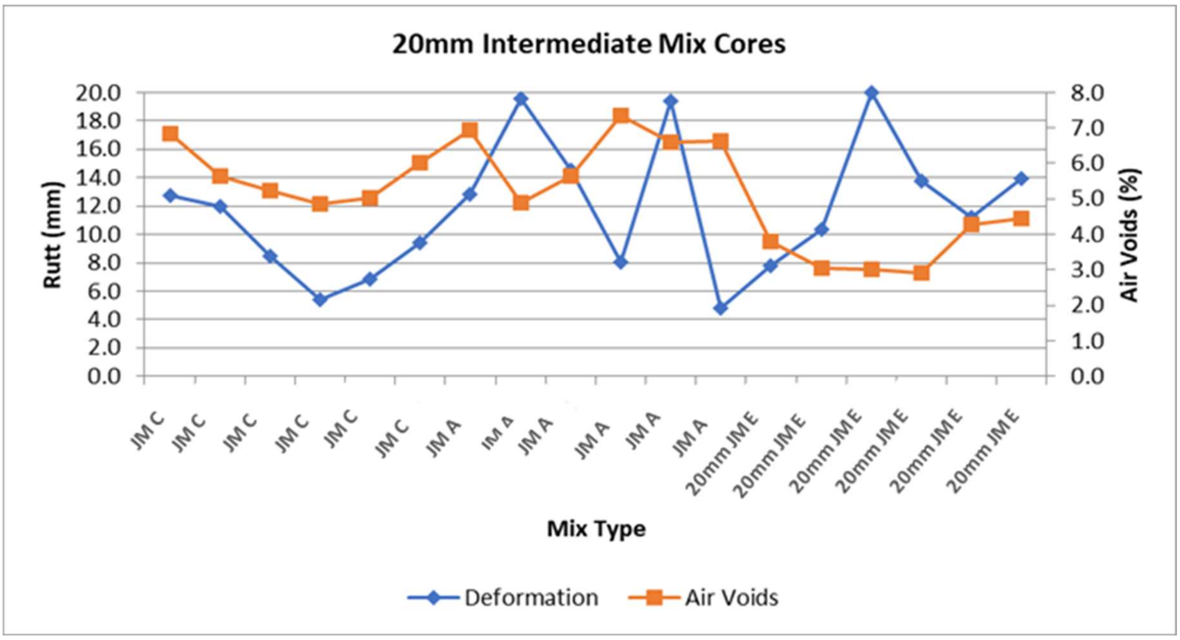


Figure 8.18: 20 mm intermediate mix cores



The JM A (2nd generation mix) from the slabs performed worst out of the 3 mixes. The 20 mm cores were varied with individual specimens from each mix type having good and bad deformation. As expected, the 14 mm intermediate mix made using A15E PMB performed better than the 20 mm intermediate mix produced with C600 asphalt.

It should be noted that Main Roads has no specification limits for Hamburg wheel tracking test results. The HWTD subjects samples to very harsh conditions that means rut depths of double or even triple the expected values under other wheel tracking tests are not necessarily indicative of poor performance (Beecroft 2015). Similarly, none of the other Australian jurisdictions specify any limits for HWTT.

8.5.7 Aggregate Petrography

In the Perth Metropolitan area, the approved mixes are commonly manufactured using crushed granite. The aggregates used in 2nd and 3rd generation job mixes (i.e. JM A, JM C, JM D, JM B) as a part of the Gateway trials are mainly composed of granite which is geochemically an acidic rock sourced from one of the quarries in Western Australia.

Austroads (2014) advises that aggregate affinity for bitumen is related to the surface chemistry of the aggregate. Bitumen must adhere to the aggregate and resist stripping in the presence of water. It is firmly established that the mineralogy and chemical composition of the aggregate are of primary importance. The acidic aggregates such as granite have greater affinity for water than for bitumen and the binder film on these aggregates may become detached upon exposure to water. The difference in behaviour is related to different electrical charges which develop on the surface of the minerals in the presence of water. Most silicious aggregates (e.g. granites) become negatively charged.

Table 8.17 summarises the petrographic details of aggregate from the Orange Grove Quarry.

Table 8.17: Petrographic analysis of the source rock

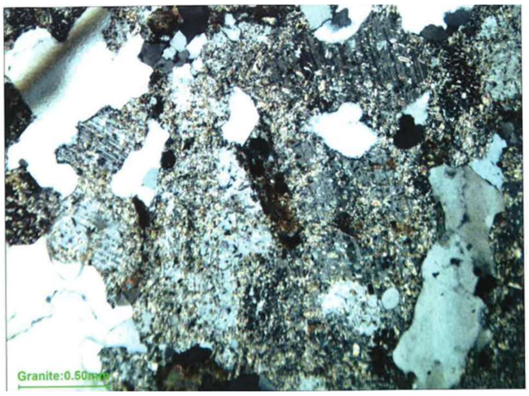
			Durable minerals		Soft, weak & deleterious minerals	
Quarry/source	Sample type	Rock type identified	Mineral	Content (%)	Mineral	Content (%)
Sample 1: Quarry in WA	Nominal 14 mm aggregate	Granite (more precisely adamellite) and subordinate meta-dolerite	Quartz	26	Sericite	5
			Plagioclase feldspar	45	Biotite	5
			K-feldspar (microcline)	5	Chlorite	1
			Epidote	8	Calcite	1
			Actinolite	4	–	–
			Sphene	1	–	–
			Others	< 1	–	–
Sample 2: Quarry in WA	Nominal 14 mm aggregate	Granite (more precisely adamellite) and meta-dolerite	Quartz	17	Sericite	6
			Plagioclase feldspar	24	Biotite	1
			K-feldspar (microcline)	16	Chlorite	1
			Epidote	12	Pyrite	1
			Actinolite	17	Calcite	< 1
			Sphene	5	–	–
			Others	< 1	–	–

Note: The purpose of the petrographic analysis of the aggregate was to assess its suitability for use as concrete aggregate and asphaltic and sealing aggregate and the potential for alkali-silica reactivity.

Figure 8.19 shows images of the supplied aggregate samples and the rocks under cross-polarised light.

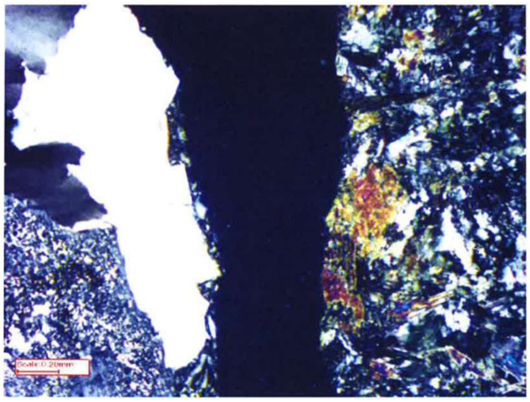
Figure 8.19: Petrographic analysis of aggregate

Sample 1: Image of supplied aggregate sample.



Sample 1: Image of granite in cross-polarised light at 4x magnification. Plagioclase crystals with sericite and epidote alteration are concentrated along the cleavage. White grains are anhedral quartz.

Sample 2: Image of supplied aggregate sample.



Sample 2: Image of 2 different rock fragments seen in the provided sample – granite (left) and greenstone (right). The image was taken at low magnification with transmitted cross-polarised light. Note sericite and epidote alteration throughout the plagioclase in the granite as well as the abundant actinolite, epidote and sphene within the greenstone.

Source: Main Roads.

Comments and interpretation:

- Sample 1: The supplied sample from the Orange Grove Quarry consists of acidic igneous granitic-style medium-grained crystalline (granite or adamellite). The rock is 85% granite, 15% greenstone (meta-dolerite) – possibly dykes and intrusions within the granite. As it contains 26% free silica (quartz), it is predicted to have the potential for mild or slow deleterious alkali-silica reactivity in concrete.
- Sample 2: The supplied sample from the Orange Grove Quarry consists of acidic igneous granitic-style medium-grained crystalline (granite or adamellite). About 40% of the sample consists of greenstone and the remaining 60% is medium-grained granite or adamellite. The free silica content was estimated to be 17%.

As reported in the literature review (Section 3), the studies indicate that Ca-feldspar and ferromagnesian minerals in aggregate relates to improved resistance of the mixture to moisture damage. On the other hand, aggregates having a high concentration of acid soluble (SiO_2 and Al_2O_3) and free silica such as albite, quartz, k-feldspar and clay are detrimental in terms of moisture sensitivity (e.g. Bagampadde 2004, Zhang et al. 2015b). The compositions of the granite aggregate samples show:

- Moderate silica content – 26 and 17% quartz and free silica respectively as quartz in granite can reach up to 60%.
- Plagioclase feldspar – 45 and 24%. However, it is not clear which plagioclase feldspar is present (e.g. albite, anorthite, oligoclase). More information is required about the type of plagioclase feldspar to establish its potential sensitivity to moisture in asphalt.
- Low k-feldspar (this mineral is sensitive to moisture damage).

Overall, the composition of the samples reported in petrographic analysis was less likely sensitive to moisture damage in asphalt. It should be noted that the aggregate composition is not the only factor in stripping. Stripping is a combined result of several factors such as the composition of aggregate, bitumen type, mix design parameters, air void content, and drainage and construction-related issues.

8.5.8 Waterproofing Seals

Main Roads Specification 510 (2022a) requires a sprayed seal to the uppermost layer of 14 mm asphalt intermediate course to waterproof the surface of the uppermost layer. However, a review of Main Roads technical documents shows that there is no requirement specified related to the application of a waterproofing seal on top of the granular subbase layer prior to the application of an asphalt intermediate course.

Analysis of the data related to the Gateway field trials shows that a temporary waterproofing seal was placed on the 20 mm intermediate mix due to inclement weather for part of the trial, then cold planed prior to the continuation of asphalt placement. The fluctuation of the in situ moisture content from 2015 to 2018 (see Figure 8.2 to Figure 8.5) indicates low moisture content in the pavement layers during different times of the year indicating Main Roads current practice is appropriate for the prevention of moisture ingress into the pavement. It should be noted that this conclusion is based on analysis of the available data related to the trials. If further study at multiple sites indicates higher moisture content in different parts of Western Australia posing serious threat to asphalt pavements in terms of potential stripping, consideration must be given to adopting waterproofing seals similar to other SRTAs to ensure adequate waterproofing and bonding between pavement layers.

8.6 Key Findings

The key findings of the data analysis are:

- The moisture content at the trials as monitored between 2015 to 2018 indicates that moisture in the asphalt intermediate course was generally low and the presence of moisture was not an issue in terms of stripping of asphalt mixes.
- Tensile strength test results indicate that the mix with higher binder content exhibits higher tensile strength. The 2nd generation mixes have lower wet strengths as compared to 3rd generation mixes. In terms of tensile strength, there is a clear improvement from the 2nd generation to the 3rd generation mix.
- The 2nd generation mix has the highest permeability of all mixes. This is due to higher air voids and lower binder content. If 2nd generation mix permeability data is discarded, the cores are relatively impermeable below 7% air voids.
- Wheel tracking test results show that the maximum tracking depth for different job mixes is generally higher for higher air voids and lower for lower air voids. Overall, the 14 mm mixes appear to be less prone to deformation as compared to 20 mm mixes. This superior deformation resistance of 14 mm mixes can be attributed to the type of binder used to manufacture the mixes (i.e. A15E PMB), whereas 20 mm mixes are produced with C600 bitumen. Similarly, the 14 mm mixes performed better in the HWTT.

9 Conclusions

A review of the critical factors that lead to stripping is presented below.

Mix design

The mix design parameters for Main Roads such as in situ air voids (3–6%), use of hydrated lime (1.5%) and testing requirements (e.g. TSR) are in line with most of the other Australian SRTAs, except that the upper limit of in situ air void content for them is 7%. Most of the US transportation agencies place asphalt at 8% in situ air voids to accommodate potential densification of the pavement under traffic during the first couple of years. Main Roads current practice for in situ air void content seems more appropriate for the following reasons:

- For a higher range of the limit (e.g. 8% or above) – poor compaction may result in air void content higher than the target upper limit leading to ravelling due to insufficient cohesion. Efficient compaction procedure is required to avoid any deviation from the target compaction. Currently, no issues have been reported in Main Roads projects regarding achieving desired levels of compaction and the standard of surface finish in colder months (i.e. May to September); however, it is advisable that Main Roads investigates the compaction practices for the colder months especially for the asphalt placed during night time and with PMB asphalt.
- For a lower range of the limit (e.g. 3% or below) – over-compaction and/or densification of the pavement under traffic loading may result in too tight a pavement with lack of space to accommodate bitumen expansion at elevated temperatures during summer.

Asphalt mix design should be tailored to produce asphalt mixes resistant to moisture ingress into the pavement and the action of water on aggregate and bitumen surfaces. The aggregate type and mineralogy are identified as the more dominant factors than the bitumen type affecting moisture sensitivity of the asphalt mix. In the Perth Metropolitan area, approved mixes are commonly manufactured using crushed granite which is categorised as hydrophilic (more prone to stripping) as compared to basic rocks.

Main Roads Specification 510 requires a detailed description of all of the materials to be used in the manufacturing of the asphalt mix including a geological description of all of the aggregates or sand to be used. The source of the materials is to be provided along with the proposed proportioning of the aggregates or sand. This clause indicates the requirements of petrographic analysis. The petrographic results can be used to evaluate the risk of stripping due to the granite aggregate. The mineral composition of aggregate is an important factor in decreasing the stripping potential and could be determined by XRF.

Discussion with selected asphalt suppliers has indicated that DoT Vic also uses granite aggregate in some of the asphalt mixes, however, there are no significant issues reported related to stripping. A detailed investigation into the matter is required. No data was found related to any investigation to ascertain whether the stripping issues are or are not associated with the use of granite aggregate in the Western Australian asphalt mixes.

In relation to the mix design:

- The report summarised the specified allowable ranges of mix volumetrics used by SRTAs. As these ranges are wide, this information does not provide insight as to whether or not the approved Main Roads mixes have similar volumetrics to those used in other SRTAs.
- Main Roads specifies a hydrated lime content of 1.5%. TfNSW specifies a minimum lime content of 1.5%. NSW requirements are of particular interest given the issues with stripping over more than 20 years (AAPA 2005, Kandhal & Rickards 2001).

- Main Roads specifies TSR testing for assessing the stripping potential of asphalt using the same test method and minimum requirements as DoT Vic and TMR. However, TfNSW uses RMS T640 (2012c) for TSR testing. Main Roads, TMR and TfNSW conduct TSR tests with freeze-thaw conditioning. The HWTT is used by several US transportation agencies to estimate the moisture resistance of asphalt mixes. Main Roads may consider using the test and the Resistance to Stripping of Aggregates and Binders test (RMS T230 (2012b)) as an indicator of stripping potential. Other tests that could be considered are related to aggregate adhesion with bitumen such as the rolling bottle , SATS , PATTI and the immersion-mechanical test.
- Permeability of the asphalt mix plays a pivotal role in water flow through interconnected voids of the mix and causes stripping.

Laboratory tests

Review of laboratory tests related to asphalt stripping identified the following laboratory tests:

- Review of international road agencies (e.g. US transportation agencies) testing requirements shows that the HWTT can be used as an indicator of stripping potential. Currently Main Roads does not use resistance to stripping of aggregates and a binder test in its asphalt specification (e.g. Main Roads Specification 510 (2022a)).
- Austroads (2014) recommends AGPT/T232 Stripping Potential of Asphalt: Tensile Strength Ratio as a preferred test for assessing the moisture sensitivity of asphalt. This is based on the Lottman test and is not totally successful in the prediction of stripping in asphalt pavements but is regarded as the best currently available for this purpose.
- TfNSW (RMS 2012b) uses test method T230 Resistance to Stripping of Aggregates and Binders in the presence of water (also known as the plate test or stripping test). The test has limitations as it is not applicable to aggregate particles which pass a 9.50 mm sieve. Main Roads investigated this test previously and identified several problems e.g. the plate needs to be clamped to the bench but this may lead to blisters on the person doing the clamping.
- As part of the aggregate selection process, a screening test is desirable to select the aggregate having the best compatibility with binder for adhesion stability. The US Strategic Highway Research Program (SHRP) net adsorption test can be considered for this purpose, however, it requires further development and validation work before its adoption.
- There were several other tests presented in this report which are conducted on a loose and compacted mix. Main Roads may consider laboratory tests on loose mix including the static immersion test (AASHTO T182, ASTM D1664), the boiling water test (ASTM D3625/D3625M), the total water immersion test (TWIT), the ultrasonic water bath technique, the rolling bottle method (BS EN 12697-11), the saturated ageing tensile stiffness test (SATS) and the pneumatic adhesion tensile testing instrument (PATTI). For instance, the rolling bottle test can be used to compare the moisture resistance of different types of aggregates (rock types) such as granite, basalt, limestone etc.

Pavement design

Drainage is one of the most critical factors in preventing moisture damage to asphalt pavements. The pavement design must consider inhibiting the water ingress into the pavement and ensure that any water that enters does not build up to the level that it generates cyclic pore pressure and thus increases the risk of stripping. Permeable subbase under the asphalt to intercept moisture should be connected to the subsurface drainage. In Perth, crushed limestone subbase, commonly used as the subbase in full-depth asphalt pavements should be investigated in terms of its suitability and drainage connections in draining moisture from the bottom of the asphalt intermediate course. As no field data was collected as a part of this project, there was no opportunity to investigate the drainage connections of the crushed limestone subbase.

As stated by Kandhal and Rickards (2001):

> It is a fundamental tenet of practicing pavement engineers that three things are vital for pavement performance: drainage, drainage, and drainage. Stripping of asphalt courses will not occur in absence of moisture and moisture vapour.

The premature failure case studies reviewed, identified saturation of asphalt layers by various mechanisms. In each case, it was concluded that saturation is the cause of the problem; stripping is the outcome.

In this regard, it is very important in designing to both inhibit the entry of water and to ensure that any water that enters the pavement does not build up to the level that it generates cyclic pore pressure and thus increases the risk of stripping. Kandhal and Rickards (2001) recommended the use of a permeable subbase under the asphalt to intercept moisture and/or moisture vapour. This subbase should be connected to the subsurface drainage.

Compaction requirements

Compaction is key to inhibit moisture damage to asphalt pavements which is controlled by in situ air void content. However, poor compaction may result in air void content higher than the target and, in turn, ravelling due to insufficient cohesion.

On the other hand, lower air void content (generally < 3%) will provide an impermeable pavement; however, other factors should be considered related to durability and performance of the pavement. Low voids lead to rutting and shoving of the asphalt mixture due to inadequate room for expansion of the binder (bitumen) at elevated temperatures.

The Main Roads target in situ air void content appears to be in line with other SRTAs in Australia and overseas; however, an efficient compaction procedure is required to avoid any deviation from the target compaction.

DoT Vic avoids placement of wearing courses in the colder months from May to September because of difficulties in achieving compaction and a high standard of surface finish. It is particularly important for PMB modified asphalt mixes that tend to become very stiff and difficult to place and compact when rapid cooling takes place. Main Roads (2022a & 2022c) also specifies that PMB asphalt intermediate and wearing courses should not be placed when the pavement surface temperature is less than 15 °C and 20 °C respectively.

Use of seals to inhibit moisture ingress

Main Roads current practice of applying a sprayed seal on the uppermost layer of the size 14 mm asphalt intermediate course and a tack coat of diluted emulsion on the prepared surface is in line with other SRTAs practice. However, they have additional requirements in terms of waterproofing seals such as a waterproofing base layer and a double application rate of tack coat on joints. With good drainage in Perth and low moisture as measured in the Gateway field trials, no additional waterproofing seal requirements are recommended at this stage unless it is supported by the presence of excessive water in asphalt pavements and evidence of stripping in WA.

It would be beneficial to consider:

- a double application rate of the binder on joints and chases for better waterproofing
- tack coat at vertical edges between old and new asphalt pavements, on the top of an existing asphalt layer and on the top of new asphalt not placed on the same day to ensure an adequate bond between pavement layers.

A prime or sprayed seal or a low cutter seal between the asphalt intermediate course and granular base can waterproof the granular base and improve adhesion of the overlying asphalt layer to the granular subbase.

However, Main Roads needs to consider the risk of trapping water at the interface prior to adopting this waterproofing practice.

Aggregate packing analysis

The Bailey method is well-established in the USA as a logical approach to aggregate packing analysis as a part of the asphalt mix design procedure. However, the method caught little attention during discussions with Main Roads, ARRB and industry asphalt experts. As permeability of the asphalt mix plays a critical role in water flow through interconnected voids of the mix and causes stripping, considering the Bailey method in the mix design for investigating packing of the aggregate can help in achieving the desired results in this regard.

Stripping cases in WA

Main Roads observed the effects of moisture in thick lift asphalt (TLA) and full depth asphalt (FDA) on a number of sites. These observations were mostly made on asphalt constructed between 2007 to 2014, when TLA and FDA became popular in WA. Observations included voided asphalt, poor mix design, no waterproofing of the larger fraction mixes, free water visible on the interfaces of cores and stripping.

The Main Roads investigation of the sites with stripping indicated that the sites with this issue were constructed using 2nd generation asphalt mixes and most of the investigations were carried out in 2012. The Gateway field trials were constructed in 2015 with 2nd and 3rd generation mixes. The results of data analysis show that there is clear improvement from the 2nd to 3rd generation mix in terms of TSR, permeability and deformation resistance (wheel tracking test). As the trial sections have not shown any stripping to date, it can be concluded that asphalt pavement performance in terms of resistance to stripping is improved with the use of the 3rd generation mix.

Gateway trial sections

The data analysis as a part of this research was entirely based on the data collected during the previous years from the Gateway field trials on Tonkin Highway as a Main Roads internal project. The project was initiated with the intent to enhance the preparedness for preventing the stripping potential of asphalt.

The key findings of the trials were:

- Based on discussion with Main Roads and the outcome of previous investigations, stripping was not a major concern at the trial sections at the time of the last investigation (2018).
- Currently no additional testing apart from petrographic testing is carried out as a part of the aggregate selection and mix design processes related to the granite aggregates to assess their stripping potential due to the hydrophilic nature of the rock. Granite aggregate used in the mixes was not deemed as moisture sensitive. Not all granites are prone to stripping. It is, therefore, recommended that the granite composition used in asphalt mixes on different projects in WA is examined to ascertain its suitability for the mix.
- In the future, if Main Roads places more and more FDA pavements and wants to prevent stripping issues, the best times to do it are at the mix design and construction stages. Although the granite used in the trials is not deemed as moisture sensitive and prone to stripping based on 2 petrographic test results, more investigation is required to develop guidance on limits for free silica and k-feldspar content in granites through petrographic and XRF testing.

10 Recommendations

The following actions are recommended based on this research:

Appropriate asphalt mix design is critical to produce mixes resistant to moisture ingress into the pavement and the action of water on bitumen-aggregate adhesion. The following measures are recommended in this regard:

- Consider implementing following laboratory tests identified to assess asphalt mix sensitivity to moisture:
  - HWTT for assessing asphalt stripping potential
  - AGPT/T232 (TSR) for assessing the moisture sensitivity of asphalt
  - RMS T230 (plate test or stripping test)
  - SHRP net adsorption test for aggregate selection
  - laboratory tests on loose mix i.e. static immersion test, boiling water test, TWIT, ultrasonic water bath technique, rolling bottle method, SATS and PATTI.
- Assess aggregate susceptibility to moisture based on its type and chemical composition for Main Roads asphalt mixes through:
  - laboratory testing on WA asphalt mixes' constituent granitic aggregates and non-granitic aggregates (e.g. basalt, limestone etc.) and comparing their results
  - use of XRF to determine silica content, mineral assemblage and molecular structure
  - review of petrographic and secondary mineral reports provided to Main Roads to meet the criteria for the asphalt mix design approval process
  - establish threshold limits for the presence of hydrophilic minerals in the aggregate rock (e.g. free silica and quartz, k-feldspar).
- For improvement in the asphalt mix design, following is recommended:
  - investigate and compare the allowable wide range of mix volumetrics used by other jurisdictions in Australia with Main Roads volumetrics as these are critical to stripping resistance
  - it is recommended that the lime content of approved mixes used in NSW and Queensland be investigated and compared to Main Roads approved mixes.

Investigate the suitability and drainage connections of crushed limestone subbase, commonly used as the subbase in full-depth asphalt pavements in Perth in draining moisture from the bottom of the asphalt intermediate course.

It is recommended that Main Roads investigate the appropriateness of the clauses, or the provision of surface and subsurface drainage, in the technical documentation (e.g. scope of works and technical criteria (SWTC)). Moreover, use of overlays and seal coats should be prohibited when there is moisture in the pavement.

Detailed investigation is recommended to examine the selected sections of the TLA or FDA pavements in WA which have undergone stripping to pinpoint the relationship of stripping to different factors and recommend appropriate solutions.

11 Future Research

The presence of low moisture in the Gateway field trial sections indicates that the Main Roads current mix design parameters and waterproofing requirements are appropriate. It should be noted that data analysed as a part of this project was collected by Main Roads as an internal project for monitoring the rials (2015 to 2018). The following data was missing:

- surface distress mapping including signs of aggregate dislodgement, loss of bitumen adhesion with aggregate, signs of stripping in asphalt, pavement layer delamination and surface cracking
- details of the asphalt mix plants and asphalt mix manufacturing procedures such as drying of aggregate and mixing temperature and efficiency are factors in achieving a good bond during the manufacturing process. The adequacy of binder coating and adhesions can be adversely affected by the presence of clay, dust or other deleterious coating on the aggregate (Austroads 2017)
- information related to stripping issues on asphalt pavements on the WA road network.

As a preparedness strategy, it would be beneficial for Main Roads to adopt appropriate measures during the mix design and construction stages. Therefore, it is advisable to carry out a thorough investigation including:

- Adopting a narrowed-down approach to investigate stripping on the road network to develop technical recommendations to resolve the issue. Investigation is required to establish:
  - the extent of the stripping issue in WA
  - if stripping is a significant issue, further investigation to establish its causes
  - investigation into different laboratory tests to pinpoint the most appropriate test(s) for moisture susceptibility of the asphalt mix
  - technical recommendations to resolve the issue if it exists.
- Evaluating the suitability of granite aggregate for the mix, although the compositions of the samples reported in 2 petrographic test results were not found as sensitive to moisture damage in asphalt. It should be noted that not all granites are prone to stripping and the aggregate composition is not a single factor in stripping. As granite is a widely used aggregate in asphalt mixes in Perth, more investigation to evaluate the suitability of the granite aggregate for the mix would be beneficial and, therefore, encouraged. For this purpose, laboratory tests on loose and compacted mix can be adopted including:
  - laboratory testing on WA asphalt mixes' constituent granitic aggregates and non-granitic aggregates (e.g. basalt, limestone etc.) and comparing their susceptibility to moisture
  - use of XRF to determine silica content, mineral assemblage and molecular structure
  - collating petrographic and secondary mineral reports provided to Main Roads to meet criteria of the asphalt mix design approval process. These reports can be used to investigate the suitability of the aggregate for the mix in terms of resistance to stripping.

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