University of Southern Queensland

Faculty of Engineering and Surveying

Problems of Stone Mastic Asphalt Use In North Queensland

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Glen Keith Allen

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Abstract

Stone Mastic Asphalt (SMA) is widely used throughout the world as one of the preferred asphalt surfacings. The history of use of this material dates back some 30 years ago, and like every product stone mastic asphalt needs to be modified and adapted to conform modern materials and manufacture, as well as suit the various local conditions. This dissertation develops and analyses the variances in all properties of SMA within a tropical climate. The aim is to provide background information into the history of the product and the current best practice, before moving into the specifications and requirements of the North Queensland region. The methodology will take the format of the testing of trial sections, analysing data and results, compiling details and collating information within the prioritized sections. The aim is to make definite correlations between specific criteria and then hypothesis on the possibilities. The outcome is the actual design criteria that leads to particular properties and arrive at the failure mechanisms of stone mastic asphalt. The applicability of this paper will be a document which aligns with the Government Standards for Asphalt design to either reinforce or alter current practices.

The impacts of filler and binder components of the mastic are assessed on the performance of SMA. The paper develops a design method to ensure that the important features of the coarse aggregate stone skeleton are attained, providing a rut resistant long life asphalt. Analysis is provides in the combining of criteria on rut resistance and fatigue performance to arrive at a mix design shows good strength, texture and stability for use in surfacing works with heavy traffic condition. Elastic and fatigue properties are assessed by analyzing the affects of fillers, binders and temperature and the relationship between stiffness and fatigue.

SMA is an unforgiving mix and requires changes and modifications to the mix design to enhance the characteristics of its performance. This is evident through the varying specifications across the State, and the North Queensland developments form a major topic within the report. Whilst SMA a premium asphalt product it is not a panacea for all pavement situations. Its use and specification requires exercising sound engineering judgment. This research will provide a rational basis for such judgment.

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Glen Keith Allen

Student Number: D83111540

Signature

Date

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The development and compilation of a dissertation is no simple task. Factors such as economics and local knowledge can be imperative to the finalization of a paper.

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Abbreviations

AAPA	Australian Asphalt Pavement Association
AASHTO	American Association of State Highway and Transport Officials
AC	Asphalt Concrete
ALF	Accelerated Loading Facility
APA	Asphalt Pavement Analyzer
APRG	Australian Pavement Research Group (Australia)
ARRB TR	Australian Road Research Board Transport Research
ASTM	American Society of Testing Materials
BC	Bituminous Concrete
BR	Polybutadiene
CBR	Californian Bearing Ratio
CC	Characteristic Curvature
CD	Characteristic Deflection
CIRCLY4	Computer Software
C320	Class 320 Bitumen
CR	Polychloroprene
CSV	Comma Separated Variable (Computer File Format)
DG	Dense Graded (Asphalt)
DMR (Qld)	Queensland Department of Main Roads (Australia)
DPM	Dilation Point Methods (of Design of SMA)
EAPA	European Asphalt Pavement Association
EMA	Ethyl Methacrylate
ESA	Equivalent Standard Axle
EVA	Ethylene Vinyl Acetate
FWD	Falling Weight Deflectometer
HDAP	Heavy Duty Asphalt Pavement
HMA	Hot Mix Asphalt
IIR	Isobutene-Isoprene Co-polymer
MATTA	Materials Testing Apparatus
MMAT	Mean Monthly Air Temperature
MVR	Mix Volume Ratio
NAASRA	National Association of Australian State Road Authorities (Aust)

NARC	National Asphalt Research Committee (Australia)
NAT	Nottingham Asphalt Tester
NCAT	National Centre for Asphalt Technology (USA)
NR	Natural Rubber
NTPT	National Transport Planning Taskforce (Aust)
OG or OGFC	Open Graded (Asphalt) or Open Graded Friction Course
PAFV	Polished Aggregate Friction Value
PBD	Polybutadiene
PE	Polyethylene
PG	Performance Grade (of Bitumen)
PMB	Polymer Modified Binder
PP	Polypropylene
PS	Polystyrene
PVC	Polyvinyl Chloride
QDOT	Queensland Department of Transport (Australia)
RLAT	Repeated Load Axial Test
RM	Resilient Modulus
RTANSW	Roads and Traffic Authority, New South Wales (Aust)
SAA	Standards Association, Australia
SAMI	Stress Absorbing Membrane Interlayer
SBR	Styrene-Butadiene Co-Polymer
SBS	Styrene-Butadiene-Styrene Block Co-polymers
SHRP	Strategic Highway Research Program (USA)
SMA	Stone Mastic Asphalt
SST	Superpave Simple Shear Tester
SUPERPAVE	*Su*perior *Per*forming Asphalt *Pave*ment System (USA)
U.F.D.	Ultra Fine Dust (Filler)
VCA	Voids in Coarse Aggregate
VFB	Voids filled with binder (%)
VMA	Voids in Mineral Aggregate
W.I.	Workability Index
WHOLAC	Whole-of-Life Agency Cost
WHOLC	Whole-of-Life Cost

w-MAAT	“weighted” Mean Annual Air Temperature
WMAPT	“Weighted” Mean Annual Pavement Temperature

CHAPTER 1 – INTRODUCTION

1.1 Outline of the Study Analysis

The need for research into the problems with Stone Mastic Asphalt North Queensland was identified from the continued failure on high profile State and National Highways. Most road authorities rely on standard specifications for the manufacture and laying of asphalt, mainly due to the contractual aspects. When failures are evident before the usable economic and physical life of the asphalt is due, the standards come under pressure. The issue becomes complicated when the problems vary depending on the area of location throughout the nation. This report will focus firstly within the State of Queensland and furthermore within the local area of Cairns in the tropical north. States tend to vary with regards to specifications and this creates a wide range of variables. Queensland has a state wide specification, although Cairns in the tropical climate require modifications which have become necessary due to failures. Whilst still complying with contract conditions these changes should be re-enforced with trials, testing and analysis. This report will endeavour to perform this requirement. The purpose and scope will be detailed in 1.4 – Research Objectives.

1.2 Introduction

State Government funding is the single main source of projects which require and specify the use of Stone Mastic Asphalt. The economic decisions on project allocations for location and types rest with the State Road Authority, called the Main Roads Department (MRD). Since the introduction of the Main Roads Stone Mastic Asphalt (SMA) Specifications in Queensland in 1996 the Cairns Peninsula District has placed a significant investment in SMA surfacing. The Cairns District is the third largest user of SMA in the state after Gympie and Metropolitan Districts. SMA has become the first choice for resurfacing operations and new pavement construction projects in the district. The district has recognised the benefits of SMA on their road network by measurable improvements in terms of performance when compared against existing dense graded

(DGA) and open graded (OGA) asphalt surfacing. It is predicted that SMA will outperform both DGA and OGA mixes in the tropical climate in the Peninsula District. The problem is that the theory behind this statement is fairly shallow.

In 1997/98 Main Roads approval was given to one of the districts suppliers for a series of submitted mix designs on a range of sizes – 10 and 14 mm, followed by a further to the only other supplier. These SMA mixes were approved with a range of different bitumen (binders) including Class 320 bitumen, Multigrade and A5S Polymer. These binder types are basically straight bitumen, graded more refined bitumen and rubber impregnated bitumen. These will be further discussed if necessary later in the paper. Bitumen is one of the main constituents of making SMA.

Peninsula district began field trials of SMA back in 1997 under the supervision of the head of the Assets section and the districts engineers and inspectors. These early SMA trials were located throughout the District and will be detailed within Table 1 in section 4 of this paper. The district trials were conducted from 1997 to 2002; with a variety of different bitumen (binder) types were included in the SMA trial.

Constituents within the mix design have also endured some modifications. Various decisions have been made based on researching of overseas experiences with SMA Mix designs and other asphalt mixes. Certain requirements have been written into the Peninsula Districts specifications for contracts containing SMA mixes since 1998.

It is the modifications such as these mentioned above that have become an experiment with further educated analysis. There are many other modifications currently being trialed.

1.3 The Problems

Despite these local modifications, the asphalt laid in certain areas is sustaining failure and is visibly under stress. Due to the increase in traffic volume and loadings, rutting has become a major form of distress on many asphalt pavements. This is a wide-spread issue and one of the main reasons why SMA was introduced, with great stone-on-stone contact. However, the problem with increasing rut resistance is that fatigue resistance may be compromised.

The safety of SMA surfaces on National Highways has come into question following a series of accidents and subsequent investigations by road authorities into concerns expressed by the police and others regarding dry road skid resistance following a number of fatal accidents on new surfacing materials. Appendix B shows publicity surrounding the concerns. The information derived from the studies, some reported in this paper is based on scientific data and provides an objective assessment of the wet road skid resistance performance of this material. There is a 30% chance that the new stone mastic asphalt surfaces will not meet the investigatory level for wet road skidding resistance in the 12 months after laying.

Surrounding the publicity section, reported rainfall intensities in the Gympie area of the time of recent fatalities were low. Given normal surface geometry, the depth of water film in such light rainfall should not have been sufficient to cause aquaplaning.

However, partial aquaplaning could occur if high speeds and worn tyres coincided with such water film thicknesses, Furthermore, surface irregularities (e.g. wheel ruts) can increase water film depth and further contribute to partial aquaplaning. These irregularities existed and possibly contributed. It can be concluded that the increased friction demand as a vehicle proceeds of the horizontal curve with the reduced super-elevation increased the risk of crashes at lower rainfall intensities.

All newly laid bituminous surfacings have slightly lower skid resistance levels compared with those obtained a few months after re-surfacing due to the binder film coating the stone. Initial values should be above the investigatory levels required for the roads in question or steps should be taken to warn motorists of the lack of skid resistance of the new material.

Throughout the Department of Main Roads Queensland, the perception seem to vary be seniority and by geographical area on the value and use of SMA. The "failure mechanisms” are deemed to be a lack of durability due excess porosity of the earlier designs. Subsequent trialing to improve the mix has resulted in stiffer and harder to lay mixes.

1.4 Research Objectives

The research comprised of identifying the criteria by which asphalt is measured, and the true design elements which are employed. The second part of the Research Methodology was to review relevant literature to ascertain the limits of these criteria and the possible outcomes in the life span of SMA through changes to the design. This will be defined in 2.2. The third element was to research all areas in the Northern Districts where SMA has been layed, and collect locations, dates, crash data and wet/dry skid resistance.

1.5 Conclusions: Chapter 1

This dissertation aims to report on linkage between SMA properties, design criteria and observed failure mechanisms. The research is expected to result in a series of possibilities and these must be related to the differences in North Queensland. A review of literature is a fairly large task when dealing with Asphalt in general, as there have been many studies and publications. The review will identify where to concentrate the testing of the trial sections and how to analyse the data. The outcomes of this study will be used for developing future specifications in general and more concisely in the Northern region to modify the existing standards.

The research also endeavors to harness the works already completed by various other throughout the state, but in particular in the north. There have been many trials with testing being performed on all, but it is the hypothesis of this work and the related correlations that back development and formal documentation.

When it comes to final outcomes and recommendations, the dissertation will aim to firstly provide a complete 'snapshot' of the current status of SMA and its usage, before leading into a direction for the future. It will be noted though that the continued development should lean toward proving that the correlations derived in the paper do perform in service. This will be achieved by further testing.

CHAPTER 2 – CONCLUSIONS FROM LITERATURE REVIEW

2.1 Introduction

This chapter will review literature to establish the tools for identifying the problems of Stone Mastic Asphalt use in North Queensland. After reviewing large amounts of material this chapter will consider the relevant main literature, and also provide extracts of the absolute data which will form the basis for the study.

2.2 SMA Development

SMA usage in Australia began in 1990 when Vic Roads conducted a trial in Victoria on the Princess Hwy. A 14mm mixture mixed using a batch plant was used based on Rettenmaier design and imported Arbocell fibre; however, the trial was not fully successful. A further trial of 14mm SMA was conducted in 1993 on the Hume Hwy and Maroondah Hwy that was deemed to be successful. Vic Roads have placed over 15, 000 tonnes up until 1996 (no tonnage data is available after this date). Later mixtures were also conducted using 10mm gradings. The early Vic Roads mixtures used C320 grade bitumen with PMB binders being introduced in 1999 for high fatigue applications.

Brisbane City Council (BCC) in Queensland trialed SMA mixes in 1992. The Rettenmaier grading was also used as their design principal for their 10mm SMA design using multigrade bitumen and manufactured using a drum mix plant. No fibre was used in these mixes. Some of the BCC mixes were considered to work satisfactorily while others did not due to the drain down of binder and the high percentage of elongated particles in the mix, with some flushing of the mix also occurring.

In 1994-1995 fibres were introduced into the BCC mixes, several types of fibres were trialed including mineral, cellulous and glass fibres. The cellulose fibres were considered by BCC to give the best results, BCC has continued to use SMA which has constituted up to 20% their total annual asphalt production.

The Department of Main Roads (QDMR) in Queensland has placed SMA in Queensland since September 1996. The first trial was located in the Metropolitan District on a section of Mt Gravatt – Catalina Road; adjoining the Capalaba Bypass using a 14mm mix manufactured using a batch plant. The trial was considered successful. QDMR has produced over 1.5 million tonnes of SMA to the end of 2002. The original QDMR specification was written in 1993 and based on a combination of the Rettenmaier design, Brisbane City Council's SMA design and the QDMR 14mm Open Graded mix design. The QDMR SMA Mix design incorporates the use of heavily modified SBS binders, typically A15E and A10E grades as specified in AP-T04 [A] with additional limited use of multigrade bitumen in the last several years. Locally these standards still apply with some added interim specifications being added.

2.2.1 Review on the use of Stone Mastic Asphalt by Road Authorities

One of the most extensive reports ever commissioned into asphalt in Queensland, and certainly ever into SMA was the Troutbeck Kennedy Report of Sept. 2005. The report followed a series of accidents on the Bruce highway near Gympie, and questioned the use of SMA on Queensland roads.

SMA is characterized by a "stone on stone" structure. SMA uses a high proportion of larger stones or aggregate that contacts each other. This skeleton of larger stones resists heavy loads by transmitting them to the pavement below. If the underlying pavement is sufficiently strong then the SMA will resist the heavier loads effectively. (A surfacing cannot compensate for a weak pavement).

Troutbeck states "The bituminous mastic is intended to hold the aggregate in place and to inhibit the ingress of moisture into the pavement and to provide durability. The mastic consists of bitumen and fine aggregate particles; it may also include a polymer modified bitumen and filler material to increase the mastic's strength. Fibers may also be added to stabilize the bitumen and to prevent the binder segregating from the aggregate during transport and placement.”

It is important that the aggregate material consist of only the larger stones (in the structure) and fines to provide effective mastic. The intermediate size aggregates are not included, as these keep the larger aggregate apart and reduce the strength of the SMA.

If SMA is designed as a mix with too little bitumen, then the percentage of air voids increases and water will infiltrate the surface and possibly the underlying areas. This is originally how Open Graded mixes were designed so-as to effectively remove excess water from the asphalt pavement Water in the asphalt can also break the bond between the stones and the bitumen and allow the bitumen to unravel. The specifications, the mixing, the transport, the placement and compaction of SMA are critical to achieving the desired result.

SMA and OGA have been developed to provide an effective surface texture. This is a prime safety requirement and helps to maintain skid resistance at the higher speeds. The texture is also useful in decreasing the water depth on the surface. These qualities make for safer roads. Skid resistance is a function of the micro texture (or the roughness of the individual pieces of exposed aggregate) and the macro texture (developed from the arrangement of the aggregate on the surface).

The Report recommends in Recommendation 2 “If is recommended that the Department of Main Roads continue to develop asphalt surfacing with a longer life and better durability while maintaining an appropriate surface texture. These surfacing are required to support the road transport task and community requirements in the future.”

The performance of SMA is dependant on the grading and material proportions (defined by the accepted specification), the mixing process and the plant, and the laying compaction process. All aspects must be closely monitored if SMA surfacing is to have a long life. Consequently, SMA is a surfacing that requires a high level of detail and constant research, fine-tuning and modification. The process of introducing SMA, or any other alternative surfacing, should be slow, careful and deliberate and involve state-wide and possibly nation-wide discussion.

2.2.2 Development and Performance of New Stone Mastic Asphalt Specification

Over the years of service of asphalt pavements, and in particular stone mastic asphalt, the performance has ranged from exceptionally high performance and requiring no maintenance, to poor performance requiring significant maintenance after two years of reconstruction. This assessment has been developed by the Cairns QDMR officers of the Cairns District and their counter-parts in other areas of Queensland.

Patane, Bryant & Vos (2005) state the following causes:

- Poor pavement type selection has design;
- Inadequate supervision by the client of the asphalt manufacture of asphalt;
- Reduced quality control con constituent materials in the manufacture of asphalt;
- Variable and high demand for asphalt pavements creating material shortages;
- Reduced asphalt mix design experience and manufacture supervision;
- Unreliable and in accurate testing data;
- Variability in the binder properties;
- Changes over time in the properties of constituent materials, in particular, fillers.

In the last three years, Queensland has been ‘adjusting’ its SMA specification to address the workability and durability concerns. The first attempt around early 2003 was mostly a tightening of requirements to increase density (reduce permeability) but resulted in mixes with very poor workability. Patane, Bryant & Vos (2005) state their observations on our Queensland mixes were constructive and they proposed the following:

- Density better than 94%
- Less use of fly ash and hydrated lime;
- Changed grading to increase VMA;

- Test sections before construction
- Mix approval based on production;
- Use of less modified PMB's;
- Use of VMA as production QA/QC.

2.2.3 Development of Stone Mastic Asphalt for Queensland

Growing demands of the road transport task in Queensland have lead to investigating the benefits of Stone Mastic Asphalt in the early 1990s. Lack of funding seems to always retard progress, but the first project was completed in October 1996. SMA has quickly become the surfacing of choice in Queensland, and whilst its characteristics as a road surfacing have lived up to expectations, permeability and constructability issues have caused some concern. Research is continuing within the technology branches of State Government to overcome these concerns and perfect Stone Mastic Asphalt as the ideal surfacing for Queensland Roads.

Hogan, Patane & Lowe (1999) explain that "In northern Europe it was seen as an alternative to dense graded asphalt to resist the damage caused by studded tyres which were used in the harsh European winters to cope with snow and ice. As axle loads increase in Europe, Stone Mastic Asphalt was seen as a solution to rutting problems which occurred if the short summers were hot. With its high binder content it was also able to cope with the cold winters by resisting cracking caused by fatigue."

The report follows to describe that an AAPA study tour of Europe suggested that SMA could perhaps provide a solution to some of the problems which has beset the Queensland road network, namely:

- Rutting
- Fatigue cracking
- Bleeding/crushed surfaces

- Inadequate texture depth for high speed operation
- High tyre noise

Brisbane City Council had been using a Stone Mastic Asphalt mix, of nominal size 11mm approximately, for some time. The BCC mix produced many successful surfaces using a multigrade bitumen binder (without fibers); however on several occasions it produced quite severe flushing and bleeding as a result of segregation and/or binder drainage.

A few trial mixes of SMA were laid in Melbourne in the early 1990s with high binder contents similar to the European mixes. These trials exhibited rutting and bleeding in subsequent hot weather and heavy traffic conditions. Also, handwork and paving of tapers can be particularly difficult with SMA due to its low workability caused by coarse grading, high filler and stiff binder.

To avoid potential flushing and bleeding problems Queensland SMA has been designed with slightly higher air voids. This can produce a SMA surfacing which is permeable under heavy traffic and rain. To avoid damage to underlying pavements which are cracked, the Department now specifies a seal or SAM or SAMI (rubber modified sprayed seal) should first be applied prior to placement of the SMA surfacing.

Hogan, Patane & Lowe (1999) suggest further research into the following main areas:

- Permeability/air voids/binder content
- Dilation point/mix volume
- Optimal binder tyres
- Wheel tracking
- Fatigue testing
- Workability

With the ongoing experience gained from projects and the technical knowledge gained from the laboratory programs, it should be possible to further improve all aspects of

Stone Mastic Asphalt surfacing to make it the ideal road surfacing for Queensland roads.

2.2.4 The behavior of Asphalt in Adverse Weather Conditions

A program of laboratory tests has been undertaken to support the development of specification clauses and associated advice on laying asphalt in adverse hot weather conditions. The laboratory programme assessed the influence of both temperature and traffic speed on the deformation resistance of hot rolled asphalt and stone mastic asphalt.

Nicholls & Carswell (2001) states "There are various physical actions that can be taken when laying hot asphalt in adverse hot weather conditions to minimise the potential problems. However, experience shows that more rutting develops when traffic speeds are reduced and this should be a factor to consider."

There is no single solution to the problems of trafficking newly laid asphalt during hot weather. Nevertheless, there are ways of reducing the risks to manageable levels by use of a procedure such as the proposed risk assessment model. By minimising the risks at all stages of the work, from mixture production to traffic control, the amount of damage induced should be within acceptable limits. The relationship implies that the permanent deformation is proportional to the traffic flow, the wheel-tracking rate at 45°C and the logarithm of the age plus one.

2.2.5 Stone Mastic Asphalt – UK Experience

Richardson (1999) states "In very general terms shifting from Porous Asphalt to thin wearing course and on to SMA, the air voids content and surface texture are reduced, but each material still has a relatively quiet surface compared with that of chipped HRA."

There has been increasing concern in the UK over the lack of resistance to rutting of asphalt wearing courses. This has led to a decrease in binder contents that has in turn resulted in a fear of possible loss of durability and of resistance to cracking. A dense material has now been made available that imparts to the road surface both high resistance to deformation and high durability through the design of a coarse graded aggregate structure having the capacity to accommodate a rich bituminous mortar by the incorporation of suitable binder carriers. The adoption of thinner layers is similar to the development in the Australian, Queensland and Cairns approach.

2.3 Skid Resistance

2.3.1 A Report from the International Society for Asphalt Pavements

(Danish Road Directorate 2002) explains "Particle packing theory has been applied to the grading curves of some typical coarse aggregates used for the manufacture of SMA. To ensure that dilation of the stone skeleton does not occur, it has been shown that the maximum size of particles in the mastic fraction varies with the maximum size and grading of the coarse aggregate used. It is suggested that the separation between stone skeleton and mastic for a SMA14 is the 2.36mm sieve and for SMA10, it is the 1.18mm sieve."

In Queensland, where thin surfacing layers (<40mm for SMA10) are used, often over old, weak pavements with high deflection, fatigue failure of SMA, rather than rutting, may be the limiting design factor. Skid resistance measured by the Sideway-force Coefficient Routine Investigation Machine (SCRIM) and the Griptester has also indicated that a level can be achieved that is commensurate with that which would be expected of the aggregate used in those same site conditions for a traditional surfacing mixture.

2.3.2 An Investigation of the Skid Resistance of Stone Mastic

The SCRIM surveys undertaken in the last 6 years on the principal road network and class roads together with High Speed Road Monitoring Surveys provide important data for the performance assessment of new surfacing materials. SMA has been universally used in across the World for some years and has been well received particularly in urban areas due to the spread of operations and the reduced noise and spray.

Bastow, Webb, Roy & Mitchell (2005) say the results show that:

- The initial skid resistance of all the SMA's in the study was similar to conventional surfacings. Here was a 30% chance of SMA having skid resistance value lower than the investigatory level for the site category in the 12 months after surfacing.
- Skid resistance improved with time and in one year the MSSC values had increased approximately 11 % and remained stable for the next two years before falling to 6% in the fifth year. The initial increase in skid resistance is generated by the surface binder being abraded exposing the coarse and fine surface aggregates which contribute to the ultimate skid resistance of the material.
- If aggregates of the specified PSV are used the skid resistance of SMA after the binder film wears away should give acceptable values for the general road category in Dorset. A SCRIM coefficient of better than 0.45 would be expected from aggregates with a polished stone value of 60.

This study based on factual data from one County Council of Dorset in the UK over a five year period indicated that SMA surfaces had a 70% chance of exceeding the investigatory level of skid resistance in the first year after laying. All sites showed some improvement in skid resistance in the succeeding two years and thereafter stabilized at a lower level. In year five 10% of the sites had some values below the recommended investigatory levels which emphasise the need to select materials with due regard to the site category.

Work is required to investigate the high speed frictional resistance of stone mastic asphalt in the initial period before the binder rich mastic mortar has been abraded. Measurements are required to determine the thickness and consistency of the binder film covering the surface aggregate in relation to other bituminous materials and whether the absorptive filler used in the mix have any impact on the skid resistance of the surface material. The research requirements across the nations seem to be similar to the Cairns region and QDMR.

2.3.3 Early and Mid Life SMA Skid Resistance

Safety applies to all stages of roadway construction i.e. from initial design, selection of materials to use of the surface by the water. Woodward, Woodside & Jellie (2005) explain that "In the UK, a range of criteria including noise, negative texture, spray generation, layer thickness, availability and cost of higher PSV aggregate, has shown the need the need for more sustainable technologies have caused a shift towards thinner, and quieter types of asphalt surfacing materials. These typically used modified bitumen or have thicker bitumen coatings to hold the aggregate particles together."

The authors recognised that the early life safety of these materials needed consideration given that a bitumen rich surface tends to have poorer wet skid resistance. This paper considers the development of skid resistance for a SMA surface using high PSV greywacke aggregate and polymer-modified bitumen. The SMA surface was periodically measured using a GripTester to determine how skid resistance has developed from early life through to mid life. The findings showed how this is different from a conventional chip seal or positive textured asphalt surface.

The review of this paper considered the development of early life skid resistance measured on-site and in the laboratory. It highlights that there are complicated inter-relationships between many factors such as type of aggregates, bitumen, and composition, and surface texture, time of the year, road geometry and trafficking conditions. These are two basic types of asphalt surfacing. Those that are positive textured where the aggregate embeds into the type and the aggregate micro-texture is either exposed or becomes quickly exposed. Vehicle dynamics are applied to what is essentially a series of point loads leading to conventional polishing of the surface starting to take place relatively quickly.

The second type has a smoother, negative or porous texture, where the aggregate does not embed into the tyre to the same degree for example SMA (porousaAsphalt).

Loading is spread over a greater area of thickly coated aggregate/matrix and it takes longer to wear away the bitumen and expose the aggregate. "As there is less aggregate embedment so the contribution of hysterisis effects on friction may be reduced."

The research has found that the combination of aggregate and bitumen has a significant effect on skid resistance during their early life. Aggregate type is important for unmodified bitumen, particularly those with variable composition as the weaker/softer/unsound particles will loosen their bitumen coatings faster. Reliance on the use of higher PSV (polishing) does not ensure high skid resistance during early life. Rather, a lower PSV aggregate which strips quickly may perform similarly, and in some cases better, than a much higher PSV aggregate.

These conclusions seem to contradict all other properties required of a surfacing mix, i.e. the development of good aggregate/bitumen bond to resist moisture induced loss of stiffness, cohesion and surface raveling. Therefore, in terms of ensuring early life skid resistance, there is a balance between safety and durability which needs to be considered. The expectation of highway materials to perform is high.

2.3.4 The German Origin of SMA

Skid resistance is an essential element of traffic safety in wet surface conditions. The skid resistance of asphalt wearing courses is generally unsatisfactory right after laying, because the binder on the material aggregate has not been worn off by traffic yet. Druschner (2005) states "In order to improve the skid resistance in this stage, it is mandatory in Germany to grit the wearing course."

Once the binder film has been worn off, the macro-texture and micro-texture are the decisive parameters for skid resistance. The macro-texture is mainly responsible for the skid resistance at higher speeds. As of approximately 80 km/h, the tyre profiles can no longer take up and/or safely carry off the water. Then the macro-texture of the wearing course has to take over this task to prevent aquaplaning. With SMA and other wearing course types, the macro-texture depends on the composition along with the paving and compacting temperature. A composition, which is rich in mortar and results in a dense surface without pronounced texture. Likewise, high paving and compaction temperatures also result in dense surfaces. Therefore, the use of pneumatic rollers should be avoided for wearing courses, especially for SMA. This will be reinforced in later sections of this research paper.

In many countries, including Europe, there is a tendency to produce SMA with a very large particle size. In Germany however, this tendency is reversed. Instead of the SMA 0/11 mainly used so far the paper explains they now frequently use SMA 0/8. SMA 0/8 has a better skid resistance, as it has a larger number of contact points to the tire than SMA 0/11 (0 to 11 mm).

CHAPTER 3 - METHODOLOGY

3.1 Aim of this Thesis

The aim of this thesis is to determine the important mix design, manufacturing and application requirements for Stone Mastic Asphalt (SMA) and how these areas influence the performance of the mix. This will be achieved by:

- Developing a design method to ensure that the important features of the aggregate stone skeleton are attained.
- Investigating the impacts of all components of the mastic on the performance of SMA mixtures as well as production and laying by comparing a generic SMA specification with the northern interim specifications.
- Quantifying the implications on performance of the modifications to the mix design of SMA.
- Developing a failure mechanism with derived limits.

3.2 Format of this Thesis

To achieve the aims of this thesis, an extensive literature review has been undertaken focusing on the areas of failure mechanisms, trial data, asphalt materials and mixture design methods, test methods, SMA specifications and previous SMA research projects. The summary of the literature review is contained in Chapter 2.

Based on the outcomes of this literature review, a program of experimental work was proposed to validate mix design procedures, assess the adherence to the specifications of manufacture and to quantify the effects of the changes necessary to achieve performance of SMA in a tropical climate. This discussion forms the basis of Chapters

4 and 5. International research and development with specifications is discussed in Chapter 6.

The experimental work is discussed in Chapters 7, 8, 9 and 10. Chapter 7 deals with the investigations into methods to ensure that the stone on stone contact of the stone skeleton is maintained. The Chapter includes the interim specification for the local district of North Queensland and concludes by outlining modification to the MRD / MRS document (2004b) Smart Surfacing. This will be followed in more detail within Chapters 9 and 11.

Chapters 8, 9 and 10 investigate the mastic fraction of the SMA and the effects of two components of the mastic; binder and filler; on the properties of the completed mix. Chapter 9 discusses the stiffness or elastic properties of the mix. The implications for fatigue life and hot weather considerations are addressed in Chapter 8. Chapter 10 considers the effect on deformation resistance by using conventional wheel track tests; and investigates a new fundamental test method Vacuum Confined Dynamic Creep for deformation resistance.

By applying the derived material performance properties (elasticity and fatigue life) to typical design examples, the implications of the choice of mastic components of SMA on pavement life and performance has been evaluated in Chapter 11, as well as the modifications to suit North Queensland conditions.

A summary of the most significant findings and recommendations for further research are presented in Chapter 12.

3.3 Specific Points

The development of this paper will depend partially on prior trial data from the research projects and the trial data which is performed during the course of testing. The basic flow though will be at a step-by-step process in comparing history with actual trial data, as follows:

- Literature review on world's best practice and general knowledge of concepts, specifications and procedures.
- Review and summaries background information.
- Start with all SMA sites in Northern Region and prioritize on the basis of:
  - is there available detail
  - crash Data
  - Traffic volumes
  - Extent of site
  - grouped into highways

  Then compare:
  - type of asphalt
  - which company/inspector
  - distinctive variables
- Reprioritize to minimize the source data, and testing
- Test these sites and categories into where areas are showing stress – visual and quick testing.
- Design a list of criteria for comparisons of the sites:
  - voids
  - Surface texture, Macro/ micro textures
  - grading
  - Bitumen content
  - PAFV of aggregate
  - Source material details
- Testing of the priority sites based on deformation to arrive at a table of comparative data between original and actual.
- Extend the correlations and make-reference to the future life of the asphalt – balance of extending bitumen content compared to pavement life.

CHAPTER 4 – ASPHALT MATERIALS, MATERIAL PROPERTIES AND ASPHALT MIX DESIGN

4.1 Introduction

Throughout history, the origins of the various mix designs are generally centred within Europe. SMA began in Germany, and the German practice in the mixing of asphalt is covered in accordance with the "Additional technical contractual conditions and directives for the construction of roadway surfaces from asphalt" and the associated "Code of Practice for qualification tests on asphalt", which is an issues paper produced in 1998 through AAPA. In Australia, we adopt the theories from these papers and qualify changes throughout the various States, where each has its own specifications. There are certain procedures which remain fairly consistent by all codes.

The material flow into the mixing hopper must take place in the sequence given below in order to achieve adequate dispersal of both the filler and the fiber throughout the mixed material and to obtain a fully coated mixture. The mixing times are slightly longer than for conventional asphalts because of the inclusion of the additives, mainly fibers which act to strengthen the mix. The order of mixing, together with approximate timings are:

- Coarse and fine aggregates introduced and mixed over a 15 second period.
- Filler introduced and mixed over a 20 second period beginning at the same time as for the coarse and fine aggregates.
- The fibers are also introduced during this dry mixing time, the exact timing being dependant on the fiber type but early enough during the cycle to ensure full dispersion but not so long as to break down the fiber. After the dry mixing time of 20 seconds, the binder is introduced and mixed with the 'dry' components over a 15 second period.

- There then follows a further 10 seconds mixing cycle followed by an 8 seconds discharge time.

Most SMA is produced in batch mixing plants with pug mill mixers, although continuous (drum) mixing plants can be used. In the latter case, palletized fibers should be used and extreme care is required to ensure an even distribution of the various components throughout the mixing process. Quality production control is an essential requirement to ensure the volumetric proportions are maintained. Due to the gap-graded (i.e. certain sieve sizes missing in the grading of the aggregates) structure of SMA it is necessary to establish an adequate quality control system for the incoming aggregates and to maintain the stock piles properly. The use of automatically controlled feed systems for the additives and fibers are recommended. Modern material handling systems provide sufficient flexibility to add different types of palletized additives to the mix e.g. via top® (cellulous fibers used to stop binder drain down in a mix and hold the product in suspension during transport), pigments, binders, polymers.

There are only two plants for manufacture in North Queensland, one batch and one drum. This report will concentrate on the batch plant, as this is how SMA is meant to be manufactured. A copy of the plant layout at the Boral Depot is attached in the Appendix F.

As stated, SMA can be manufactured in conventional batch and drum mix asphalt plants although some modification may be required in order to effectively handle fibers and the amount of added filler. Generally, reclaimed asphalt pavement is not suitable for inclusion in SMA unless screened and separated into the grading fractions required for the SMA mixture.

The performance of SMA is based on a strong gap graded aggregate composition. Variations in the relative proportion of structural aggregate and the fine aggregate filling the void spaces in that aggregate skeleton will have an almost linear effect on the voids on the mixture. Particular care must be exercised to avoid variations in the proportions of aggregate passing the critical 2.36 to 6.7mm sieve sizes (depending on nominal size of mixture). SMA is sensitive to overfilling of the aggregate structure with mastic. If that occurs, the mastic bears the loading. As unmodified mastic has almost no deformation resistance, premature rutting is likely to follow.

SMA manufacture requires good grading control and accurate proportioning of aggregates. Filler, fibers and bitumen requires careful calibration of all feed equipment, as well as ensuring that equipment is capable of operating at the feed rates required. Mixing procedures must provide uniform, consistent mixing of materials.

Both batch plants and drum mixing plants are used to manufacture SMA. SMA requires an increase in mixing time to ensure the fiber and fines are adequately dispersed in the mix uniformly, and that there is no balling up of the fiber and fines in the mix. This may not be possible with some of the drum mixing plants, as mixing time can not be increased or prolonged. The Australian practice of using drum mixing plants may result in fibers coming into direct contact with the heating flame and being burnt. Fiber distribution within the SMA should be monitored to ensure that even distribution of the fiber is achieved. The production of SMA should be closely monitored to ensure that all aspects of the production process are met and satisfied. Drum plants also have problems with the exact location of fiber, filler and bitumen entries to achieve the correct mixing.

4.2 Asphalt Materials

Asphalt is a mixture of aggregate and bituminous binder, with or without added mineral filler or fibres, produced in a mixing plant. Each of the component materials need to be carefully selected and controlled to ensure that they are of a quality suitable for the asphalt and the expected performance. The purpose of a mix design is to determine the best proportions of the available aggregates, binder, filler and fibres to give a product that is durable, workable, has adequate resistance to deformation and adequate flexibility to withstand cracking and fatigue. In the case of the wearing course, it is also necessary to provide surface texture and skid resistance appropriate to the speed environment in which the pavement is located. Provision of some of these characteristics is often mutually exclusive.

The specifications have maximum and minimum values and envelopes of limits to remain within. The mix design is also specific to the source materials in the local regions of manufacture. There has been considerable research into the appropriate tests and specification limits for the various components of an asphalt mix. This paper focuses on the performance of complete mix and how the combined grading and material types impact on this performance. Component materials were selected to comply with current specifications; typically those published by Queensland Department of Main Roads and Standards Australia.

To minimise the effects of raw material variability, comparative testing was performed using raw materials from the same source stockpile or manufactured batch. Variances can then be highlighted by separating out the material source issues, whilst maintaining whether this could be an issue.

4.2.1 Bitumen Binders

Much of Europe including the United Kingdom use penetration grading systems (EAPA 1998, BS 3690-1:1989) however available bitumen grades do not correspond between countries (Loveday and Bellin 1998). New Zealand also uses a penetration grading system (AUSTROADS 2000a). In the United State of America, the penetration grading system was replaced by two viscosity grading systems (Roberts et al 1996) that are now being superseded by the SUPERPAVE™ system. This introduced a performance based binder specification intended to perform equally well for both modified and unmodified bitumen (Browen et al 1996). Doubts have been expressed as to whether the specification is applicable to both plain and polymer modified binders (Oliver 1996, 1997). The Georgia Department of Transport have added a phase angle requirement to their SUPERPAVE™ PG76-22 binder specification to ensure that polymer modification is used to meet the binder grade requirements (Watson et al 1998). SUPERPAVE™ also introduced new testing equipment for physical tests that are related to pavement performance parameters being partly influenced by the binder (Brown et al 1996).

In Australia, bitumen is classified on the basis of the mid-point of its viscosity range at 60°C measured in Pascal seconds (Pa.s.). The range of viscosities is imposed at 60°C and 135°C to confine bitumen to an acceptable range of temperature susceptibility. To exclude bitumen with high temperature susceptibility, which may be too brittle at low temperatures, a minimum limit is placed on the penetration at 25°C. These controls on consistency at the top, middle and near the bottom of the practical temperature zone are considered to adequately specify the rheological properties of bitumen and also focus attention on the bitumen consistency at temperatures that are relevant to possible performance problems (AS2008-1997).

4.2.1.1 Temperature Effects on Rheology of Bitumen and Asphalt

Rheology is the study of the deformation and flow properties of materials (AS39982-1991). Bitumen is a thermoplastic material whose strength and physical behavioral properties are directly related to temperature. At ordinary temperatures (10°C to 30°C), most bitumen is too stiff and hard to handle. For it to be sprayed, pumped and mixed or compacted in an asphalt mixture, its viscosity must be greatly reduced. Typical viscosity ranges and the approximate corresponding temperatures for typical Class 320 bitumen are given in Table 4.1 (Armour 1988, AAPA 1998b). The viscosity of bitumen can also be altered by fluxing or cutting where volatile fractions such as diesel and kerosene are added. This method is typical used for spray sealing surfacing work (NAASRA1984a).

It has been well established that the rheological properties of the bitumen binder affect the asphalt pavement performance. The properties of the binder affect the properties (e.g. resilient modulus and deformation resistance) of the asphalt for a given set of conditions of temperature and loading. Therefore, the sensitivity of the binder to temperature affects the properties of the asphalt (Austroads 1992a, Roberts et al 1996). Some typical relationships, between binder and asphalt properties for a range of temperature are given in Dickinson (1984).

Application	Viscosity Range	Temperature
Spraying Operations	0.05 to 0.1 Pas	>180°C
Pumping	0.5 to 1.0 Pas	130°C to 140°C
Multi-tyre rolling	2 to 100 Pas	70°C to 115°C
Steel wheel rolling	0.5 to 10 Pas	90°C to 145°C
Mixing	<0.2 Pas	>150°C

Table 4.1 - Typical Viscosity and Temperature Ranges for Class 320 Bitumen (After Armour 1988, AAPA, 1998a)

The higher temperature (40°C to 60°C) rheological properties are related to the rutting performance of pavements. The rheology at intermediate temperatures impacts on the fatigue cracking of pavements. The low temperature properties of the binder are related to the low-temperature thermal cracking of the pavement. Reduced rutting, improved fatigue life, and lower low-temperature stiffness values have been measured in asphalt mixtures made with binders with improved rheological properties (Bahia and Kamel 1994).

Temperature is the one of most important factors in determining the modulus of asphalt, fatigue life and permanent deformation of asphalt layers (AUSTROADS 1992a). Shell (1978) introduced the procedure for design purposes to determine “weighted” mean annual air temperature (w-MAAT) from mean monthly air temperatures (MMAT) from a given location. By using a relationship between air temperature and pavement temperature, “weighted” mean annual pavement temperature (WMAPT) can be determined (AUSTROADS 1992a). By using the WMAPT, the effects on design of daily and monthly variations in the pavement temperature are taken into account. The WMAPT gives the “effective” asphalt temperature and thus “effective” asphalt performance properties.

Rutting in asphalt occurs due to the plastic flow of the material. Plastic flow is an irreversible process caused by high stresses applied by vehicles, sustained elevated temperatures on a hot day or a combination of both (APRG 1992). Therefore, for assessing rut resistance, the Maximum Pavement Temperatures (T_{max}) is used (APRG 1997b).

Because of the effect of temperature on the rheology of the binder and the corresponding effect on the asphalt mix, laboratory temperature test conditions are chosen to reflect the expected in-service temperatures approximate to the critical performance conditions for the type of test being conducted.

4.2.1.2 Loading Rate Effects on Rheology of Bitumen and Asphalt

Because of the visco-elastic nature of bitumen, the properties of asphalt are dependent on the rate at which it is loaded. For example, a faster loading rate will give a greater modulus (as shown in Figure 4.1), increased deformation resistance and increased fatigue life in the controlled stress mode of loading. The effects can be very significant, especially in pavement areas such as intersections, bus stops and car parks.

To determine the properties for a given traffic speed, regardless of the method used, the loading time in seconds can be derived from a simple inverse relationship (AUSTROADS 1992a). Debate continues as to whether this relationship is appropriate. Vic Roads (1993) suggests that the relationship between loading time and traffic speed depends on the method used to estimate modulus. Guidance is given in selecting MATTA rise times and time of loading for the Shell monographs (AUSTROADS 1992a) for estimating asphalt modulus for various pavement design speeds.

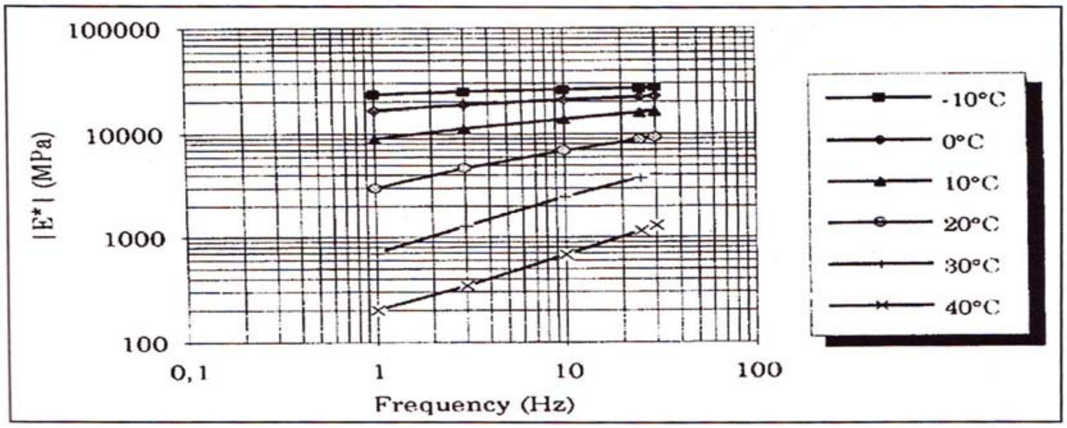

Figure 4.1 - Effect of rate of loading (Hz) on stiffness modules as a function of test temperature (After di Benedetto an de la Roche 1988)

4.2.2 Polymer Modified Binders

4.2.2.1 Types of Polymer Modified Binders

Whilst there are a large number of polymer products, only a few are suitable for modifying bitumen (Isacsson and Lu 1995). Polymers are commonly divided into three broad categories: plastics, elastomers and rubbers. Plastics can in turn be subdivided into thermoplastics and thermosets (or thermosetting resins) and elastomers into natural and synthetic rubber. Thermoplastics soften and flow when heated but reharden on cooling. The process can be repeated a large number of times. Thermoset materials are produced by the direct formation of network polymers from monomers, or by cross-linking linear prepolymers. Heating causes irreversible transformation as a result of chemical reactions. Elastomers are characterized in their elasticity, which allows them to totally or partially recover their initial dimension after being subjected to stress or an increase in temperature.

The most commonly examined thermoplastics for modifying bitumen include polyethylene (PE), polypropylene (PP), polyvinyl chloride (PVC), polystyrene (PS) and ethylene vinyl acetate (EVA) (Lu 1997). EVA, a random copolymer of ethylene and vinyl acetate, was used for modifying bitumen (APRG 1997A) but it is being replaced by EMA. EVA may improve high temperature properties without altering low temperature flexibility of the base bitumen which leads to improved deformation resistance of asphalt containing EVA modified bitumen (Maccarrone et al 1997a).

Whilst EMA and EVA modified bitumen's have increased fuel resistance, the plastomeric types should not be used where a high degree of flexibility is required (AAPA 1998a).

Important thermosetting polymers include alkyds, amino and phenolic resins, epoxies, unsaturated polyesters and polyurethanes. These polymers can increase the strength of the bitumen by reacting chemically to form a strong three-dimensional network structure that cannot be returned to a fluid condition by heating. Two-component epoxy resins blended with bitumen display the properties of modified thermosetting resins rather that those of bitumen. They give outstanding performance as road binders in cohesion, adhesion, oil/fuel resistance and durability. In recent decades, various thermoset-bitumen systems have been developed, but due to their cost are only applied to a limited number of critical pavement surfacing conditions (e.g. specific types of bridge decks and airfields) (Roberts et al 1996).

Elastomers (rubbers) such as natural rubber (NR), polybutadiene (BR), polyisoprene (IR), isobutene-isoprene copolymer (IIR), polychloroprene (CR), Styrene-butadiene copolymer (SBR) and wtyrene-butaduence-styrene block copolymer (SBS) have been used in experiments to modify bitumen (Isacsson and Lu, 1995, Roberts et al 1996). The polymers may be added to the bitumen in different forms such as crumbs, powders, lattices and solutions in liquid hydrocarbons. Of the elastomers, SBS copolymers have attracted the most attention for bitumen modification (Isacsson and Lu 1995). The polymers consist of styrene-butadiene-styrene triblock chains and are a two-phase system formed by polystyrene blocks (PS) within a matrix of polybutadiene (BR). Above the glass transition temperature of polystyrene (i.e. +100°C), the effectiveness of the polymer cross-links rapidly diminishes. When cooled, the polystyrene domains reform and the strength and elasticity are restored. The thermoplastic nature of SBS polymers at elevated temperatures and their ability to provide a continuous network on cooling are the reasons for their attractiveness as bitumen modifiers (Lu 1997).

The SBS modified binder is relatively insensitive to temperature and rate of loading and confers similar properties to the asphalt mix. Due to higher amounts of energy required to deform the PMB/SBS, it increases the resistance to deformation (rut) and reflective cracking of the asphalt (Srivastava et al 1992). Asphalt containing PMB/SBS has been shown to have lower flexural stiffness compared to the same mixes with C320 bitumen however the affect of changing the rage of loading was less significant for the PMB/SBS mixes (Maccarrone et al 1997a).

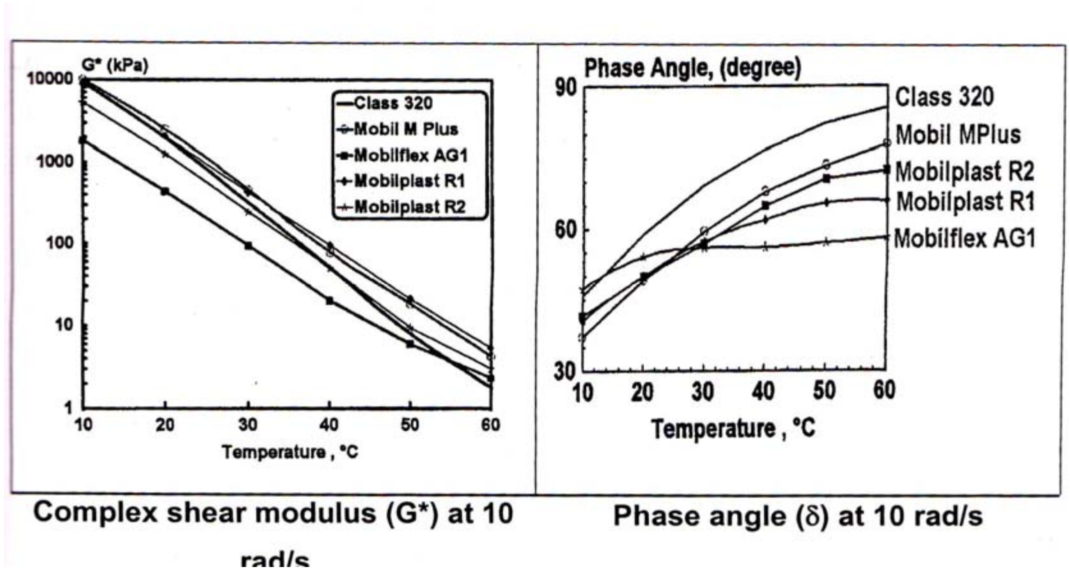

Figure 4.2 – Temperature dependency of complex shear modules and phase angle for Class 320 and polymer modified binders (After Maccarrone et al 1996 and 1997a)

The type of PMB most commonly used with SMA is SBS which is an elastomeric polymer type. Brown et al (1997a) reported that SMA incorporating an SBS PMB produced mixes that were more rut resistant than SMA with unmodified binder. The improved rut resistant properties of mixes manufactured with both SBS and EVA polymer modified binders were demonstrated in the Beerburrum (Queensland) Accelerated Loading Facility (ALF) trials (APRG 1996).

Superior fatigue lives are also reported as a consequence of using an SMA/SBS system. The properties of PMB are illustrated in Figure 4.2 (Maccarrone et al 1996 and 1997a), where SBS and EVA modified bitumen's are compared with unmodified Class 320 bitumen. The reduced phase angles dependency on temperature of SBS and EVA shows lower viscous flow and higher elasticity of the polymer modified binders. This translates to greater rut resistance and lower stiffness of the asphalt over a greater range of typical pavement temperatures. SBS demonstrates the best overall performance with respect to viscous resistance and elasticity.

4.2.2.2 Temperature Effects

Improvements in the low-temperature properties of bitumen due to polymer modifications were indicated by decreases in creep stiffness, Fraass breaking point and glass transition and limiting stiffness temperatures (Isacsson and Lu, 1995). The relative improvement varies with the base bitumen, polymer type and polymer content. Compared with thermoplastics (EVA and EBA). The elastomers (SBS and SEBS) appeared more effective in improving bitumen low-temperature parameters (except for the limiting stiffness temperature). For a given polymer, the relative reduction in creep stiffness depends on the bitumen grade (Lu and Isacsson 1997a). The influences of polymer modification may also vary with testing conditions (temperature and loading time) (Claxton et al 1996).

Polymer modification also significantly improves the temperature susceptibility of bitumen's. Temperature susceptibility is not a single-valued parameter but depends on temperature range being considered, loading time as well as the property being measured (Lu and Isacsson 1998). The service temperature range of bitumen's can be extended by the addition of SBS polymers depending on the concentration of the modifier and the nature of the base bitumen. The maximum achievable extension is in the order of 25°C for high temperature performance (rutting resistance) and up to 20°C for low temperature performance (thermal cracking) (Vonk and Valkering 1996). The effects of polymer modification can be unfavorable at low temperatures (Bahia 1994).

4.2.2.3 Measuring Effects of Polymer Modification

Statistically significant relationships exist between the parameters of dynamic mechanical analysis, creep tests and conventional methods (Lu 1997). Conventional test parameters such as penetration and softening point were not considered sufficiently adequate for characterizing polymer modified binders (Vonk and Valkering 1996). The degree of polymer modification reflected by difference measurements varies considerably and predicts significant benefits from polymer modification in practice, appropriate and fundamental parameters should be used (Lu and Isacsson 1997b). For example, whilst there is strong evidence that the rutting resistance of asphalt is dependent upon the visco-elastic properties of bitumen's and PMB's, debate continues over what is the most important parameter of the binders to be measured (Murray et al 1998).

Overseas research had reported an excellent correlation between rutting resistance and the Strategic Highway Research Program rutting parameter ($G^*/\sin\delta$) Claxton et al 1996, Murray et al 1998). Doubts about the reliability of SHRP rutting parameter when applied to SBS modified binders have been reported in Europe (Oliver 1996). The Australian study recommended that consistency, as measured on the ARRB TR Elastometer, be used to characterize PMBs n specifications for rut resistance and that softening point may be suitable for control of manufacture of the binders (Oliver 1997). These parameters are now included in Australian specifications (AP-T04, 2001).

An ongoing study has found that whilst the consistency property, measured using the ARRB TR Elastometer, changes with hot storage time for most polymer modified binders, the other properties do not change significantly. Research is continuing to determine whether there is any change in the laboratory performance (Resilient Modulus, Dynamic Creep, Wheel-Tracking Rate, Beam Fatigue) of asphalt mixes manufactured from PMB stored at high temperatures (Remtulla et al 2001).

4.2.2.4 Australian PMB Classification Systems

In Australia in 1992, the AUSTROADS (1992b) Committee on polymer modified binders (PMBs) released guideline and specifications covering the use and applications of PMBs in asphalt and sprayed sealing applications and presented procedures for handling, storage, application and safety. The approach adopted was largely prescriptive where polymer type and concentrations were the key focus (APRG 1997b). Also introduced was the nomenclature for eight sealing and seven asphalt PMBs with typical material properties. The binder type designations, SB"x" and AB"y", denotes sprayed seal and asphalt binders respectively. The numbers, "x" and "y", were intended for identification only, and not necessarily intended as a ranking. The Asphalt Binder classifications were AB1, AB2, AB3, AB4, AB5, AB6 and AB25. Information on the impact of PMB on the performance of Dense and Open Graded Asphalts were presented – SMA had not been used in Australia at that time.

A perceived problem with the 1992 specification was that users could associate classifications with polymer content (e.g. AB5 = 5%SBS) (Parry et al 1998). In 1997, a new PMB classifications system was introduced based on the type of binder system application as a prefix, an arbitrary numerical designation and the predominant polymer group represented by the PMB as a suffix. Binder systems are coded as "S" (for sealing grades) or "A" (for asphalt grades). Polymer groups are coded as "E" (for elastomeric polymer types), "P" (for plastomeric polymer types) or "R" (for granulated crumb rubber materials). For example, A10E is an asphalt grade PMB based predominantly on an elastomeric polymer type and arbitrarily designated as 10 (APRG 1997b).

The Code E includes SBS (styrene-butadiene-styrene), SIS (styrene-isoprene-styrene), SBR (styrene-butadiene rubber), natural rubber, PBD (polybutadiene), chloroprene and other similar polymer types. The Code P includes EVA (ethylene vinyl acetate), EMA (ethylene methacrylate), APP (atactic polypropylene), various forms of PE (polyethylene) and other similar polymer types. The Code R includes crumbed rubber usually from old tyres. Whilst the code letters allow various types of modifiers, those typically used for the grades of PMB for asphalt applications are A10E (SBS), A15E (SBS), A20E (SBS), A25E (PBD), A30P (EMA,EVA), A35P (EMA,EVA) and A40R (scrap rubber). Guidelines are included for the choice of PMB to improve the performance of Dense Graded Asphalt in rut resistance and fatigue resistance, Open Graded Asphalt and High Modulus Asphalt (APRG 1997b).

The 1992 AB25 and the post-1997 A40R classifications were included to allow the use of crumb rubber and were based upon the "dry: addition of the crumb rubber directly to the asphalt mixing plant (APRG 1997b). Inclusion of mixes manufactured by this method is inappropriate for a binder specification and the classification has been removed from later specifications (AUSTROADS 2000b).

APRG (1997b) was revised and issued in late 2000 as technical report AP-T04 "Austroads Specification Framework for Polymer Modified Binders" (AUSTROADS 2000b) and is the subject of ongoing review and will probably reduce the number of classes and adopt a more "user friendly" classification system.

Whilst based on APRG (1997b), Queensland Department of Main Roads introduced its own classification system (MRS11.18 of 12/99). It has five PMB classes for asphalt (A) and restricts the polymer types to SBS (S), EMA (M), and EVA (V). The numeric code denotes the minimum consistency in kPa.s at 60°C which is more meaningful than the arbitrary value adopted in AUSTROADS (2000b). The Queensland classifications (with the nearest AUSTROADS equivalent class) are A0.6S (A20E), A5S (A15E), and A10S (A10E, A1.8M (A30P and A2V (A35P).

4.2.3 Aggregate

Aggregate can be described as material composed of discrete mineral particles of specified size or size distribution, produced from sand, gravel, rock or metallurgical slag, using one or more of the following processes selective extraction, screening, blasting, crushing (AS1348.1:1986). Aggregate is normally classified as either coarse or fine however there is some difference of option as to where the transition occurs between coarse and fine aggregate. AS1348.1:1986 defines coarse aggregate as "of such size that is substantially retained on a sieve of specified size, commonly 4.75mm or 2.36mm according to usage." AS2758.5:1996 uses a definition of "an aggregate having a nominal size of not less than 5mm. The convention used by road authorities in Australia is that the 4.75mm sieve is the transition between coarse and fine aggregate (APRG 1997a, DMR (QLD) MRS11.33, RTANSW 1998). This convention will be adopted for this paper.

Aggregates typically make up 90% to 95% of the mass of all asphalt mixes and provide a very substantial proportion of the load carrying capacity. Their selection is critical to mix performance. Specifications and standards set limits on the test properties that must be achieved by the aggregates (AS2758.5:1996, DMR (QLD) MRS11.33). Most aggregate tests and/or related specifications have been developed over a period of time and reflect local conditions and properties of available aggregate sources (Kandhal et al 1997).

4.2.3.1 Coarse Aggregate

The Queensland Department of Main Roads asphalt specifications (MRS11.30, MRS11.33, MRS11.34 and MRS11.36) set the same limits for coarse aggregate for all asphalt types. The test included are flakiness index, ten percent fines value (Wet), wet/dry strength variation, degradation factor, water absorption, crushed particles, weak particles and polished aggregate friction value (PAFV). Based on the predicted life of SMA, research could be justified as to whether there should be a separate (more stringent) requirement of PAFV for SMA. PAFV values have been the centre of much discussion in the Northern Region of Qld. Main Roads, and a separate specification was developed with a higher reading than the rest of the state – 50 vs. 45 – see Appendix C for results from local quarries which form the aggregate supply. Also consult Appendix G for a copy of the Interim Specification for SMA for District 11 Peninsula: the only area with its own unique modifications to the standard specification in the state out of 15 Districts. This document will be referred to throughout the paper. More stringent requirements for flakiness index and ten percent fines value (Wet) have been specified for heavy duty pavements (Jones et al 1998).

For asphalt to be used in heavy and very heavy traffic applications, AAPA (2000b) sets more stringent limits for the coarse aggregate properties of los angeles abrasion loss, unsound stone content, marginal stone content, flakiness index, water absorption and PSV/PAFV. The expected service life of SMA is greater than DG and OG asphalt for the same applications. This implies that SMA will be subject to greater traffic during its life. It can therefore be seen that the two intrinsic properties required of the coarse aggregate in SMA are strength and polishing resistance (Loveday and Pellin 1998).

4.2.3.2 Fine Aggregate

The Queensland Department of Main Roads asphalt specifications (MRS11.30, MRS11.33, MRS11.34 and MRS11.36) set the same criteria for fine aggregate for all asphalt types. The specifications are brief and simply state "Fine aggregate shall consist of natural sand particles and/or crushed rock or crushed stone particles of size smaller than 4.75mm but larger than 0.075mm. The aggregate shall be clean, hard, durable, and free from clay and other aggregations of fine material, soil, organic material and any other deleterious material". A recent specification for a heavy duty pavement in Queensland included additional limitations on maximum Plasticity Index and Clay Index of the fine aggregate (Jones et al 1998).

The selection of fine aggregate is a balance between stability and workability. For heavy duty applications (where SMA is typically used), where a high degree of rutting is required, wholly crushed fine aggregate is recommended (AAPA 1998a). Some specifications for SMA limit the amount of natural sand to a maximum of 50% of the fine aggregate (Nicholls 1998(a), prEN 13108-5:2000).

4.2.4 Filler

Filler is a fine material, the majority of which passes a 75µm sieve derived from aggregate or other similar granular material. The common fillers used in Australia are portland cement, hydrated lime, and ground limestone; cement plant flue dust, fly ash, ground slag and bag-house dust (APRG 1997a). The choice of filler will normally depend on availability and cost, although hydrated lime (Baig and Wahhab 1998) and Portland cement both have adhesion improvement qualities (APRG 1997a).

According to numerous studies the properties of mineral filler have a significant effect on the properties of the asphalt in terms of permanent deformation, fatigue cracking and moisture susceptibility. The introduction of environmental regulations and subsequent adoption of dust collection systems (baghouses) has encouraged the return of most fines to the asphalt mixture. Maximum filler to binder (F/B) ratio of 1/2 to 1/5, based on weight, is used by many agencies to limit the amount of minus 0.075mm material. However, the fines vary in grading, particle shape, surface area, voids content, mineral composition and physio-chemical properties; therefore, their influence on the properties of asphalt also varies. The maximum allowable amount should thus be different for different fines (Kandhal 1981).

Fillers for use in Australia are currently covered by AS2357:1980. As well as those materials listed in Table 3.7 of the Australian Standard filler may be “any other materials nominated or approved by the purchaser”. The Queensland asphalt specifications (DMR) QLD) MRS 11.30, MRS11.33, MRS11.34 and MRS11.36) refer to the combined filler complying with AS2357 with the additional requirements of being… “Free from lumps, clay, organic matter and any other deleterious material” and with “voids in the dry compacted filler of not less than 38%.”

The F/B ratio is particularly important in the climatic condition of North Qld. It has been monitored closely in each of the plants within the area and forms apart of the mix design approval, as well as part of the modification to the interim specifications.

4.2.5 Fibres

Fibers used in SMA are typically cellulose fiber in a palletized form pre-blended with bitumen, or in loose form. In the early nineties several other stabilizers were trialed including glass fiber and rock wool fiber. Recent trials using acrylic fibers have been conducted by QDMR in North Queensland.

Fibers can generally be purchased in two forms, loose fiber and pellets. Loose fiber can be packaged in plastic bags or in bulk. Pre-weighed plastic bags are often used in batch plant production. The bags are made from a material that melts readily at mixing temperatures. The appropriate number of bags can, therefore, be added to the pug mill during each dry mixing cycle. Filler bags can be elevated to the pug mill platform by conveyor and manually added to each batch. The process is labour intensive and best suited to relatively low tonnage projects.

An alternative to the use of pre-weighed bags is to blow a metered amount of bulk material into each batch. Machines specially designed for handling and blowing bulk fiber materials have been developed by some fiber manufacturers. Accurate calibration and control of density, as well as separation of fibers so that they adequately disperse in the asphalt mixture, are important features of such materials. The Pioneer Plant in North Queensland has a blower attached to the plant, where-as Boral is still very labour intensive.

In batch plants the fibers are generally dispersed by adding to the dry aggregates and dry mixing for a short time (no more than 10 seconds), although some fiber manufacturers recommend adding fiber at the same time as binder.

The fiber blown system can also be used in drum mix plants. In this method it is imperative that the fiber line is placed in the drum beside the bitumen line and merged into a mixing head so that the fibers are captured by the bitumen before being exposed to the high velocity gases in the drum. If fiber is not properly captured by the binder it will be lost to the dust collection system.

Palletized fibers can be used in both batch and drum mixing plants by adding direct to the pug mill or at the RAP collar of drum mixing plants. Palletizing assists in improving the ease and accuracy of metering as well as reducing the likelihood of fibers becoming airborne and carried out by the plant draft. Some palletized fibers are mixed with a small amount of bitumen binder that must be allowed for in the overall binder content of the mix. Bulk palletized materials are placed in a hopper and metered into either the pug mill or drum mixer. Appropriate feed calibration is an important step in all fiber addition systems.

Compared to conventional dense graded asphalt mixes, SMA mixes generally have higher bitumen content and contain more coarse aggregate.´Drainage´ occurs when the bitumen runs off and through the aggregate during delivery or in the hopper of the paver. The main causes of drainage are excessive binder content in the mix and high temperature of the mix. The worst problems were reported on SMA projects where there was a combination of excessive binder content as well as high mix production and placement temperatures (Scherocman 1997). Fibres are used as drainage inhibitors in asphalt and the types available include cellulose, rock wool, fiberglass and other mineral sources. German SMA was originally produced using asbestos fibres. Whilst asbestos was perfectly suited from a technical point of view, its application was prohibited for health reasons. Cellulose fibres now have a 90-95% share of the German market (Schrimpf 1998). The Schellenberg drain-down test can be used to determine mix propensity to “draining” (APRG 1997a).

Because of their increased stiffness, multigrade binders and Polymer Modified Binders (PMB) may reduce the need for binder drainage inhibitors however at the high bitumen contents typically used, a drainage inhibitor will still be necessary. Brown et al (1997a) compared the effect on binder drainage of the incorporation of SBS and polyolefin polymers and cellulose and rock wool fibres into the bitumen binder. They concluded that the fibre stabilizers were superior to the polymer stabilizers in preventing binder drainage in SMA mixes. The polymer stabilizers however produce more rut resistant mixes as they provide increased mastic support to the stone skeleton.

Polymer fibres do not normally interact chemically with the bitumen. Experiments with the use of fibres to modify bitumen and asphalt have investigated increasing the toughness of the asphalt by increasing the amount of energy absorbed during fatigue and fracture testing (Lu 1997). One study demonstrated that the addition of cellulose fibre to SMA mixtures may affect performance in ways not being considered at present. While addition of small amounts (0.3%) will reduce binder drainage, greater additions may affect in-service properties such as cohesiveness, stiffness and resistance to permanent deformation in unexpected ways which need to be considered when predicting in-service performance (Woodside et al 1988).

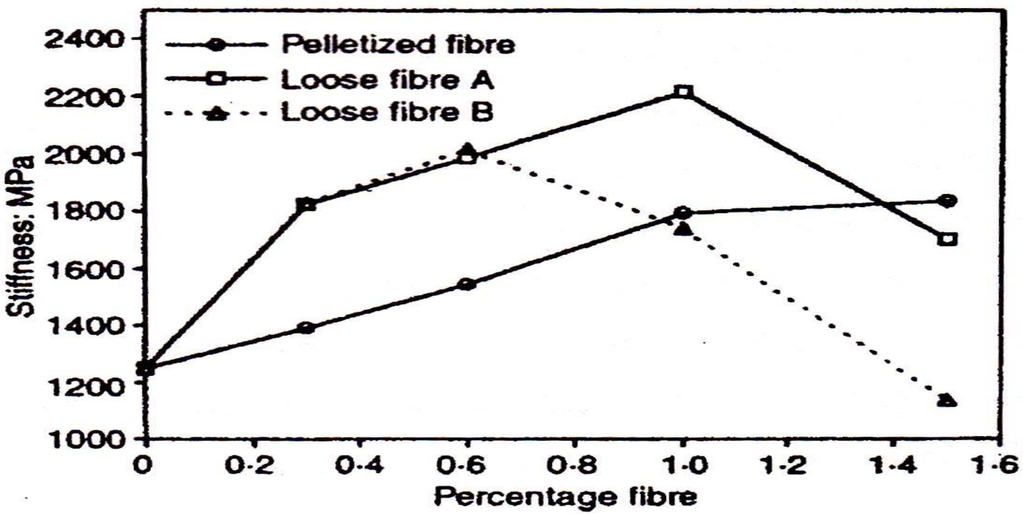

Figure 4.3 - Effect of cellulose fibre addition on SMA mixture indirect tensile stiffness modulus (After Woodside et al 1998)

The addition of cellulose fibres can affect the stiffness modulus as measured with the indirect tensile test as shown in Figure 4.3 (Woodside et al 1998). In terms of maximum stiffness, the optimum addition of loose fibre was dependent upon the fibre, that is, 1.0% for loose fibre A and 0.6% for loose fibre B. Stiffness also increased with the addition of palletized fibre; however, an optimum value had not been reached with the addition of up to 1.5% fibres. SMA mixes typically contain 0.3% of cellulose fibres to prevent binder drainage.

4.3 Asphalt Mix Design

4.3.1 Background to Mixture Design Studies

Asphalt mixture design is undergoing a transition throughout the world from the empirical mix design methods (for example, Marchall and Hveem) to performance-based or performance-related methods (Luminari and Fidato 1998). The aim of the design of asphalt mixes is to determine the proportions of bitumen, filter, aggregate and any other materials that will produce a mix that meets the appropriate constructability and performance criteria (Lay 1998). Whilst pavement design methods use values for stiffness and fatigue relationships, there is also a range of serviceability issues that need to be addressed as part of the mix design process. These include resistant to permanent deformation (rutting), resistance to low temperature cracking, durability, and resistance to moisture induces damage, skid resistance and workability (Roberts et al 1996).

4.3.2 Performance Based Mix Deign Methods

4.3.2.1 United States of America

The American Superpave ™ mix design method was the end result of the "Asphalt" research effort of the Strategic Highway Research Program (S.H.R.P.). The Program operated from 1987 to 1992 and represents the integration of more than 25 research areas in a single system for the characterization and design of asphalt mixes. The procedure contained in this method includes specifications of the raw materials, test methods with the specially developed equipment, the strict mix design method itself, and a related software system ("core"). The system is referred to as the *Su*perior *Per*forming Asphalt *Pave*ment System that has been termed Superpave ™.

Superpave ™ is supposed to substitute the specifications on materials and the mix methods currently used in the 50 states of the USA (Roberts et al 1996). This will create a single, performance based system which can provide results tailors to the different climatic and traffic conditions present for the different classes of roads in the United States and Canada. This method is applicable to mixes such as Hot Mix Asphalt (HMA) whether virgin or recycled, of closed (dense) grading, with or without modifies bitumen, as well as a variety of special mixes such as Stone Matrix Asphalt (Luminari and Fidato 1998). It can also be applied to newly constructed wearing courses, base course layers and for resurfacing of deteriorated pavement surfacing layers, with the aim or selecting appropriate materials, reducing and controlling permanent deformations, and cracking, whether due to fatigue or low temperature. The flexibility of the system permits mix design taking into consideration, both separately and in combination, the three main distress factors, and predicting, the influence of aging and sensitivity to water at the onset of these types of deterioration (Roberts et al 1996). The usage and relevance of the US SHRP test methods and specifications are being considered by a number of countries (Stacey 19994, EAPA 1999).

4.3.2.2 Europe

During the 1990's, the implementation of the European Union single market has given transport issues, including road pavements, a stronger European dimension. Cross border contacts and co-operation has grown rapidly. The EU Commissions views on the key elements affecting the competitiveness in the construction sector include quality, construction process, regulatory environment and technology. (EAPA 1999) One of the major impediments to effective competition has been the difference standards and techniques applied by different countries. Work is currently underway to harmonise the standards or road building materials in the countries of the European Union (EAPA 1999).

Between 1988 and 1995, two succeeding RILEM Committees: TC 101 BAT "Bitumen and Asphalt testing "and TC 152 PBM "Performance of Bituminous Materials" concentrated a major effort in this area (Francken 1998). An early part of this research was a "state of the art" review of asphalt mixture design methods and Luminari and Fidato (1998) outlined the advantages and disadvantages of each method.

Elements for a new mix design method were proposed and the aims were to evaluate the possibilities of implementing rational concepts and testing procedures for the design and manufacture of bituminous materials in order to cope with the present and future conditions of use in pavement construction. The final purpose of the committees was to recommend significant test procedures for binder evaluation, mix design and performance assessment of bituminous materials. A large international inter-laboratory testing program was undertaken (Eustacchio et al 1998). In an era when strong changes are expected in experimental procedures and specifications for asphalt materials in Europe, the RILEM project provides some of the fundamental research for future decisions. (Francken 1998)

4.3.2.3 Australia

In 1988, AUSTROADS, the Australian Asphalt Pavements Association (AAPA) and ARRB Transport Research (ARRB TR) started work on a project to develop a performance based asphalt mix design procedure which measured relevant, fundamental mix properties. This required the development of test equipment that was affordable, accurate and easy to use (Wonson and Bethune 2000). This new approach is intended to replace the existing Marshall and Hubbard Field procedures. The intention of the procedures is to allow cost effective asphalt mixes to be designed for mechanistic pavement design. The guide steers the users through the various processes and indicated the type of results that can be expected (APRG 1997a). Figure 4.4 illustrates the new provisional Australian Procedures.

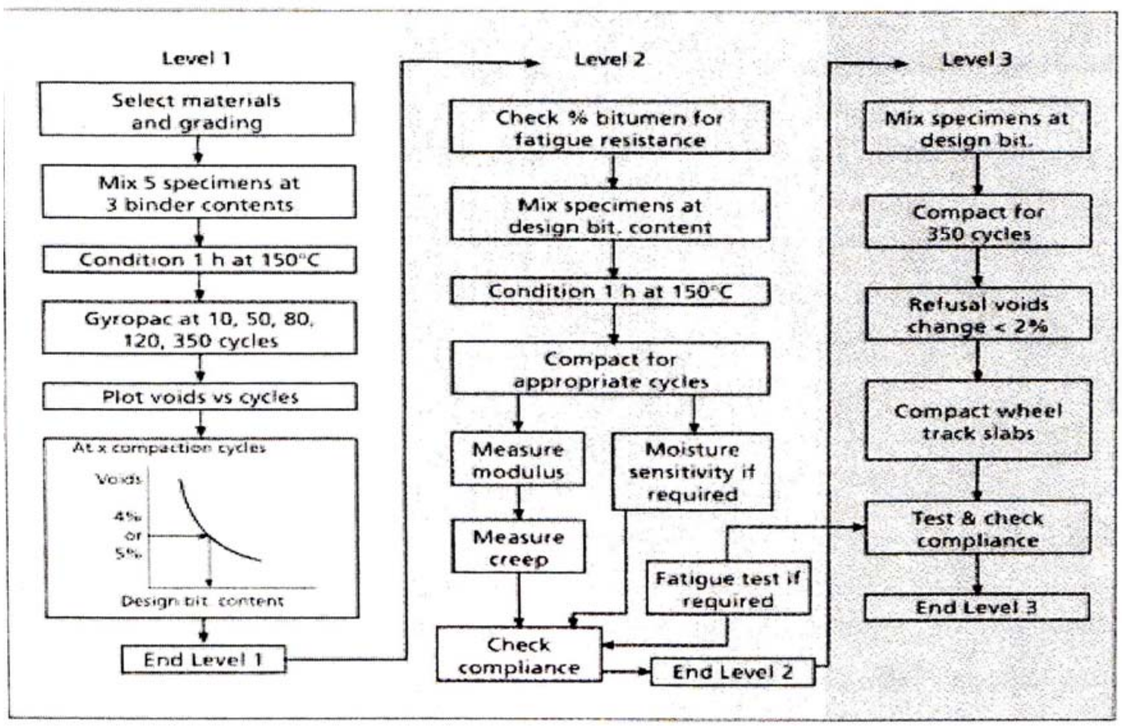

Figure 4.4 - Flow diagram of asphalt mix design by the AUSTROADS (APRG 1997a) method.

The mix procedure is divided into distinct phases. In the first phase, the volumetric proportions are determined. The second phase involves carrying out performance related tests such a resilient modules, dynamic creep and fatigue to ensure that they meet specified acceptance criteria. The uncompacted asphalt is conditioned by heating in an oven for 1 hour at 150°c to simulate binder hardening during transport and placing and approximates one or two years or post construction compaction by traffic.

Mixes are designs for light, medium and heavy traffic situation that are simulated by 50, 80 and 120 cycles respectively by the gyratory compactor.

- For level 1 (light traffic situations) the mix design process ends once the volumetric properties of the compacted conditioned sample have been satisfactorily achieved.
- For level 2 (medium and heavy traffic situations) performance related testing is undertaken after the volumetric properties are achieved.
- For level 3 (very heavy traffic situations) the mix is also subjected to 350 cycles of the gyratory compactor to ensure there are at least 3% are voids at what could be considered refusal density for the mix. Further performance related testing may be undertaken. This requirement is currently under review. With the new proposed limits being a minimum of 2.0% voids at 250 cycles (Oliver 2001).

4.4 Material Properties

The majority of test methods for compacted asphalt and its component materials such as aggregates, bitumen binders, fillers and fibres are covered by Australian Standard test methods such as the suite of tests contained in AS 2891 “Methods Sampling and Testing Asphalt” or State Road Authority test methods (DMR(Qld) 1998a). Many of these tests are based on the American Society of Testing Materials (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) methods. Brown et al (1996) gives a good overview of the significance and expected results from the ASTM and AASHTO methods.

The traditional mix design tests like Marshall Stability and flow are still widely used throughout the world (Luminari and Fidato 1998, EAPA 1998, DMR (Qld) MRS11.30 to MRS11.36). These traditional mix design methods (e.g. Marshall and Hveem) are neither performance-based nor performance-related. They try to predict performance by measuring certain empirical properties, but neither method can ensure that the designed mix conforms to specific pavement performance criteria (Lumunaru and Fidato 1998).

The recent trend has been to develop and use compaction methods and test that simulate the field conditions and the introduction of performance based and performance related specifications. Performance based test methods are typically conducted under similar conditions as in the pavement and allow determination of physical properties that are directly related to the performance. Performance related properties are indirectly linked to and influence the pavement response to loading, however do not control the performance such as the mineral aggregate specification. Undertaken between 1987 and 1992, the major project in this area was United States' Strategic Highway Research Program (SHRP) which developed the Superpave™ mix design methods (Luminari and Fidato 1998). Australia has adopted a similar performance related design method (APRG 1997a).

The main thrust of the Australian Asphalt Pavement Association Research and Development (AAPA R&D) program, established in 1988, has been to develop equipment and test methods to determine the fundamental properties of asphalt mixes. These properties can be used in pavement design and to provide a datum whereby new and improved mixes can be rationally compared. This has been coupled with the desire that the new methods involve relatively simple, low cost testing equipment and test methods. In 1998, the AAPA R&D Policy Group selected three asphalt mix parameters as being the most important for investigation in their first program. These were Stiffness Modulus, Deformation Resistance and Fatigue Resistance. As an outcome of this research, the following test methods have been produced (Wonson and Bethune 2000):

(a) <u>Australian Standards</u>

- AS 2891.2.2 – 1995, Sample Preparation – Compaction of asphalt test specimens using a gyratory compactor.

- AS 2891.12.1 – 1995, Determination of the permanent. compressive characteristic of asphalt – Dynamic creep test.

- AS 2891.13.1 – 1995, Determination of the resilient modulus of asphalt – Indirect tensile method.

(b) AUSTROADS Provisional Methods

- AST01: 1999, Deformation resistance of asphalt mixtures by the wheel tracking test.

- AST02: 1999, Stripping potential of asphalt – tensile strength ratio.

- AST03: 1999, Fatigue life of compacted bituminous mixes subject to repeated flexural bending.

- AST04: 1999, Asphalt binder content – ignition oven method.

- AST05: 1999, Sample preparation – compaction of asphalt slabs suitable for characterization.

- AST06: 1999, Asphalt binder drain off.

- AST07: 1999, Asphalt particle loss.

The Australian provisional mix design procedure includes performance related testing in the following areas – Resilient Modulus (Indirect Tensile Modulus), Dynamic Creep (Direct Compression Test), Wheel Tracking Test, Beam Fatigue (Flexural Test) and Abrasion Loss (APRG 1997a).

4.4.1 Measurement of Air Voids Content

Asphalt mixes in Australia are specified in terms of the compacted voids content: measured and reported as the degree of compaction. The fundamental performance properties of compacted asphalt such as Resilient Modulus, Dynamic Creep, Wheel Tracking Rate, Fatigue, Permeability and Marshall Stability are directly related to the voids content (APRG 1996, APRG 1997a). Whilst typical relationships for Dense Graded Asphalt were developed as part of the Accelerated Loading Facility (ALF) trials conducted at Beerburrum in Queensland (APRG 1996), the establishment of similar relationships for SMA is the subject of continuing research.

The amount of type voids in a mixture is not only a function of the amount of compaction or quality of the binder but also the grading of the aggregate, all of which will affect performance (Laitinen 1998). There are several methods of measuring and classifying voids in asphalt and many relationships between those methods. These are categorized in Figure 4.5 (NAASRA 1984a). Whilst many countries use the same terminology, different definitions and formulae may result in different values being quoted for an identical mixture depending on the methods specified. It is important to be aware of the specification and the test methods being used when comparing results from different countries and even within the same country (Laitinen 1998).

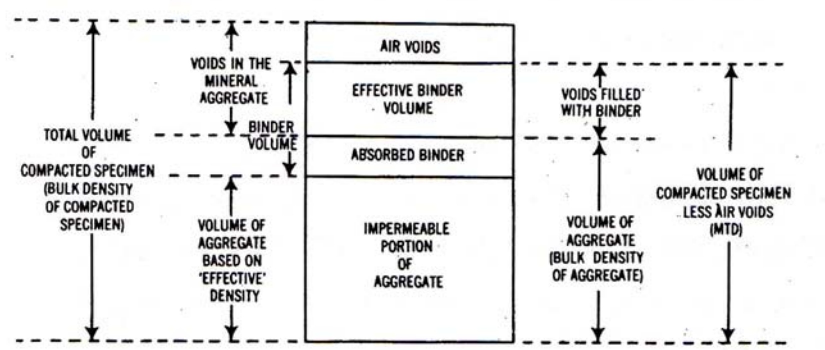

Figure 4.5 - Voids relationships in asphalt mixes (After NAASRA 1984a)

In Australia the air voids in the compacted sample are calculated from following equation (AS2891.8 – 1993):

$$AV = \frac{P_{max} - P_{bulk}}{P_{max}} x\ 100$$

Where AV = air voids in the compacted sample, in percent

P_{max} = maximum density of mix, in tonnes per cubic metre (From AS 2891.7.1, AS 2891.7.2 or AS 2891.7.3).

P_{bulk} = bulk density of the compacted mix, in tonnes per cubic metre (from AS 2891.9.1, AS 2891.9.2 or AS 2891.9.3).

From the definitions associated with this equation, there are 3 allowable methods for the determination of the maximum density of the mix and three methods for the bulk density of the compacted mix. The maximum density of asphalt can be determined by displacement in water (AS 2891.7.1), trichoroethane (As 2891.7.2) and methylated spirits (AS 2891.7.3). The methods involving chemicals especially trichoroethane are now infrequently used due to workplace health and safety issues. The water displacement methods is in common use in Australia (APRG 1997a), as well as many overseas countries (Brown et al 1996, Laitinen 1998), because it does not require any solvents or specialized equipment. The maximum density can also be calculated by using aggregate, filler and binder proportions and bulk densities of each (AS 2891.8 – 1993).

The water displacement method may give a slightly different maximum to that calculated from the individual components but is more accurate and quicker for a specific mixture (Laitinen 1998). It has been long recognized that the method used to determine the maximum density should always be reported (NAASRA 198a, Laitinen 1998).

The Australia Standards method allows three methods of determining the bulk density of the compacted asphalt. The waxing (AS 2891.9.1) and pre-saturation (AS 2891.9.2) methods involve immersing the samples under water to determine a buoyant mass which is used to calculate the sample volume. These methods are not suitable for mixes with high voids contents and large interconnected voids. The mensuration procedure (AS 2891.9.3) involves measuring the diameter and height of the specimen which is used to calculate the sample volume. Australian Road Research Board recommends the mensuration procedure for calculating the air voids in Open Graded and Stone Mastic Asphalt mixes (APRG 1997).

One of the limitations of the wax coating method is that the specimens may not be suitable for further testing due to the wax on the exterior of the specimens (Harvey et al 1994). The Queensland Department of Main Roads has wax sealed (Q306A) and pre-saturation (Q306B) methods for dense graded asphalt and mensuration (Q306D) method for open graded asphalt, however there is also a silicone coated (Q306C) method for dense graded asphalt. It is used in preference to the wax coating method where the asphalt core or pat has excessive voids and where the results may be affected by water absorption during testing. After testing for bulk density, the silicone coating can be removed to allow further testing to be undertaken (Q306C). There is no test method particularly stated for SMA, however DMR (Qld) Specification for Stone Mastic Asphalt (MRS11.33) calls up the silicone coated (Q306C) method.

A decision was undertaken at the national level in Australia that asphalt test methods should be standardized across the country. The methods adopted for bulk density were the pre-saturation (AS 2891.9.2) and mensuration (AS 2891.9.3) procedures. For maximum density, the water immersion or 'Rice' method (AS 2891.7.1) is recommended (APRG 1997a). Despite this decision, DMR (Qld) continues to specify its silicone coated method for the measurement of bulk density.

A comparison between voids content measured by the pre-saturation (water emersion) and mensuration methods in given in Figure 4.6 (Oliver 2000) for laboratory compacted mixes. This figure shows that the relationship between voids content measured by the two methods is dependent on the mix type and the size of the sample being measured. A straight line relationship appeared to give the best fit for the DG mix over the range data. For the SMA mixture, a logarithmic of similar curve best described the relationship. At high voids contents, the voids in the mix will become interconnected and thus, whilst the mensuration voids will continue to increase, the water immersion voids will not. This will result in a curve that is asymptotic to the horizontal axis at high voids contents (Oliver 2000).

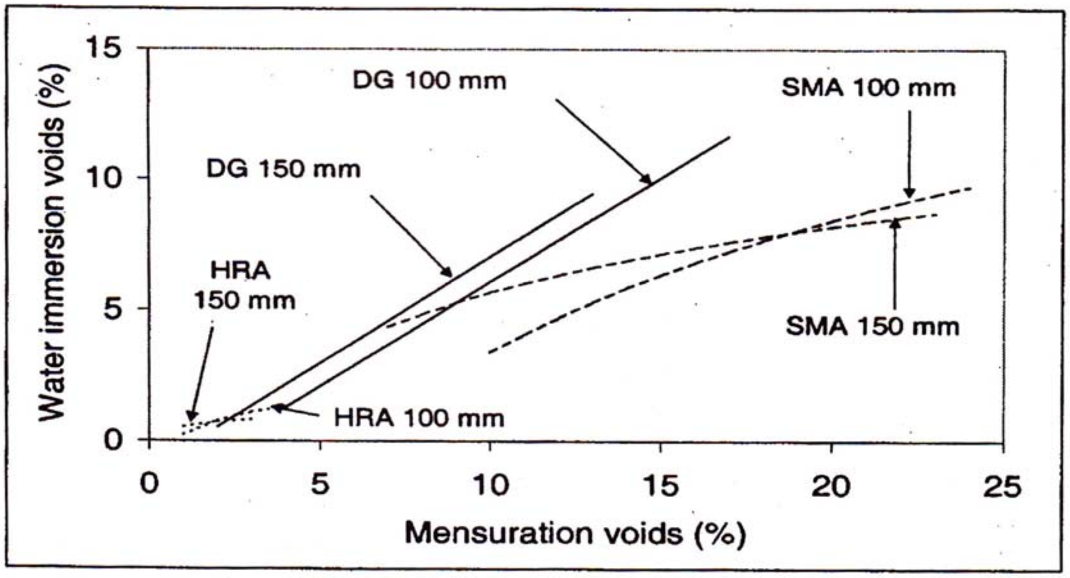

Figure 4.6 - Comparison between voids contents measured by two methods for three mixes and two sample diameters (After Oliver 2000).

4.4.2 Laboratory Compaction Methods

APRG (1997a) sets out the number of laboratory compaction cycles to be used in the design of asphalt mixes for varying traffic categories. The gyratory compaction levels adopted are intended to simulate the compaction which occurs on the road after some traffic. They were determined by a national exercise which correlated gyratory compaction with Marshall Compaction since extensive field experience indicated that Marshall Compaction densities were usually achieved in the field after several years of traffic compaction. Generally, 75 blows Marshall compaction is considered to be equivalent to heavy traffic compaction in the field, 50 blow Marshall to medium traffic, and 35 blow Marshall to light traffic. It was determined that 50, 80 and 120 gyratory cycles were approximately equivalent to 35, 50 and 75 Marshall blows, respectively (APRG 1997a).

SMA is typically used in heavy duty applications (EAPA 1998, AAPA 2000a), therefore it would be expected that the mix would be designed using 120 cycles or 120/350 cycles to be consistent with APRG (1997a). It is inconsistent that APRG (1997a) requires that the mix design criteria should be achieved after 80 gyratory cycles. Later research by the ARRB TR (1998) used 120 cycles.

4.4.3 Modulus of Elasticity

The dynamic complex modulus and resilient modulus tests are used to measure the modulus of elasticity (Croney and Croney, 1998). A major difference between the resilient modulus and the complex modulus us that inelastic as well as elastic deformations are measured in the complex modulus test (Roberts et al 1996). The notion of resilient modulus has been defined to make better allowance for the conditions of loading on pavements. The resilient modulus, or reversible modulus, corresponds to the ration of repeated stress to the reversible strain. The Europeans use the complex modulus (Francken Vanelstraete, 1996, di Benedetto and de la Roche, 1998) where as the USA favours the resilient modulus (Barksdale et al, 1997).

Because of its simplicity and applicability to test field cores, the repeated load indirect tensile resilient modulus test is the common method of measuring stiffness modulus of asphalt in the USA (Roberts et al 1996). Although there appears to be some flaws in the methods, the advantages appear to outweigh the disadvantages (Read and Brown 1996).

Whilst some researchers have concluded that the use of the indirect tensile test must be restricted to temperatures below either 20°C (di Benedetto and de la Roche 1998) or 30°C (Read and Brown 1996) to ensure that the linear elastic theory is applicable for bituminous materials, Barksdale et al (1997) performed their indirect tensile testing programme at 5°C, 25°C and 40°C. AS/NZS 2891.13.1 specifies a test temperature of 25°C however the test consists of 5 conditioning pulses followed by 5 loading pulses and creep may not be a concern over such a short loading period. Creep may become an issue when the indirect tensile test is used for longer term fatigue testing.

4.4.3.1 Australian Test Procedures

Australia has adopted the resilient modulus as measured by an indirect tensile test procedure to describe the elastic modulus of asphalt (APRG 1997a). During 1990, a Melbourne company, Industrial Process Controls Ltd, developed a piece of apparatus similar to the Nottingham Asphalt Tester (NAT) called the Materials Testing Apparatus (MATTA). By coupling developments in pneumatic control values with digital control technology, the new machine was able to equal an even exceeds the performance of many expensive electro-hydraulic machines (Tritt and Feeley 1994). The MATTA has been adopted as the 'standard' equipment to be used for the determination of the resilient modulus using a pulsed load with standard reference test conditions (AS/NZS 2891.13.1).

4.4.4 Fatigue Testing Methods

Different test methods have been used through out the world to measure fatigue. The principal methods and their characteristic are discussed by Matthews et al (1993). It was considered that the repeated flexure test offered the best combination of simulation of filed conditions and simplicity of test. Sinusoidal and haversine wave shapes are now the most commonly used in laboratory fatigue tests (Said 1998, 1996).

4.4.4.1 Interpretation of Test Results

Following an extensive literature review, Baburamani (1999) concluded that there are 3 main methods used to evaluate and predict the fatigue characteristics of asphalt mixes. They are initial strain – fatigue life, dissipated energy – fatigue life and facture mechanics – rate of crack propagation. A brief review of each method is given below.

4.4.4.1.1 Initial Strain – Fatigue Life

The results of controlled strain and controlled stress testing can be interpreted in terms of a relationship between life, *N*, and the initial strain amplitude (με). In the conventional strain approach, the relationship is given by an inverse relationship:

$$N = \left| \frac{K}{\mu\varepsilon} \right|^{b}$$

Where K and b are mix-dependent constants. The values of K and b vary with bitumen/binder type, temperature and frequency of loading. The exponent 'b' features in all the fatigue life prediction approaches and this form of the relationship can be readily used as part of the mechanistic design process.

4.4.4.1.2 Dissipated Energy – Fatigue Life

Fatigue damage in viscoelastic materials can be explained using stored and dissipated energies. The energy balance is influenced by rheological properties of the mix and the binder, which are in turn functions of temperature, frequency or loading and stress/strain. Development and accumulation of damage is evaluated in terms of dissipated energy and number of cycles. The initial phase angles between stress and strain waveforms are indicative of the viscous or elastic nature of the material. During a dynamic bending test in controlled stress or strain sinusoidal loading force, phase angle and dissipated energy/cycle per volume will change due to the change in the mix behaviour and damage accumulation.

The relationship between the number of cycles to fatigue failure N_{FAT} and the cumulative (total) dissipated energy per unit volume W_{FAT} is given by:

$$W_{FAT} = C(N_{FAT})^{m}$$

Where C and m are experimental constants related to mix stiffness and phase angles (Shell 1978). It is claimed that factors such as test type, temperature, frequency of loading, rest periods and mode of loading (controlled stress or strain) do not appear to influence the relationship between cumulative dissipated energy to failure and number of cycles to failure (Baburamani and Potter 1996), however this may need to be moderated in light of the results obtained from the Strategic Highway Research Program (SHRP) (di Benedetto and de la Roche 1998).

Whilst APRG (1997a) states that interpretation of the dissipated energy data was one of the measures under consideration, it offers no guidance as to how the data was to be assessed. The test method (AST 03:1999) requires that initial dissipated energy per cycle and cumulative dissipated energy at failure are recorded. Because there is no clearly defined failure point for constant strain testing, it may be difficult to apply the dissipated energy approach (di Benedetto and de la Roche 1998).

The dissipated energy approach assumes that the dissipated energy causes damage to the asphalt. Some researchers have challenged this assumption because the dissipated energy does not destroy bonds, but rather appears to act to mainly to mainly to heat the specimen (di Benedetto and de la Roache 1998).

The application of energy considerations to the structural design of asphalt pavements appears to be the major difficulty to be overcome for the wide spread use of the cumulative dissipated energy approach for the design pavements (Matthews et al 1993), when compared to strain based life prediction models that can be readily used in accepted design procedures (AUSTROADS 1992a).

4.4.4.1.3 Fracture Mechanics – Rate of Crack Propagation

In fracture mechanics, fatigue is considered to develop progressively through the 3 phases of crack initiation, stable crack growth and unstable crack propagation. It is assumed that the second phase consumes most of the fatigue life and fracture mechanic models have been based on this phase using long established fracture mechanics principles. The Paris law of crack propagation relates the increase in crack length per load cycle to the stress intensity factor, K_C. It also provides a means of including specimen configuration, boundary conditions and load effects and a means of estimating the size of the plastic zone ahead of the crack tip, in a power law relationship. Pronk (2001) has attempted to apply fracture mechanics to SMA, however found that due to the stone matrix, failure consisted of a series of parallel cracks which made it impossible to define a crack speed or measure the effective crack length. The Paris law of crack propagation could not be applied to the SMA.

4.4.4.2 Loading Type

It is widely accepted that the mode of loading has an influence on the laboratory fatigue results because the response of asphalt mixes varies according to the input constraints, i.e. force (stress) or displacement (strain). In the controlled strain test, the displacement amplitude is maintained constant and the force required to maintain the initial strain level decreases gradually after crack initiation, as the flexural stiffness of the mix is effectively decreased. The failure, or termination point, is arbitrarily selected as a certain reduction (generally taken as 50%) in the initial stiffness, arbitrarily defined as the stiffness at the 50th cycle of the test, as there is no well defined fracture of the specimen (Baburamani 1999).

In the controlled stress mode of loading, the force amplitude is maintained at the same level as the initial force. As a result of repetitive application of this force, the displacement amplitude increases until it reaches twice the amplitude, when the flexural stiffness is reduced to half the initial flexural stiffness, which constitutes failure (Baburamani 1999). It should be highlighted that the fatigue life using a controlled strain can be up to ten times as long as the life in a test using controlled stress when starting from the same initial strain level (di Benedetto and de La Roache 1998).

The fatigue-modulus relationship is particularly important for thin asphalt pavements over inbound base material. In this situation, the relatively high modulus of the asphalt may have very little impact on the overall pavement stiffness, or on the level of tensile strain in the asphalt. The asphalt layer functions essentially in strain (displacement) control mode, rather than stress (force) control mode, and may be subject to a high strain level. Effectively, the strain level in the asphalt layer cannot be reduced by using a stiffer asphalt mix. Therefore, a potential improvement in fatigue resistance of the asphalt can be very significant (Kadar and Donald 1994).

Some debate continues as to the relationship between in-situ asphalt thickness and the appropriate test method. For typical SMA applications as a thin surfacing layer, the controlled strain mode of loading is appropriate (di Benedetto and de La Roche 1998, Baburamani 1999, APRG 1997a). The controlled strain method is generally used in Australia since the majority of asphalt surfacings are relatively thin (APRG 1997a).

4.4.4.3 Temperature

Temperature is one of the most significant factors affecting the fatigue of asphalt (APRG 1997a). Asphalt is a viscoelastic material which means that its stiffness properties are dependent on temperature and rate of loading, the stiffness of the mix influences the fatigue life of the asphalt, therefore the effect can be considered in terms of the influence of temperature on mix stiffness (Baburamani 1999). For the controlled strain testing, an increase in test temperature will reduce the stiffness and increase the fatigue life. Therefore for fatigue life, it is the lower pavement temperatures that are of interest.

The standard test conditions for Australia are 20°C (AUSTROADS AST03:1999), however other test temperatures (5°C to 20°C) can be used to investigate the effect of temperature on fatigue life (APRG 1997a). A temperature of 20°C provides a lower bound on Weighted Mean Annual Pavement Temperature (WMAPT) for the majority of Australia (AUSTROADS 1992b). All fatigue testing for this thesis was undertaken at the standard temperature of 20°C.

4.4.4.4 Australian Fatigue Testing Method

Wonson and Bethune (2000) give an overview of the recent developments in fatigue testing equipment in Australia. In early 1994, the Fatigue project group of the Australian National Asphalt Research Committee (NARC) adopted the SHRP M009 test method as the basis of a draft Australian Standard Test Method (Sonadinos 1994, NARC 1995). The test method is currently the AUSTROADS Provisional Method AST03:1999, "Fatigue Life of Compacted Bituminous Mixes subject To Repeated Flexural Bending".

A Four Point Bending Fatigue Test Apparatus, such as that manufactured by the Australian company Industrial Process Controls (IPC), is the preferred test in Australia (ARPG 1997a). Due to the relative cheapness of the Four Point Bending Fatigue Test Apparatus, the Trapezoidal Fatigue Test Apparatus has not been adopted for regular fatigue testing (Bullen et al 1996, Wonson and Bethune 2000).

4.4.4.5 Haversine Loading Conditions

In the Australian procedure (AUSTROADS AST03:1999), testing is carried out under controlled strain at 20°C with continuous haversine loading at 10 Hz. Strain level is user-specified and is generally in the range of 100 $\mu\varepsilon$ to 1,000 $\mu\varepsilon$. Testing is normally continued until the mix stiffness is reduced to half of its initial (50^{th} cycle) value (APRG

1997a).

Alford et al (1997) used an auxiliary data acquisition system to measure the actual induced strains on the extreme fibres of the test specimens. They found that the actual applied force, to maintain a required constant displacement from the origin, transforms from haversine to sinusoidal within the first 5 load cycles, and then remains sinusoidal for the duration of the test. Pronk (1996) in detailed mathematically investigation into the theory of the four point dynamic bending test demonstrated that due to phase lag between the applied force and the resultant displacement, a haversine force distribution cannot be achieved by applying simple point load forces at the two inner clamps.

In a displacement controlled test, permanent deformations (creep) lead to a new equilibrium condition in which the stresses and strains are fully sinusoidal. According to the manufacturer (IPC), tests had shown that the force signal in a haversine displacement controlled mode changed to a full sine signal within the first few cycles, causing them to use the same equations to determine the stress, independent of the kind of displacement signal selected (King 1998). It appears that this equilibrium is reached so fast that the test is very similar to a displacement controlled test using a sine loading (Pronk and Erkens 2001).

It can be concluded that whilst the displacement is haversine in shape, the applied load is sinusoidal. Because the IPC software uses peak-peak values for stress and strain, the values reported by the software are consistent with the actual stress state in the test specimens.

4.5 Summary

The mechanistic design of road pavements requires elastic properties and a performance model as the inputs to the design process. The model to be used for the assessment of pavement performance has been identified as the AUSTROADS (1992a) method. The material performance model for fatigue has been identified as fatigue strain method.

Because bitumen and hence asphalt is a viscoelastic material, properties of which are dependant on the duration of loading, some of the concepts of vehicle loading of pavements have been reviewed. The time of loading becomes important when determining test loads in the laboratory and relating these to pavement performance.

The components that make up asphalt mixtures include the binder, aggregates, filler and fibres. Polymer modified binders and the reasons for their use have been reviewed. The effects of the polymers on the performance of the bitumen and the resultant change in the properties of the asphalt have been highlighted. The need to use high quality aggregate for the manufacture of stone mastic asphalt has been demonstrated.

The raw materials need to be combined in the correct proportions to give the asphalt mixture its required properties. The traditional methods of asphalt mix design such as Marshall, Hveem and Hubbard-Field are empirical methods where the limits are set based upon prior experience with the type of mix. The methods cannot be used in situations where new mix types, materials and loadings are introduced. The empirical tests only permit a generic assessment of the performance of the mix and do not permit eh determination of the intrinsic properties of the materials. Their manner of stressing specimens is very different than the ways in which the asphalt is stressed in the pavement.

CHAPTER 5 – ASPHALT MANUFACTURE, STORAGE, TRANSPORT AND APPLICATION

5.1 General Concepts of Manufacture.

5.1.1 Production Temperatures.

These will vary with placing conditions and materials being used. Generally, production temperatures will be in the range of 150-165°C. Exceeding this range would increase the susceptibility of the mixture to binder drainage. Mix production temperatures of up to 180°C have been noted for modified binders and temperature of up to 170-175°C for non-modified binders. Higher temperatures increase the risk of binder drainage and binder effects should be checked for the maximum production temperatures proposed.

Cellulose fibers can be damaged by high temperature and it is important that they do not come in contact with aggregates or drum mix gases at a temperature greater than 200°C. Such restrictions do not apply to mineral fibers such as rock wool and glass fiber.

With mixing times, the addition of fibers to SMA mixes generally requires an increase in mixing time to ensure that the fiber is adequately dispersed and the entire product uniformly mixed. In batch plants an increase in both dry and wet mix cycles of 5-15 seconds may be required. In drum mix plants, the bitumen injection line may need to be relocated when palletized fibers are used to allow for complete mixing of the pellets before adding bitumen. Drum mix mixing times may also need to be increased by reducing production rate or changes to mixing configuration.

In all cases, the effectiveness of mixing should be monitored by visual inspection to ensure the absence of clumps of fibers or pellets in the mixture and sufficient coating of aggregate particles. If necessary, wet times should be increased or any other changes made to improve mix uniformity.

5.1.2 Storage

SMA mixes should not be stored for extended periods of time at elevated temperatures. This could result in unnecessary and detrimental binder drainage. In most cases, storage should not exceed 2-3 hours, and never stored overnight. As a rule, and as in the case of other types of asphalts, SMA mixtures should not be intermediately stored in the loading silo for long periods; otherwise damaging changes in the binder may occur. The loading areas in the transport vehicle must be clean. With a suitable anti-sticking agent being used in the trays, with diesel not being permitted.

5.2 Transport

In addition to the manufacture of the bituminous asphalt mixture in the mixing plant, special significance is attached to the transport of the material to the point of application; normally the bituminous asphalt mixture is transported in a heavy goods vehicle covered with tarpaulins (double-sheeted) and/or in a heat-insulated vehicle.

Covering the hot asphalt mixture should prevent damage to the bitumen as a result of oxidation due to the effect of the oxygen in the airflow during transportation, because otherwise there is the possibility of a hardening of the bitumen equivalent to up to two binder type gradings. This is then associated with a negative effect on the cohesive behavior of the bitumen on the mineral material. Furthermore, rapid cooling of the bitumen asphalt mixture should also be avoided, particularly in unfavorable weather and long transportation times, as should also the ingress of water into the hot mixture. In fact, in city road construction (mainly capital city), where relatively low quantities of asphalt are involved and work progresses more slowly, the use of heated HGV containers and insulated trailers with horizontal belt conveyors have proved to be particularly advantageous.

It is a mistake to believe that the cooling of the bitumen asphalt mixture over long transportation routes can be compensated for by the use of excessive mixing temperatures. Obviously in North Queensland, the climatic condition is such that road transport temperatures are always high, but covering is still mandatory to achieve the extended travel distances. SMA can only be carted a maximum distance of 2.5 hours to stay in the approved specification. The tarps help to extend the time if there is extended delays on site as well.

Haul times for SMA should be as short as possible. Increased temperature plus vibration from vehicles can result in excessive binder drainage. The high binder content may cause SMA mixtures to adhere to truck bodies to a greater extent than conventional asphalt materials. Particular attention must be paid to cleanliness of truck bodies and proper use of release agents. Even in summer, the transport vehicle must be covered with windproof tarpaulins to avoid cooling off of the mixture and damaging oxidation of the binder as a result of contact with the oxygen in the air.

5.3 Application

5.3.1 Combined Procedures

In the Main Roads Standards manual, in addition to general aspects, the Department describes the composition of the bituminous mixture, its manufacture, transportation and the application and compaction of SMA. The manual is a useful guide for the client, the asphalt producer and the contractor applying the product.

When preparing the qualification test, voids content of 3.5 Vol.-% should be targeted in construction classes SV and I. In all other cases, and when using a PmB, a value of appox, 3.0 Vol-% is required. The total coarse aggregate content has been reduced from 75 to 73% by mass. This contributes to a better homogeneity in the mixture.

The following points are given for application and compaction:

- The temperature of the mixture on the finisher hopper should be as uniform as possible, i.e. no cold accumulations of the mixture in corners and crevices.

- The road finisher that is used should be adjusted to the speed of application, such that an appropriate level of precompaction is achieved, i.e. it should not be too high. Basically the rolling operation should follow soon after the finisher.

- A minimum of two rollers is required for each application track.

- The compaction should be achieved with heavy tandem or three-wheel rollers (service weight>9t).

- Vibration compaction should only be carried out at sufficiently high mix temperatures and after static compression.

- Vibration should not be used if the later temperature is below 100°C. As a rule, vibration should not be used in the case of a rigid foundation (e.g. concrete) and for course thickness of less than 2cm, since this can lead to loosening of the foundation and break up of the aggregates.

- Rubber wheel rollers are ineffective for the compaction of SMA. They are contra-productive under certain circumstances and are no longer used.

- Any additional manual operations (hand-laying) with SMA should be carried out quickly and without delay and, if possible, at the same time as the application with the finisher. The roller compaction must be undertaken without any delay after the application. A lack of precompaction by the finisher should be accounted for in the thickness of the application.

- From the German experiences, they have the following notes to be observed when treating the paved surface:

- In order to increase the initial grip characteristics the precautions referred to in ZTV Asphalt-Stub 01 [4], similar to our SMA 10-14mm should be taken account of in the detailed estimates.

- The quantity (of grot) for spreading is 1 to 2kg/m²; in addition to the 1/3mm aggregate size, a dedusted / light bitumen coated crushed sand 0.25/2mm has also been proven in use. If possible, 2/5mm stone chippings should not be used due to the higher noise emission. Amazingly enough, in Australia, and definitely in North Queensland, this is not performed, and probably seen as a specification past our requirements

- The material to be spread can be applied wither directly behind the finisher beam or between the first rollers, but in any case it must be applied to the still adequately hot and bondable surface and rolled in. A mechanical method of spreading should be employed to obtain an even surface appearance.

- After the application, the compacting and subsequent treatment a minimum period of 24 hours should be allowed, if possible, to allow the surface course to cool before the road is released for use by traffic.

5.3.2 SMA Design.

Queensland and Victoria State Road Authorities both have issued specifications for the manufacture of SMA materials using nominal sizes 7, 10 and 14mm. The Australian Asphalt Pavement Association has produced SMA Asphalt Design and Application Guide, AUSTROADS has published APRG Report No.18 and Australian Standards has published AS2150-1995 Hot Mix Asphalt, which covers SMA mixes in sizes 7, 10 and

14mm.

Australian practice in SMA design is based on the use of coarse crushed aggregate particles and mastic consisting of fine crushed aggregates, natural or crushed sands, filler, fiber and binder with a gap graded particle size distribution. The coarse aggregate particles give SMA its resistance to deformation by the aggregate skeleton created, achieving a stone on stone contact. The mastic of fine aggregate, filler, fiber and binder fills the voids between particles to enhance the SMA's durability and resistance to water susceptibility.

The Department in North Queensland is instrumental with regards to adapting the designs to the local conditions. The district produced an SMA 12 mix with varying mix properties but predominately with a 12 mm single sized aggregate. Design grading and other details are in the Appendix H.

5.3.3 Production Specifics with Aggregates and Fillers.

The German specification requires coarse aggregates to be of the highest quality crushed aggregate available with cubical shape, high durability and resistance to polishing. Aggregate with elongated or flat partial shape should be avoided. Typical coarse aggregate types used in Australia are basalts, granites and hornfels. The flakiness index for the coarse aggregate varies from 20-35% depending on the State or Government Authority issuing the specification. To adhere closer to the German experience the flakiness index for coarse aggregates should be no more than 20%. The aggregate degradation factor and polished stone values should be increased so only high quality aggregates are used in the manufacture of SMA as all aggregates sources are not suitable for the manufacture of SMA. In Germany, fine aggregates are required to be of the highest quality crushed aggregates, but in Australia natural sands are allowed which may adversely affect the mix stability.

Plants may require modification to handle the large proportion of added filler. Both filler feeding and weighing systems may need upgrading to handle the larger quantities and deliver the required amount without restricting production rates.

In drum mix plants it is important that the filler is captured by the binder and aggregates as soon as it is added to the mixture. This is best achieved by introducing filler through a line that is placed next to the bitumen line so that the filler is coated with bitumen before it is exposed to high velocity gas flow through the drum. This keeps the filler in the mixture rather than being lost into the dust collection system.

Where filler must be introduced via the belt feed, it should be done by placing the filler on the belt before any other aggregate component. In this manner, the filler is protected to some degree by the coarse aggregate as the aggregate stream enters the drum, although the potential loss of filler is much greater using this procedure than that recommended above.

Once again, in the North Queensland region there are only a limited number of quarries, so the source rock is basically fixed. PAFV (aggregate polishing values) numbers are an issue, but basically the main problem is achieving a cubicle stone for the grading of the aggregate that the quarries produce. The other issue is the variability in gradings of the aggregate with a large standard deviation.

5.3.4 SMA Laying.

Weather Conditions: in order to achieve proper placement and compaction, placing of SMA mixtures in cold or inclement weather should be avoided. SMA should not be placed to pavement temperatures below 10°C, particularly where polymer modified binders are used. The decision to place SMA will also depend on wind conditions, pavement temperature, thickness being placed and equipment and procedures to be used in placing and compacting the mix.

Surface Preparation: the surface is generally the same as for conventional asphalt mixes. Loss of shape or depressions in existing pavements should be milled or filled using a regulating layer and any distressed areas properly repaired. While SMA has shown superior performance, it cannot be expected to perform as desired when it is used to cover up existing pavement problems. All surfaces should be tack coated prior to placing SMA using materials and application rates appropriate to conventional asphalt construction.

Spreading: normal good practice should be followed. Attention should be paid to such factors as maintaining a steady forward speed of the paver and constant material flow through the paver so that a uniform head of material is maintained ahead of the screed with feed conveyors and augers operating nearly continuously. As SMA mixes are stickier and less workable than dense graded mixes, hand work should be avoided or kept to a minimum. The difference between the loose layer thickness spread by the paver and the compacted layer thickness of SMA mixed is generally less than that of dense graded mixes and must be taken into account in determining and setting paver thickness controls for spreading.

Compaction: the procedures require some variation to that used for conventional mixes. The preferred method of compaction of SMA is with heavy, non-vibrating, steel wheel rollers. As very few asphalt contractors have such compaction equipment it is often necessary to use vibrating steel wheel rollers. In such cases, breakdown rolling should be done with one or two passes in non-vibrating mode before using one or two vibrating passes. Care must be taken to avoid drawing binder to the surface of the SMA by excessive vibration, and to avoid fracturing of coarse aggregate. Generally, only low frequency vibration should be used. Compaction procedures must be monitored and modified if required.

Breakdown rolling should begin immediately behind the paver and the roller must stay close behind the paver at all times. If the rolling operation gets behind, placement should slow down until the rollers catch up with the paver.

Multi tyred rolling is not recommended for SMA, and the primary reason is to avoid drawing binder to the surface and flushing of binder. Heavy trafficking of freshly placed SMA, while it is still hot, may also have a similar effect. Pick-up of the binder-rich SMA mortar can be a further difficulty with multi tyred rollers and traffic on hot surfaces. The multi-tyred rollers can also reduce the skid resistance of the mixture by closing up the surface texture. A high standard of field compacted density of SMA is desirable for good performance so that optimization of compaction procedures is an important element of placing. The standard of compacted density for SMA mixes should be no less than that adopted for conventional dense graded mixes.

The compaction process as applied to SMA in Australia differs from the compaction practice used in Germany. As stated above, compaction is commenced immediately behind the paver by heavy steel drum rollers with no vibration used. Despite the known problems, vibratory compaction and multi-tyred rollers have been used on occasions to achieve field compaction. Vibration should be avoided to achieve compaction as degradation of coarse aggregates will occur and induce bleeding of the binder to the surface. Typical air voids of laid SMA surfacing are generally much higher than in Germany where air voids are typically 3.0 -3.5%. SMA layed in the early nineties were recorded to have field air voids in the range of 6-9% although they are now in the range of 5-7%. In Queensland, SMA mixtures have produced field voids in the range of 6-11%. The opening up to traffic of SMA mixtures has seen mat (roadway) temperatures in the range of 80-110°C, compared with common German practice of 40 to 60°C.

Static compaction by heavy duty rollers with a weight of 8-12 tonnes close to the paver ensures a high compaction rate (>97% recommended). As stated previously, one train of thought to increase the initial skid resistance of the surface is to spread sand/chippings fraction e.g.1/3mm at 0.5-1.0kg/sqm or aggregates e.g. 2/5mm at 1.0-2.0 kg/sqm on the hot surface.

Opening to Traffic: SMA wearing surfaces should not be opened to traffic until the surface temperature falls below about 40°C to avoid drawing of the binder to the surface by the initial traffic.

The initial skid resistance of SMA may be relatively low until the binder film is worn from the surface by the traffic. A technique that has been used in some countries is to lightly grit the surface with a coarse sand, or small sized (i.e. 5mm) crushed aggregate, to assist in wearing the binder from the surface in situations where skid resistance is of prime importance. Alternatively, speed restrictions may be applies for a short period of time.

In North Queensland, problems are associated with this pavement temperature being reached in the time required to open the work to traffic. Contractors request the approval to water cool the layers, which is not desired but sometimes necessary. Early life rutting and deformation is evident on some projects which were opened to traffic too early, although other factors require assembly in these situations.

CHAPTER 6 – STONE MASTIC ASPHALT RESEARCH DEVELOPMENT AND SPECIFICATIONS

6.1 Introduction

The major International, Australian and Queensland specifications for Stone Mastic Asphalt (SMA) and in particular their requirements for the aggregate gradings, bitumen and voids contents are reviewed. Whilst the design of SMA is still largely "recipe" based, there is growing interest in more fundamental design methods that ensure that a stone skeleton with stone-on-stone contact is achieved. The following is a brief summary of the principal requirements of a range of typical SMA specifications and/or design guides. Brief details of a number of other European specifications are given in EAPA (1998).

6.2 Review of Specification for SMA

6.2.1 Germany (EAPA 1998, Loveday and Bellin 1998)

In German guidelines, stabilizing additives are specified as 0.3-1.15% organic or mineral fibre, silica or polymer. For heavy traffic situations, the binder grade is specified as 65(pen) or PMB45 and 80(pen) for other applications. The pen is for penetration which is a standard test at 25 °C. Three grading classifications are used based on the sieve size that retains 10% or less of the material – for the 0/11 mix this is the 11.2mm sieve, 0/8 uses the 8.0mm sieve and the 5mm sieve for the 0/5 mixes. Bitumen content for the 0-11S mix is a minimum of 7.0% and a minimum of 7.2% for a 0-5 mix. The 'S' signifies mixes for heavy traffic applications and these mixes must include only crushed sand.

Marshall Compaction is specified with a voids content range 3.0-4.0% for the 0-11S and 0-8S mixes and 2.0-4.0% for the 0-8 and 0-5 mixes.

6.2.2 Draft UK Specification (Nicholls 1998a)

The draft UK specification describes whether a modified binder or, alternatively, bitumen with a stabilizing additive to be used is a choice of the Contractor otherwise it will be in an Appendix. Unmodified bitumen's are at to have a nominal penetration of 50 or 100 and at least 0.3% (by mass of the total mix) of stabilizing additive (Cellulose, mineral or other suitable fibre) is required. For a modified binder, the base bitumen before modifications is to have a nominal penetration of 50 or 100 or 200. For a 14mm nominal size SMA, the binder content is specified as 6.5-7.5% (by mass) compared to 6.5-7.0% (by mass) for the 10mm nominal size SMA.

Laboratory specimens are manufactured using 50 blow/face Marshall Compaction. At the target composition, the air voids content is specified to be within the range 2-4% where the maximum density is determined by the ‘Rice’ method and the bulk density by an uncoated water immersion method.

6.2.3 Draft European Specification (prEN 13108-5:2000)

This specification could be described as a compromise document. As well as providing two basic sets for the grading and binder content referred to “Basic set plus set 1” and “Basic set plus set 2” which cannot be combined, there is also provision for a National Annexure to give grading curves with a clear indication of the required values. The sizes included in “Basic set plus set 1” are SMA-4, SMA-8, SMA8E, SMA-11E, SMA-16 and SMA-22. “Basic set plus set 2” includes SMA-4, SMA-6, SMA-6E, SMA-10E, SMA-14 and SMA-20. The letter “E” refers to an additional requirement that only crushed fine aggregate is to be used. Laboratory specimens are to be compacted by impact compactor (Marshall 2x50 blows), gyratory compactor (200 gyrations) or vibratory compactor. Two methods of determining maximum density are allowed however only one hydro-static method for determining bulk density is allowed.

A single value of the voids content is specified for each of the seven traffic categories which range from Category 1 for the highest traffic to Category VII for the lowest traffic. The voids contents range from 2.5% for Category 1 to 5.5% for Category Vii with a 0.5% increase in voids content per traffic category. The binder content requirements are limited to minimum values. For a given grading, there are two minimum bitumen contents specified. These are based on the target voids content being ≤ 4 % and > 4%. The target voids content is determined from the appropriate traffic category. A summary of specified values is given in Table 6.1.

"BASIC SET PLUS SET 1"							
MIX	**TARGET VOIDS**	**SMA 4**	**SMA 8**	**SMA 8E**	**SMA 11E**	**SMA 16**	**SMA 22**
Binder Content (%) Min	≤ 4%	7.2	7.0	7.0	6.5	6.0	5.7
	> 4%	6.7	6.5	6.5	6.0	5.5	5.2
"BASIC SET PLUS SET 2"							
MIX	**TARGET VOIDS**	**SMA 4**	**SMA 6**	**SMA 6E**	**SMA 10E**	**SMA 14**	**SMA 20**
Binder Content (%) Min	≤ 4%	7.2	7.0	7.0	6.6	6.2	5.9
	> 4%	6.7	6.5	6.5	6.1	5.7	5.4

Table 6.1 - Minimum bitumen contents (%) for Draft European Standard SMA Mixtures (After prEN 13108-5:2000)

The binder contents in Table 6.1 are based on an aggregate density of 2,650 kg/m³ and a correction factor is applied which is the ratio of 2,650 kg/m³ divided by the aggregate density in kg/m³. Effectively, this is a conversion for volume of bitumen since the binder content is specified in percent based mass.

6.2.4 Draft AASHTO Specification (NCAT 1998c)

The draft AASHTO Specification allows laboratory specimens to be prepared using either Marshall Hammer compaction or the Superpave™ Gyratory Compactor. For both compaction methods the air voids content is specified at exactly 4% with the note that for low traffic volume roadways or colder climates, air voids contents less than 4.0 percent can be used, but should not be les than 3.0 percent. There is a minimum VMA requirement of 17%. The concept of Voids in the Coarse Aggregate (VCA) is introduced with the requirement that the VCA for the mix (VCA_{mix}) must be less than the VCA of the “dry rodded” coarse aggregate (VCA_{DRC}).

Minimum binder contents are specified based on the combined aggregate bulk specific density. Minimum binder content varies from 6.8% for aggregate with a density of 2,400 kg/m³ to 5.5% for aggregate with a density of 3,000 kg/m³. Performance grade binder appropriate for the climate and traffic loading conditions at the site are to be used.

6.2.5 Australia

6.2.5.1 APRG Report No. 18 (APRG 1997a)

This report was published in 1997 and reflects the state of the knowledge in Australia at that time. The mix design criteria were to be achieved after 80 cycles of a gyratory compactor. Three mix sizes are listed being Size 7mm, Size 10mm and Size 14mm with the voids content range of 3-5% specified for all mixes. The specified bitumen content ranges are 6.5-7.5% (by mass of mix) for Size 7mm, 6.3-7.0% for Size 10mm and 6.0-7.0% for Size 14mm mixes. There are minimum VMA requirements of 19%, 18% and 17% for Size 7mm, Size 10mm and Size 14mm respectively.

6.2.5.2 Stone Mastic Asphalt – Design & Application Guide (AAPA 2000a)

The recipe approach to SMA design, based on the AAPA Design Guide (AAPA 2000a) is now the preferred method of use in Australia (Oliver 1999). The same three mix sizes are specified as were included in APRG (1997a) however the voids content range has been narrowed to 3.5-4.5% and the mixes have a coarser grading. Minimum VMA requirements are the same as APRG (1997a). Typical design binder content ranges are 6.0-7.0% (by mass of mix) for Size 7mm, 6.0-7.0% for Size 10mm and 5.6-6.8% for Size 14mm mixes which are generally slightly less than those in AGRG (1997a).

The guide (AAPA 2000a) suggest that laboratory compaction for volumetric testing is usually carried out using 80 cycles of gyratory compaction except particularly heavy duty applications where 120 cycle may be used. Where gyratory compaction is not readily available, mixes may be compacted using 50 blows of Marshall Compaction. Some specifications also set minimum air voids for refusal density taken as 350 gyratory cycles.

6.2.5.3 Queensland Department of Main Roads MRS11.33

When SMA was first being layed in North Queensland, the current version of MRS 11.33 was dated Interim 5/97. SM10 SM14 mixes were specified with samples to be compacted with 50 Marshall Blows per face with a minimum stability of 8kN and minimum flow of 2mm. The specified voids content range for both mixes was 3-7% with bulk density to be measured by either the wax coating (Q306A) or silicone coating (Q306C) methods. Minimum VMA requirements of 14% for SM10 and 13% for SM14 are specified. The binder content of the design mix was specified as between 5.9% and 6.9% (by mass).

The 12/99 version of MRS11.33 introduced some changes to the requirements. Whilst

50 blow/face Marshall Compaction was retained the minimum stability was reduced to 6 kN and the minimum flow remained unchanged at 2mm. The reduction in Marshall Stability results in a value similar to several overseas specifications (EAPA 1998, NCAT 1998c). The Draft AASHTO Specification also includes the note that successful SMA mixtures have been designed with Marshall Stability values below 6.2 kNm therefore this requirement can be waived based on experience (NCAT 1998c).

In the 12/99 version of MRS11.33, the range for the specified voids content increased to 2-7.5% with the bulk density to be measured by the silicone coating method (Q306C). Minimum VMA requirements have changed to 15% for SM10 and 14% for SM14. A fundamental change has been that the binder content is now specified as the minimum unabsorbed binder volume rather than by mass as used in earlier editions. The specified minimum binder volumes are 13% for SM10 and 12% for SM14.

The Interim 5-97 version of MRS11.33 had a default option that if a modified binder was not specified in the addendum (where job specific options are contained) then Class 320 bitumen was to be used. The 12/99 version of MRS11.33 removes this default and the binder type must be specified in the addendum.

The current MRS 11.33a and b specification are discussed throughout Chapters 7 and 9, as well as the introduction of the Northern mix - SMA12. Appendix H Provides a comparative graph of the asphalts from this specification.

Various modifications became necessary to adapt the specification to the local source materials and the production possibilities.
The main changes to the specifications were as follows:

- Density better than 94% (characteristic value);
- Less use of fly ash and hydrated lime;
- Changed grading to increase VMA; predominately 100% hydrated lime
- Implementation of 100 tonne preliminary trial sections prior to construction of local projects.

Mix approval based on production results from 100 tonne test sections;

- Use of a different PMB and less modification;
- Introduction of a free binder volume requirement.

These changes have come about through the past years of experience with SMA projects, and the continual trials with the associated testing which helps to develop the failure mechanisms.

6.3 Trends in SMA Specifications

6.3.1 Aggregate Grading

Figure 6.1 shows a comparison between the centre-line grading curves of two Australian SMA Mixes and the draft specifications of Europe, U.K. and U.S.A. Part (a) of Figure 6.1 is for SMA10 mixes whereas part (b) is for SMA14 mixes. For the international mixes, the "nearest" similar size mix has been used for the comparison. Figure 6.2 shows a comparison between the centre-line grading of various Australian SMA Mixes. Again part (a) is for SMA10 mixes and part (b) is for SMA14 mixes. The grading curves of SMA are typified by the definite "step" between the course and the fine components compared to dense graded asphalt were this is a smooth transition.

From Figure 6.1(a), it can be seen that the step generally occurs around 2.36mm for all mixes with the exception of the AAPA (2000a) mix which has the coarsest of all SMA10 gradings. The AASHTO 9.5mm grading is very similar to the DMR (Qld) MRS11.33 SMA10. Whilst the UK 10mm has a defined "step" around 6.7mm, it is the finest of all mixes at 2.36mm/ All mixes have about 10% passing the 75 µm sieve which forms the filler part of the mastic. Comparing the Australian SMA10 mixes (Figure 6.2 (a)), it is seen that the AAPA (2000a) SMA10 is the coarsest of all mixes and the most significant difference between it and the DMR (Qld) SMA10 occurs in the middle of the range of sieve sizes and particularly on the 4.75mm and 6.7mm sieves.
As shown in Figure 6.1(b) for the SMA14 mixes, the "step" in the grading typically

occurs around the 4.75mm sieve. The AASHTO 12.5mm grading is very similar to the DMR (Qld) MRS11.33 SMA14. The AAPA (2000a) SMA14 has the coarsest of all the SMA14 gradings. Comparing the Australian SMA14 gradings (Figure 6.2(b)), the differences between DMR (Qld) and AAPA (2000a) gradings are note as significant as the differences between their SMA10 gradings.

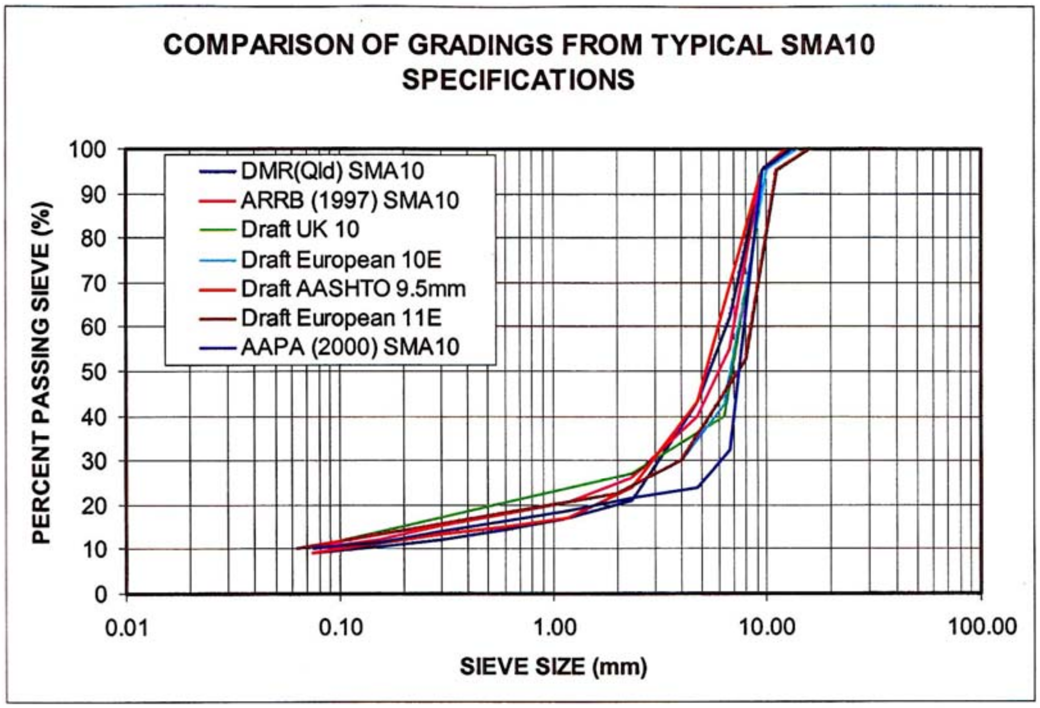

Figure 6.1 (a) – Comparison of typical International and Australian gradings for SMA 10

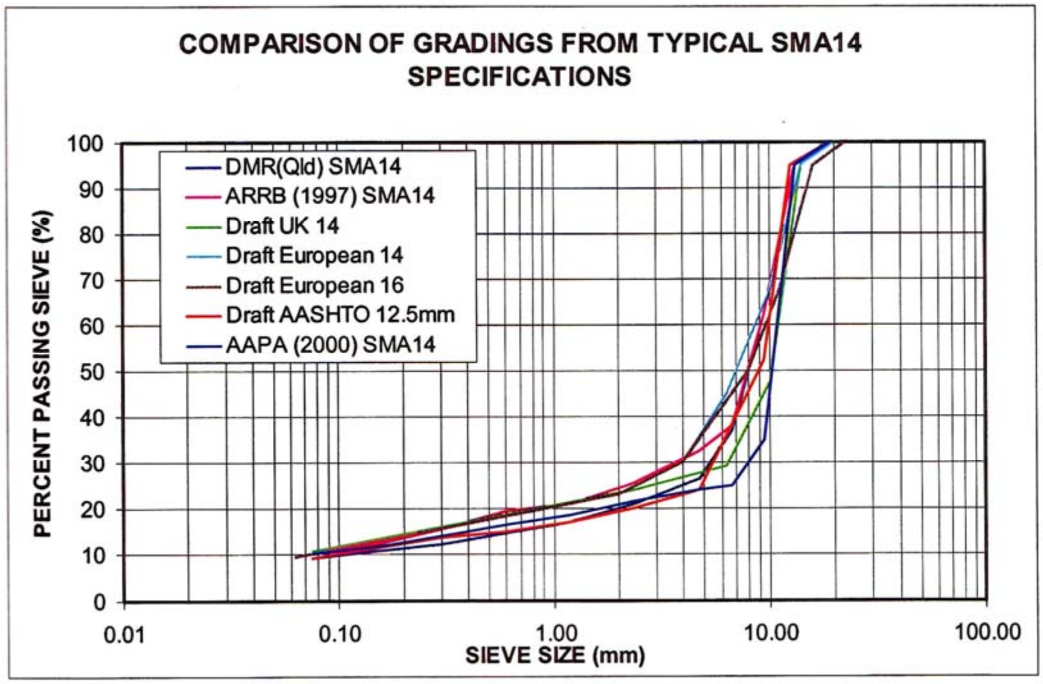

Figure 6.1 (b) – Comparison of typical International and Australian gradings for SMA 14

6.3.2 Compaction Methods

The laboratory compaction methods specified demonstrates the transition across the world in pavement design with combinations of Marshall, Gyratory and Vibratory compaction being included; with the European specification including all three methods. The specifications with Marshall Compaction use 50 blows per face. The Europeans equate this to 200 of their gyratory cycles. AAPA (2000a) quotes 80/120 gyratory cycles or 50 blow/face Marshall Compaction from which it can be conferred that there is a correlation.

6.3.3 Bitumen Content

A number of trends in specifying bitumen content can be identified such as specifying minimum binder content rather than a range with a lower and upper limit. Typically the bitumen content varies with maximum aggregate size and is a reflection of the respective volumes and densities of the aggregates and bitumen. Another trend is that of recognizing the importance of the volumetrics of the SMA mixes. This is reflected in the increasing use of binder volume either as a direct volume requirement (MRS11.33 of 12/99) or as an adjustment to binder volume based on density of the aggregate (NCAT 1998c. prEN 13108-5:2000).

The minimum binder contents specified in Australia are typically lower than those quoted for the Europe, UK and USA. This could be a reflection of the different climates especially when considering the sub-tropic climate of Queensland compared to Europe and a desire to avoid potential surface “flushing” problems.

6.3.4 Voids Content

The majority of specifications have a very tight limit on voids contents with a typical range between 2% and 4% or ± 0.5% o the specified value. The voids contents within the range 2-7.5% as specified by DMR(Qld) MRS11.33 is a significantly wider range than specified in the majority of the international specifications. Comparison of these values requires an appreciation of the compaction and testing methods used in the source specifications.

CHAPTER 7 –DESIGN CRITERIA FOR STONE MASTIC ASPHALT

7.1 Introduction

This chapter discusses the research undertaken to address the following areas of the research programme:

- The development of the relationship between Marshall and "gyropac" compaction for SMA;
- The development of the relationship between voids content determined by mensuration and silicone coating; and
- The extension of the Dilation Point Method of design for the stone skeleton of SMA.

7.2 Comparison of Compaction Methods

Figure 7.1 shows a comparison made between the voids contents for a plant produced SMA10 manufactured to DMR(Qld) MRS11.33 and compacted by varying numbers of Marshall blows/face (20, 30, 40, 50, 60, 70 and 75) and "gyropac" cycles (50, 80, 120, 180 and 350). The samples were compacted in 100mm (nom.) diameter moulds. From Figure 7.1, it can be seen that the data is separated into two distinct groups with the Marshall compacted samples generally having less than 8% voids and the gyratory compacted samples having more than 8% voids. For the samples compacted at 140°C, it can be inferred that 25 blows/face Marshall compaction produces the same voids content as 350 gyratory cycles.

Table 7.1 shoes a comparative study undertaken by the Queensland Department of Main Roads (DMR (Qld) 2001a). For the SMA10 with a variety of filler and binder combinations, it can be seen that the voids content after 350 gyratory cycles is of the same order as that obtained with 35 Marshall blows/face. For the SMA14 mix, the voids content at 350 gyratory cycles would be obtained at less than 35 Marshall blows/face. The research undertaken by Stephenson 2002 shows that laboratory prepared mixes has the same trend as the plant produced SMA12 tested for this thesis.

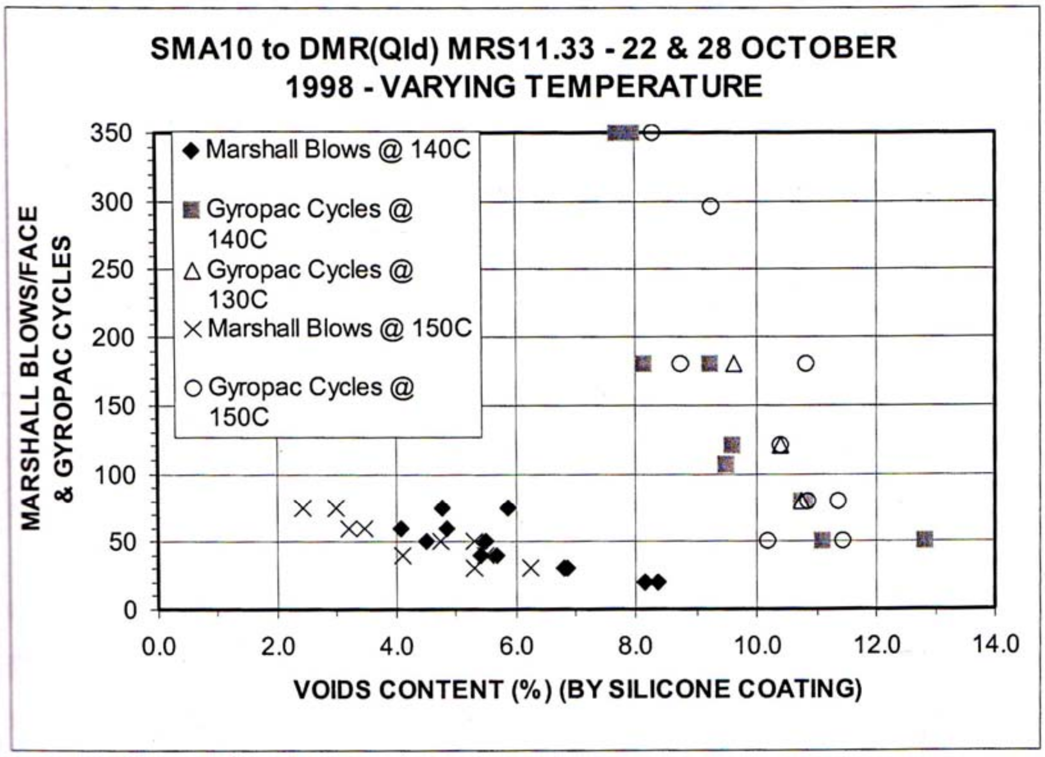

Figure 7.1 – Comparison of voids contents from Marshall and Gyropac compaction

	SM10				DG14N	SM14
	AB5 + ULTRA FINE DUST FILLER	C320 BITUMEN + ULTRA FINE DUST FILLER	C320 BITUMEN + HYDRATED LIME FILLER	C320 BITUMEN + FLYASH FILLER	C320 BITUMEN + UFD FILLER	AB5 + FLYASH FILLER
Marshall 35 Blows	5.8	5.0	7.1	4.5	5.7	4.8
Marshall 50 Blows	3.9	4.6	6.0	3.2	4.8	3.8
Marshall 75 Blows	3.5	3.7	4.2	2.0	3.4	2.8
Gyropac 80 Cycles	7.7	8.3	9.3	7.2	4.6	6.9
Gyropac 120 Cycles	6.8	6.6	8.3	6.0	3.0	6.4
Gyropac 350 Cycles	5.7	5.8	6.2	4.5	1.4	5.3

AB5 PMB contains SBS to DMR(Qld) MRS11.18 Classification A5S (AUSTROADS A15E)

Added filler is 6% by mass to give a total filler content of 10% by mass

Table 7.1 – Effect of compaction on voids content of various mix types

(After DMR (Qld) 2001a)

For the DG14N mix, 50 Marshall blows/face produced and voids content of 4.8% which is very similar to the 4.6% for the sample compacted with 80 gyratory cycles. Compaction by 75 Marshall blows/face (3.4% voids and 120 gyratory cycles (3.0%) produced voids contents of a similar magnitude.

The relationship that 50, 80 and 120 gyratory cycle was approximately equivalent to 35, 50 and 75 Marshall Blows, respectively (APRG 1997a) appears to apply to the dense graded asphalt however it is not appropriate for the SMA mixes to the DMR (Qld) MRS11.33 specification. An assumption that 35 Marshall blows/face produces a voids content similar to 350 gyratory cycles is more appropriate for this material.

The relationship between Marshall and gyratory compaction was further investigated by a grading and bitumen content analysis of compacted samples of the plant produced SMA10. The samples selected were some of those compacted to produce the data shown in Figure 7.1. All samples were prepared from the same batch of material so it would be anticipated that the grading before compaction should be identical. The gradings for all samples compacted by gyratory compaction were found to be generally within ±1% passing on the same sieve size. It can reasonably be concluded that the gyratory compaction has not altered the aggregate grading. There does however appear to be a trend of reduced bitumen content with higher number of cycles. The continued kneading action at high temperature may be causing some flushing of the binder which is absorbed by the paper inserts used on the wearing plates or left as a smear on the internal faces of the mould.

Inspection of the data in Table 7.2 for the Marshall compaction shows a trend for the percentage passing the 6.7mm and 4.75mm sieves to increase as the number of Marshall blows increase from 20 to 50 blows/face. The sample compacted with 75 blows/face (M11) does not fit this trend and it is observed that its voids content is higher than the samples compacted with 40 and 50 blows/face. A duplicate samples compacted with 75 blows/face (M12) did produce a voids content lower than the samples compacted with 40 and 50 blows/face. Although this testing is from former research and from some years ago, the points to be emphasized remain the same and the trends still apply.

PLANT PRODUCED SMA10 TO DMR(QLD) MRS11.33 - 28 OCTOBER, 1998

		Marshall Compaction Blows / Face				Gyropac Compaction Cycles			
		20	40	50	75	50	120	180	350
Sample No.		**M9**	**M5**	**M4**	**M11**	**G9**	**G12**	**G3**	**G2**
Compaction Temp		141	141	141	140	143	132	142	140
Voids - Silicone Coated		8.4%	5.7%	5.5%	5.9%*	11.1%	10.4%	9.2%	7.7%
Voids - Mensuration		13.5%	9.4%	8.6%	9.7%#	17.4%	16.2%	16.9%	14.4%
GRADING	***SPEC***	**M9**	**M5**	**M4**	**M11**	**G9**	**G12**	**G3**	**G2**
% pass 13.2mm Sieve	*100*	100	100	100	100	100	100	100	100
% pass 9.5mm Sieve	*90 to 100*	94	95	95	97	95	95	96	95
% pass 6.7mm Sieve	*54 to 70*	68	69	74 ^	68	67	67	68	66
% pass 4.75mm Sieve	*36 to 50*	40	43	46	43	38	38	38	39
% pass 2.36mm Sieve	*16 to 26*	25	27 ^	27 ^	27 ^	25	25	25	24
% pass 1.18mm Sieve	*13 to 21*	20	21	21	22 ^	20	20	21	20
% pass 600µm Sieve	*11 to 18*	15	16	16	16	15	15	16	15
% pass 300µm Sieve	*9 to 15*	11	12	12	12	11	11	11	12
% pass 150µm Sieve	*8 to 13*	8.9	10.0	9.6	9.5	9.5	9.7	9.4	9.9
% pass 75µm Sieve	*7 to 11*	6.0 ^	8.5	7.1	8.5	7.5	7.4	7.5	7.8
% Bitumen (by mass)	**5.9 to 6.9**	**5.34**	**5.27**	**5.36**	**4.92**	**5.27**	**5.34**	**4.44**	**4.34**

* Duplicate Sample M12; Silicone Voids = 4.8%
\# Duplicate Sample M12; Mensuration Voids = 7.9%

Table 7.2 – Effect of compaction on grading of plant produced SMA10 to DMR (Qld) MRS11.33

Comparison on the grading, final voids content and number of Marshall blows/face, indicates that the reduced voids contents of the Marshall samples compared to the gyratory compacted samples is due to breakdown of the larger sized aggregate as part of the Marshall compaction process. In a number of cases, the grading has been altered to the extent that it is outside the specification limits. This is particularly the case with the samples compacted with 50 Marshall blows/face which is the current Queensland specification design compaction.

Research at NCAT (1998b) indicated that aggregate hardness affects the relationship between the bulk densities of samples compacted with the Marshall hammer and the SHRP gyratory compactor. It was recommended that for SMA mixtures utilising aggregates with LA Abrasion loss values of 30%, a design number of gyrations of 70 be used. For harder aggregates (loss value less than 30%), a design number of gyrations of 100 was proposed.

Mixes designed by the Marshall method can have artificially low design voids contents which are the result of the fracturing of the larger aggregate particles during the laboratory compaction process. This particle fracture may not be reproduced under field compaction conditions resulting in excessive in situ voids contents for mixes designed by the Marshall Compaction method.

7.3 Comparison of Methods of Measure Voids Content

The testing in this section was performed mainly by Stephenson 2002 with a comparison by various organisations and individuals was undertaken between the “mensuration” and “silicone coated” voids for a range of samples including plant produced SMA10 and laboratory trial mixes. The voids were first determined by the mensuration method before the same samples were silicone coated so that a direct comparison could be made between the voids contents determined by the two methods. The water immersion (Rice) method was used to determine the maximum density. The mensuration method will include both the internal voids (voids which have no connection to the surface) and surface voids. Because the silicone coating can fill the surface voids, they are excluded from the voids measurement in the silicone coated method. Because the coating prevents the entry of water, it would be expected that relationship between mensuration and silicone coated voids would continue to rise with increasing voids contents which contrasts with the relationship between mensuration and “uncoated” water immersion voids (Oliver 2000).

Figure 7.2 shows a comparison made between the voids content measurement methods for a plant produced SMA10 manufactured to DMR (Qld MRS11.33). The laboratory samples were compacted in 100mm diameter moulds. Those with mensuration voids content less than 12% were compacted by the Marshall hammer whilst the higher voids content samples were compacted with the Gyropac. A linear regression line has been fitted assuming that since the silicon coating will fill the surface voids, both methods will be capable or recording zero voids (i.e. y intercept = 0). It can be seen that silicone coated voids are 60% of the mensuration voids for the same sample. At high voids contents, some concern may exist as to whether the silicone is filling more than just the surface voids. The two high values above 20% void suggest that this may be occurring due to the very open surface texture of these samples.

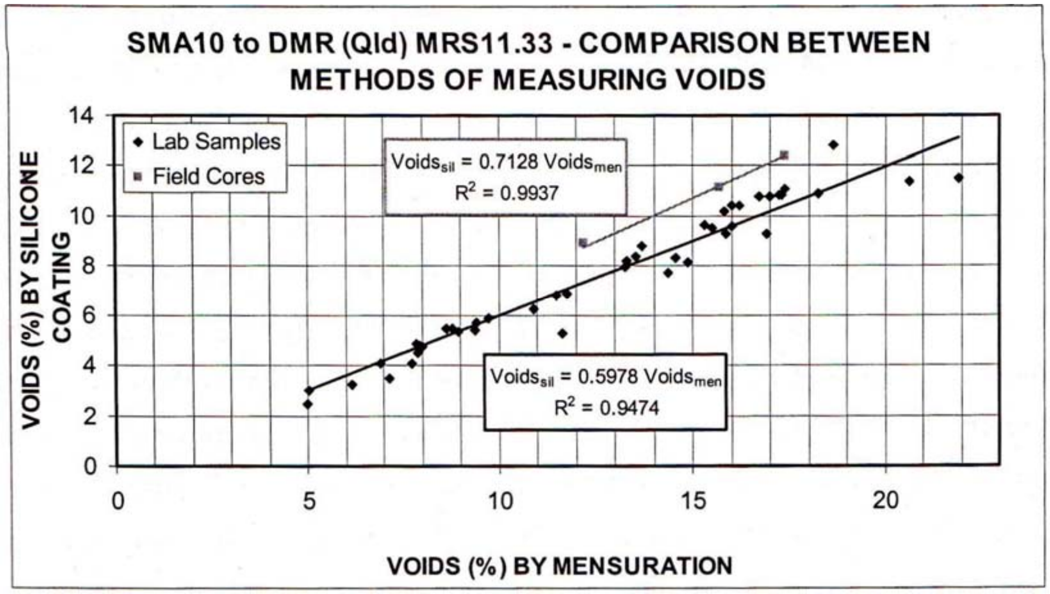

Figure 7.2 – Relationship between methods of measuring voids content for plant produced SMA10

Three cores that had been removed from the pavement where the same mix was placed were also available and these are also shown in Figure 7.2. The cores had a 150mm nom. diameter with a 30mm nom. thickness. Being cores removed from the pavement, they had smooth sides and a surface texture very different from the laboratory compacted samples. It can be seen that silicone coated voids are 71% of the mensuration voids for the same cored sample. The difference between voids measurement methods for 100mm (60%) and 150mm (71%) samples is of a similar magnitude to that reported for the sample size effect for Dense Graded Asphalt (Oliver 2000). Due to the different thickness of the cores and the laboratory samples, the different relationships may also be explained due to the surface side effect of the mould (Lorio et al 1999).

This further highlights the need to understand which test method is being used when comparing results and the limitations of any test method. Such variables are just one of the difficulties in technology transfer between countries. Figure 5.2 provides a basis for comparing mensuration voids and silicone coated voids.

7.4 Trials

7.4.1 History in North Queensland

In 1997 Main Roads approval was given to one of the districts for a submitted SM10 mix design followed by further SM14 and SM10 mix designs being approved in 1998 for both of the districts suppliers. These SMA mixes were approved with a range of different binders including Class 320 bitumen, Multigrade and A5S Polymer.

The district basically has recognized the long term fatigue benefits of SMA on the road network when compared against other asphalt types. Although still in the period of service having not reached a full pavement life history in some sections, SMA is generally performing well.

The SMA trials are located in various areas of North Queensland as per the table in Appendix D. Due to the number of trial sections within each of the locations; a priority list has been developed on the basis of varying mix types and varying visual performance criteria.

SMA trials conducted in 2004 by Northern District Townsville revealed workability problems associated with "overliming" the mix. There was difficulty found by the contractor in placement of the SMA14 mix design where the added filler component comprised hydrated lime only. A revised mix design was then trialed with reduced hydrated lime content (50:50 Fly Ash/ Hydrated Lime) which provided improved working characteristics during paving and compaction operations.

Trials were also conducted in Peninsula District in 2004 aimed at improving workability of the mix and reducing permeability. Trials were conducted on Brinsmead-Kamerunga

Road where the hydrated lime content filler content in the SM12 mix design was significantly reduced. Considerations were made for the configuration of asphalt plant used in the trial as only one type of filler could be stored at a time. To overcome this issue blended filler (50:50 Fly Ash/ Hydrated Lime) was supplied for the trial.

A decision was made by Mr. Dave Hamilton of DMR based on research of overseas experiences with SMA Mix designs and other asphalt mixes to substitute the existing asphalt filler (fly ash) with hydrated lime filler. This requirement was written into the Peninsula Districts supplementary specifications for contracts containing MRS 11.33 SMA mixes in 1998.

With the current configurations of both asphalt plants in the district having the capacity to store only one type of filler a time there is an added benefit to all asphalt mixes produced where the filler used in hydrated lime. Filler is used to control void content, but much research into varying types has proved that products such as lime create a hydrating effect to set up and help with density of the mix.

As a result of these trials there was interest generated within RS&E (Regional Systems and Engineering) related as to the effect of different fillers on the mastic component within SMA. Consequently investigation and research of filler effects upon the mastic followed. The research findings confirmed the trial outcomes with respect to "overliming" of the SMA mix causing problems with workability and over stiffening the mastic component.

The design speed adopted years before a design was 80 km/h. In the circumstances, having regard to likely driver behavior on overtaking lanes, the predominate speed is limited due to geometry. Recent testing on Skid Resistance on various sections of the trial sites also shows values of 160-180mm, which converts to a Skid Resistance factor of 2.5-3.5mm. The minimum parameter for intervention is 0.6mm so the sections are clearly performing well and certainly past the initial stage of early life skid resistance.

7.4.2 Case Study - Trial Site

There have been numerous trials within the North Queensland area, with varying mix types, project quantities and constituent proportions. One of the main sections where trials have been conducted is the Gillies Range. This section was divided into 8 similar sub-sections where the bitumen types were varied as were the contractors.

Prior to 1994, the Gillies range was predominately a sealed surface with locations where radius curves were acceptable at reduced speeds, but would be considered too tight for modern highway engineering practice. In 1994, the road was widened in sections by resurfacing with asphalt and widening the shoulders. New works started to provide overtaking lanes in each direction. To achieve this, there were adjustments to horizontal and vertical curves.

Skid resistance is related to tyre contact area and road surface friction characteristics. Typically, both open graded asphalt and stone mastic asphalt provide a better surface texture as regards drainage, splash and spray, than does the finer surfaced dense graded asphalt. On the other hand they provide less real tyre contact area than dense graded asphalt does. However, open graded asphalt is more forgiving in the wet. It allows surface water to be forced through the pavement surface under the weight of the tyre and thereby allows better tyre contact with the road surface in the wet, compared to these other asphalt surface types.

A full list of the trial sites is included in Appendix D. Apart from the Gillies site there are many areas where SMA has been trialed with different binders, fines ratios, and aggregates. The sites all have varying underlying layers from straight C170 seals to open graded asphalt.

The actual skid resistance characteristics of the previous sealed surface sections at the trial sites are not known. SMA is generally an appropriate asphalt for good skid resistance, even compared to open graded asphalt. However, given the open graded asphalt's inherent ability to promote contact with a vehicle's tyre when excess water is present on the road surface, it is considered that it provides a greater degree of surface friction, when water is present on the road surface, than the SMA surface is able to. The problem with open-graded asphalt is its life span and problems associated with unraveling on tight geometry. In short, the combination of the curve's reduced super-elevation on the range section and poor surface drainage required a greater reliance on side friction in wet weather. SMA was therefore the preferred asphalt and a good section for a trial.

7.4.3 Mix Volume Ratio

The mix volume ratio is one of the new standard SMA tests. The method to perform the test was developed in-house using information contained within technical references. It describes the procedure for calculation of the mix volume ratio of stone mastic asphalt. This ratio is defined as the volume of the components other than the coarse aggregate (>4.75m) within a compacted mix expressed as a proportion on the volume of air voids contained within the coarse aggregate in a dry rodded condition. The method required determination of the binder content and grading of the mix, the particle density on a dry basis of the coarse aggregate, the compacted unit mass of the coarse aggregate and the compacted density of the mix.

Basically, from raw data, the test compares the compactive density of the aggregate to the design. In straights forward terms though, the aggregate is tested as per the grading or size of the various sieves. This design being calculated with much theory, it is obvious of the aggregate is exactly on the centre line grading of the envelope, the test will pass with a ratio of 1. Any deflection to say the upper limit of 55% from the centre line 50% will give a ratio of above 1.

The chances of a centre line grading every time are slim, and the effect of this on the performance is limited. If the aggregate stays within the total envelope then it is within the design limits. If it strays then the mix is out of specification for many reasons. The test is more about ensuring compliance with another test than any direct future performance.

7.4.4 Mix Sensitivity

This test was also developed in-house using techniques evolved through internal Main Roads research investigations into laboratory tolerance mix preparation and evaluation. This method describes the procedure for the preparation and testing of asphalt tolerance mixes for evaluation of asphalt mix designs submitted for approval.

The test has two separate parts with two specific percentage criteria. Firstly we take a sample with aggregate from the fine side of the grading of high fraction, with the high side of the bitumen content. This should be maintained at 1%. The other part is a sample with the coarse side of the centre line grading or low fraction, with the low side of the bitumen content range. Both samples must also stay within the maximum permitted variation of the gradings, the second part maintaining a result of 6%.

The test basically ensures that the balance between fine and coarse aggregate to the binder maintains a balance: within a sensitivity range. Once again though, the issue with this test is still one of checking that the sample is conforming to the design. It purely checks whether the samples are maintaining a result which keeps the level within the approved limits. Testing of various samples have proved that this is a design control test. The issue is about continuing to develop design tests that reflect the in-situ asphalt product from a design perspective.

7.4.5 Field Voids

The relative voids of SMA are firstly established from the design mix approval. Developed for certain loadings and volumes, the mix has a target voids content to achieve the desired performance. In practice, the specifications are written to provide a maximum compactive effort as a percentage whilst maintaining depth and surface texture. With SMA, we usually find that the gap graded nature of the mix which promotes stone on stone contact means that the compaction is achievable, but cannot be surpassed by many percentage points.

The field voids are a very important part of the laying, testing and audits, as it ensures that many of the design mix criteria are being provided with a chance of being achieved. The voids have been mentioned previously throughout the report and this is the check that it is being kept within tolerance of the design. Section 7.5 will provide a detailed analysis of the trial sites and compaction results of field voids.

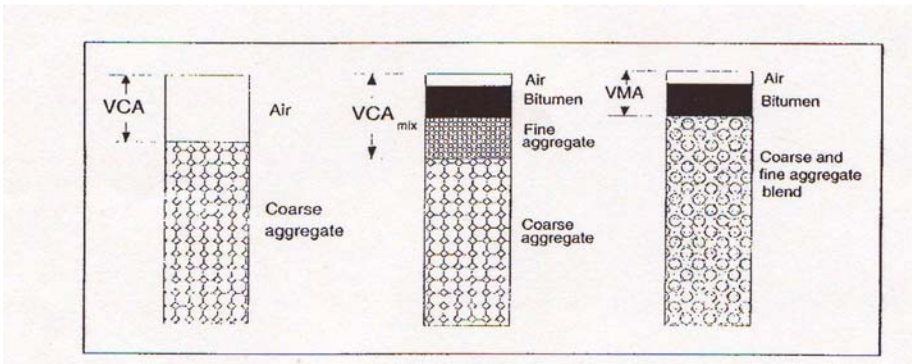

Figure 7.3 - Voids in Mix

The comparison between a design mix from 'pat samples' and the production field voids link many of the design criteria together. The testing is fairly simple to perform, and the conceptual picture of the air voids can be seen in Figure 7.3.

7.4.6 Filler Binder Ratio

This has been a very crucial test in the break through of knowledge for SMA. The filler content of the mix is set at a level not only to provide the total fines content required, but also to ensure that the quantity allows for the properties of the filler to be active. In recent times it is now an issue of calculating whether a production plant naturally produces or looses fines. The level of fines can then be adjusted in respect to the addition of fines or filler compared to the total allowable grading content of the fines fraction.

What type of filler needs to be determined in designing the mix and this has been discussed previously. Certain types of filler can have varying degrees of how absorbent they are, which affects the ability to “lock up” the fines within the binder. This affects the level of free binder within the mix which is vital to performance and specifications are now provided to check the level. Excess binder will cause the mix to go “fatty” and any free and unmixed binder will tend to float to the surface. Alternatively, too much filler is describes as a filler rich mix which can cause the asphalt to be ‘doughy or spongy’, and compaction can then be a problem

Mineral filler can be introduced from a number of sources: bag-house fines, fly-ash lime. The type does certainly affect the properties of the mastic, and in North Queensland the approved mix designs are currently for 100% hydrated lime. The technology input here is that it changes with the ability of the mix to flow over time. The lime component especially acts in a strengthening mechanism rather that just a component of the fines ratio. This test is certainly in all of the latest specifications, and Section 7.5 provides results on this test from the data on the trial sites.

The test will predict the importance on the effect of fillers and absorbent fines, and whether the mix will therefore be ‘fatty or boney’. It also determines the ratio of effective vs. free binder volume on the mix. The results will also use the data to determine the fixed binder fractions, which is the amount of bitumen absorbed into the filler. The test uses raw data which compares the bitumen content % with the minus 75mm fines ratio sieve size. The ratio should be 1.3. The fibre within the SMA mix design also acts to help trap free binder, depending on types and on the specific entry point into the mixing process.

7.4.7 Specific Determinants

The relationship of all constituents is obviously important. Some of the parts to this recipe based mix remain fairly constant and maintaining the volumes and sizes within a reasonable range will provide a stable mix. Other parts require the analysis of sensitivity in the mix where slight changes in volume and changes in specific types of constituents are compared.

Binder tends to be vitally important, and the ability of the filler to trap free binder especially for rutting. Of course if there is good stone to stone contact SMA shouldn't rut. Excess binder will result in flushing, as discussed previously, and especially on hot days the bitumen is dragged to the surface forming' blotchy fatty spots'. If excessive, these spots join together and form larger deposits.

One of the inherent issues that arise out of the results over the years in North Queensland is the production capabilities. Due to the continual control testing and auditing of asphalt in general, the two production plants are consistent enough to place production limits on the various procedures. The problem becomes similar to that of the source materials in the area, in that the economics of the situation are such that adjustments need to be made to the mix designs of SMA to incorporate the capabilities and constraints. Aggregate gradings are one of those fairly constant measures, and these will be reviewed through test data in the next section.

7.4.8 Site Details

The sites were not only prioritized on the basis of issues and type, but also on data from yearly condition surveys on the roadways. Appendix C shows print-outs from some of the sites with rutting, cracking and profiles of each section. Appendix D then lists the sites where SMA has been layed in totality, and then the reduced table. Appendix E gives an example of the sites where bleeding is occurring and where different types of binder have been trialed.

7.5 Testing

Results from much of the testing for SMA is about conformance and maintaining a balance between upper and lower limits on certain criteria. The graphs and tables in the following sub-sections have been derived from trial site raw data from North Queensland, and from the Boral plant. The main areas of representation are surface texture, filler – binder ratio, field voids, gradings and polished aggregate values. Appendix F shows a summarized table of the raw data for the specific project areas.

7.5.1 Surface Textures

Various local and in-house Department trials of SMA surfacing on a the main highways demonstrate that particles tend to collect in the voids in the surface and so effectively reducing the texture depth. As soon as the surface is opened to traffic, a high proportion of the grit particles are then picked up by vehicles tyres and dispersed to the sides of the road.

Table 7.3 below shows the change in texture depth that might be expected over the first few months of trafficking. An initial texture depth by the sand patch method of just over 1.5mm may reduce to around 1.1 or 1.2mm within the first twelve months, depending on the traffic density, before tending to level off during a further period of trafficking. Associated with this behavior, the change in surface friction with speed is also of interest. The required friction level of SMA surfaces seems to require local clarification, but as an interesting feature of some of the completed testing is that the friction of the SMA surface does not appear to be influenced by speed to any great degree over the range examined. This might suggest that texture depth may not be quite so critical for this type of material, but further work would be needed to substantiate such a claim.

Macro Texture Averages					
SITE	**1998**	**1999**	**2000**	**2001**	**2002**
SMA – Right Wheel track	0.37	0.38	0.97	0.94	1.07
SMA – Combined	0.38	0.36	0.97	0.97	1.05
01S-0707-I					
Left Wheel track	1.50	1.45	1.14	1.27	1.28
Right Wheel track	1.45	1.46	1.05	1.41	1.39
Combined	1.47	1.45	1.09	1.34	1.33
01S-0707-D					
Left Wheel track	1.47	1.55	1.16	1.37	1.35
Right Wheel track	1.44	1.57	1.12	1.41	1.37
Combined	1.45	1.56	1.14	1.39	1.36

Table 7.3 – Macro Texture Averages

Evidence to date also suggests that SMA can tolerate application on a relatively irregular, existing surface without detriment to the regularity of the final surface. This may well be related to the need for a smaller surcharge of material for compaction by the paver's screed and rollers.

7.5.2 Filler – Binder Ratio

The ratio of the various projects and the fixed binder fractions can be seen in Figure 7.4. When comparing the results to the recognized figure of 1.3, the following correlations can be drawn from the data.

As mentioned previously, the results should indicate a relationship between the binder and filler and can be used as a guide for the fixed binder fraction and the free binder. The results show that generally the ratio is higher than the specification which can either indicate too much binder, not enough fines/filler or a combination. Of course it is certainly possible that the filler may not be the right type.

The fixed binder can be directly calculated from the filler/ binder ratio, and this gives us a relative figure for the free binder. With binder being promoted as the trigger for fatigue and good pavement life, a mix such as SMA tends to have increased bitumen content.

FILLER / BINDER RATIO

SECTION	PRODUCTION MIX Average	Std. Dev.	Range
Caravonica -647	1.40	0.10	1.51 - 1.23
Park Ridge - 647	1.38	0.11	1.49 - 1.21
Rex Range	1.40	0.05	1.47 - 1.34
Gillies	1.27	0.10	1.35 - 1.05
McCoombe - 647	1.32	0.08	1.49 - 1.20
Cook Hwy. - 20A	1.41	0.05	1.56 - 1.36
Bruce Hwy. - Wrights	1.42	0.09	1.58 - 1.33

Figure 7.4 – Filler/Binder Ratio

7.5.3 Field Voids

The general minimum compaction value is currently set at 94% for SMA. The figure has fluctuated over the past years and generally has moved in an increasing direction from 91 to 94%. The data from the various trials would show that most of the compactions only just pass the minimum for voids. Figure 7.5 shows that on the trial locations the percentages vary over a couple of percent. Even though some projects would be deemed to have failed, they were layed before the interim specification. The figures alert us to the fact that contractors are only achieving the minimum of compactive effort from the particular year of specification.

Figure 7.5 also shows a representation of the field densities achieved on the prioritized trial sections. The % compaction scale is a CV average, although the chart is populates to produce a full spread of data.

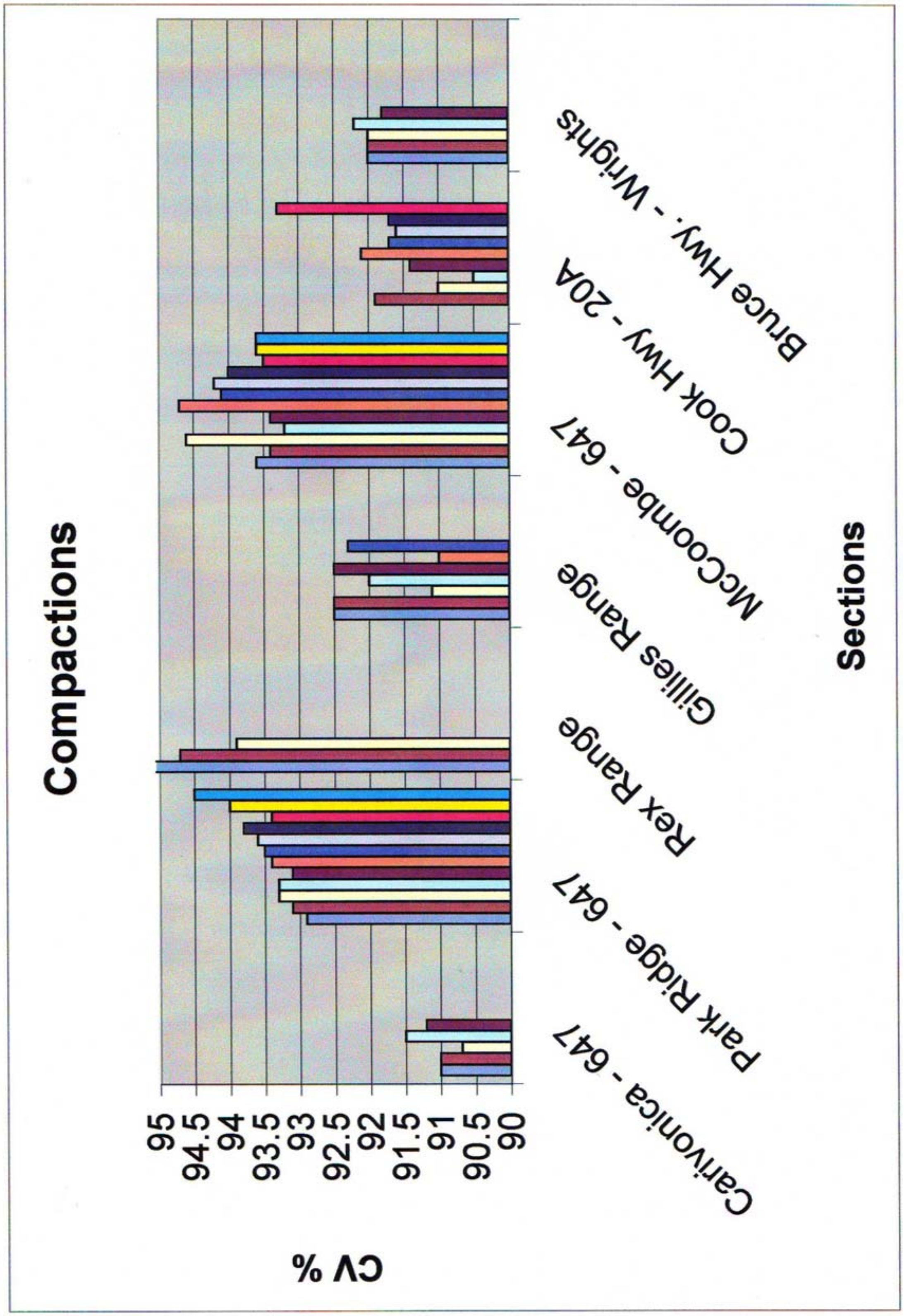

Figure 7.5 - Compactions

The compaction effort obviously affects the density result, but it must be done correctly and with appropriate equipment. It is not simply a case of extra roller effort to achieve a higher result. Even with everything being correct, it is really the mix that allows for proper and achievable compaction.

The field voids are then a direct relationship, and the results of the testing are seen in Figure 7.6. The spread of results should stay reasonably close, and with the scale being in 0.5% range, most of the project sections are tight in range. If design, production, and then laying could stay consistent then the figures should remain within a tight envelope. The problems arise out of pure volume which requires many days of manufacture and field works.

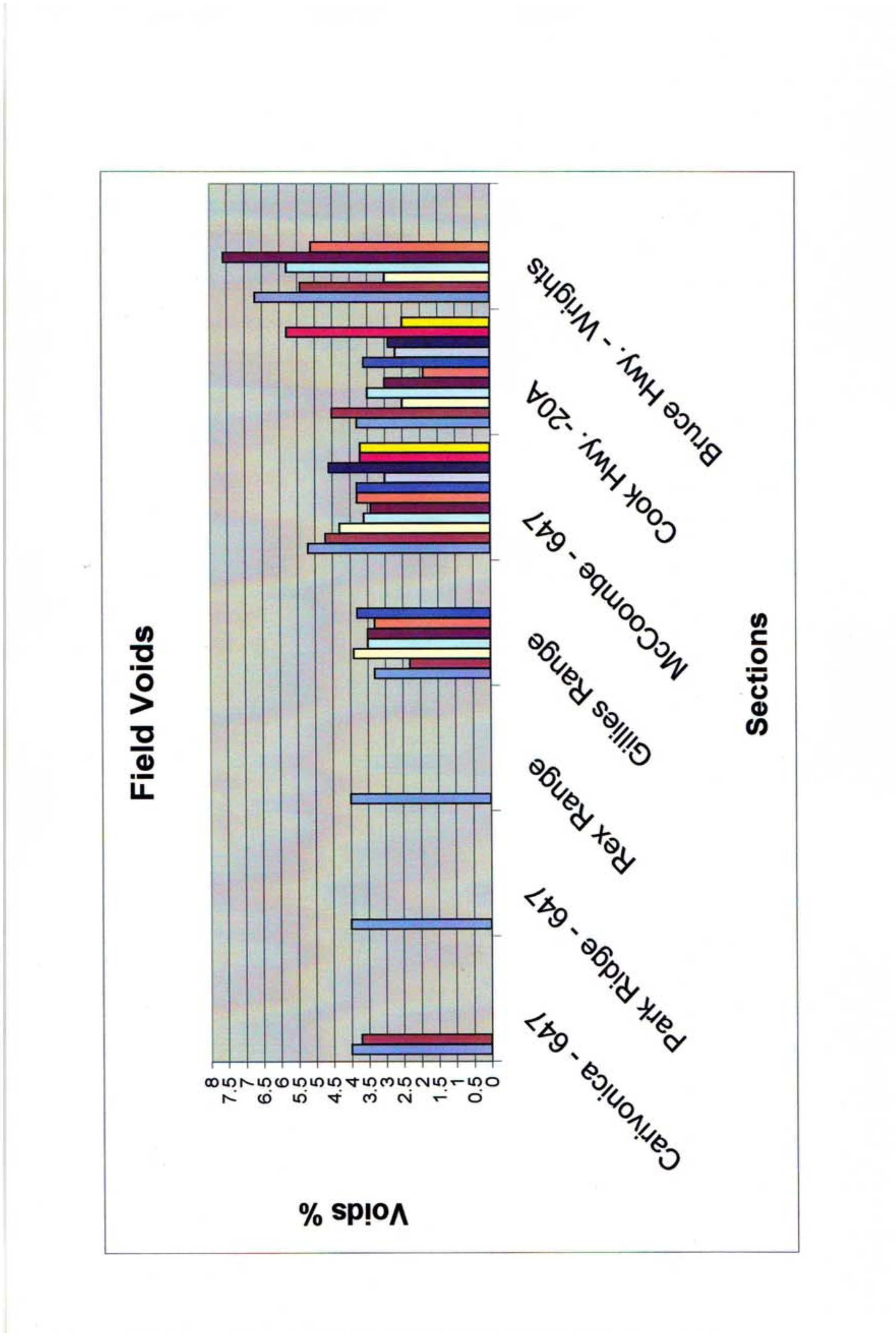

Figure 7.6 – Field Voids

7.5.4 Gradings

The grading samples are taken in every production day, so the amount of results is such that averages make more sense to compare in a graphical sense. The limits of the envelope show trends of the various samples which tend to bounce from the upper to the lower sides in certain cases. The graphs of the various combined grading curves for SM 10, 12 and 14 mixes are in Appendix F and H. The total gradings for all of the trial sites on every daily production data was placed against the target upper and lower limits. The results basically proved the daily production was in specification.

There is a tendency for the actual gradings to bounce from one side to another of the limits, but there is a specification to cover this criterion. All samples passed, so the benefit of further analysis was limited for this area. This does not detract from the importance of this test, but merely allows a check on conformance. The discussion about tightening the limits is worthy of further analysis and this is mentioned in later sections of the report.

7.5.5 Polished Aggregate Friction Value

The draft District report indicates that the aggregate used in the SMA was above but close to the minimum acceptable value for the characteristic known as Polished Aggregate Friction Value. From the construction records the District report suggests that this characteristic may have been slightly below the specified minimum value. Hence the aggregate used to manufacture the SMA may have been more susceptible to polishing.

The early life skid resistance was similar to that of conventional bituminous surfacings and improved with time but this could take up to two years and in exceptional cases three years to achieve. Thereafter the material remained consistent before experiencing a decrease in skid resistance in the following years as the aggregates at the surface polished.

As mentioned previously, the texture of 14mm SMA measured in the twelve months following re-surfacing is below the optimum required for a new surfacing but is still above the recommended threshold level of 0.6mm. SMA surfaces with 10mm coarse aggregate give a higher skid resistance but have 25% lower texture.

As mentioned previously, Appendix C shows the testing of the PAFV's over the years at the various North Queensland quarries. It's worth noting that the results only just pass the State wide specification of 45, and rarely pass the interim and current North Queensland specification of 50.

7.6 Specifications

Texture depth has a marked impact on the high speed skid resistance of a road surface. The average Sensor Measured Textured Depth (SMTD) on the sections examined has an initial value lower than expected of a new surface. There is a marginal improvement in the year two before a gradual decrease in the following three years.

Friction on low textured surfaces falls more rapidly with speed than for high textured surfaces and of the 14mm SMA sites considered all but one were above the specified threshold value of 0.6mm but five had a SMTD value below 0.7mm in the year after re-surfacing.

The single sized nature of the aggregate skeleton in SMA produces a relatively high void content filled with binder rich mastic mortar. This mixture allows the coarse aggregate to be re-orientated during rolling and presents flat sided aggregates at the surface. This helps provide the noise reduction welcomed by many motorists and residents but reduces the materials ability to provide adequate texture. An important aspect in the manufacture of the material is to maintain a volumetric balance to avoid fatting up the mastic mortar which exacerbates the problem of poor texture.

The sensor measured texture depth (SMTD) of the sections of road examined in this paper produced a mean value of 0.92mm with a range of 0.59 to 1.4mm. Whilst texture depth seems to fluctuate over the give years the general trend is one of a slight decrease due in part to the contamination of the void space.

The various districts throughout Queensland have been developing their own databases of issues and problems, and these are transferred back to the corporate technology bases of Main Roads in Brisbane for analysis. Whilst this is a process where-by formal and overall specifications are assessed, developed and finally implemented into a specification that is State wide, the process can take a number of years. In the interim, various areas of the department have become instrumental in ensuring that careful minor changes to the mix design are instigated through approved trials so-as to continual enhance the performance of SMA. Hence the use of interim specification guides, of which North Queensland has one for the SMA 12 mm mix. Appendix G has a reduced version showing only the mix proportion tables. Another area is South East Queensland which has recently developed an interim specification. The following table shows a summarised comparison of the differences in the two specifications, as well as the current State wide document.

The following table 7.4 is a complete condensed outline of the areas where all of the three major and current specifications differ i.e. MRS 11.33 (State Wide Spec.), SMA14, MRS 11.33b (North Queensland Spec.), SMA12 and MRS 11.33 (SE Queensland Spec.), SMA14. The major modification for the SMA12 specification is the PAFV of 50. For the SE Queensland specification, this includes the latest concepts of SMA Design, and has modifications in Air Voids and Mix Volume. Ratio. For voids there is now an increased upper limit in the field voids of 0.5%. For the mix Volume Ratio is has a design now specified at 0.95% and a production limit of 0.99%.

Property	Unit	Limit	Value		
			Specification Type - Stone Mastic Asphalt		
			MRS 11.33b SMA 12	MRS SE Qld. SMA 14	MRS 11.33 SMA 14
Marshall blows	Number	-	50	50	50
Stability	kN	Minimum	6.0	6.0	6.0
Flow	mm	Minimum	2.0	2.0	2.0
Stiffness †[1]	kN/mm	Minimum	2.0	2.0	2.0
Air voids in the compacted mix	%	Minimum Maximum	1.0 tol, 2.5 design 6.0 tol, 3.5 design	1.0 tol, 2.5 design 6.0 tol, 4.0 design	1.0 tol, 6.0 tol
Voids in mineral aggregate (VMA)	%	Minimum	16.0	17.0 des., 16.0 prod.	14.0
Binder Volume (unabsorbed) †[2]	%	Minimum	14.0	13.0	14.0
Binder drainage	%	Maximum	0.3	0.3	0.3
Sensitivity to water	%	Minimum	80	80	80
Wheel tracking rate	mm/kCycle	Maximum	TBR	TBR	TBR
Wheel tracking rut depth	mm	Maximum	TBR	TBR	TBR
Mix volume ratio †[2]	-	Maximum	1.00	0.95 des, 0.99 prod.	1.00
Flakiness Index	Limit	Maximum	15	20	30
Crushed Particles	%	Minimum	100	80	80
PAFV	Value	Minimum	50	45	45
Compaction Standard	%	Minimum	90	91	92
CV	%	Minimum	94	94	93

Table 7.4 SMA Specification Comparisons

7.7 Mix Design Methods for SMA

The highly rut resistant properties of SMA depend on the establishment of stone to stone contact in the mix. This stone to stone contact is assumed to be formed by the coarse aggregate, which in Australia, is defined as the material retained on the 4.75 mm sieve. Sufficient fine aggregate, filler, binder and fibres are added to produce a durable mix but not interfere with the establishment of the coarse stone to stone contact.

In Germany where "Splittmastixasphalt" (SMA) originated, there is no true method. Mixes are selected from an array of standard mixes defined as "recipes." These have been developed through years of experience and have been applied to the various levels of traffic volume by using the Marshall test to analyse the voids in the mix and to select the percentage of bitumen (Luminari and Fidato 1998). The process is thus entirely empirical.

A good overview of European practice is given by EAPA (1998). A draft European Code for SMA has been released (PrEN 13108-5:2000). In the United Kingdom, the design method for their rolled asphalt is specifies in a British Standard, BS 598: part 107(BSI 1990), however, there is no parallel design method for SMA. A draft US specification based on a series of specific sub-clauses to amend BS 4987 and the Specification for Highway Work has been published (Loveday and Bellin 1998). The design procedure is based on achieving a specified target grading and using Marshall compaction.

The early SMA projects in the US were generally built with the coarse aggregates having 100% of the material passing the 12.5mm or 19.0mm sieve. In addition, the amount of combined aggregate (coarse and fine aggregates and mineral filler) was typically in the following ranges:

· Passing the 4.75mm sieve – 27% to 33%;

· Passing the 2.36mm sieve – 18% to 23%; and

· Passing the 0.075mm sieve – 8% to 12%.

This lead to a 30-20-10 “rule of thumb” targeting 30% passing the 4.75mm sieve, 20% passing the 2.36mm sieve and 10% passing the 0.075mm sieve (Scherocman 1997). Brown et al (1997b) have demonstrated that the SMA mixes have tended to become coarser from year to year since 1991 with the specification for percent passing the 4.75mm sieve now in the range of 20% to 28%. The reduction was made based on research that indicated for most SMA mixtures the percent of material passing the 4.75mm sieve had to be less than 30 percent to ensure stone on stone contact and to meet minimum VMA requirements. The Australian experience also reflects the increasing coarseness of the SMA grading from SAA (1995) to APRG (1997a) and AAPA (2000a).

In Australia, these mixes are currently being designed using a “recipe” method. Typical compositions and volumetric properties for Stone Mastic Asphalt mixes are available (SAA 1995, APRG 1997a, APPA 2000a). In Queensland then DMR MRS11.33 specification provides a grading envelope within which the target grading must lie, together with requirements for binder and fibre type and content. Compaction is by Marshall hammer and Marshall Stability and Flow requirements are stated. MRS 11.33 also has a requirement unique to SMA for a “mix volume ratio” that is defined by test method Q318-1998. This ratio is defined as the volume of the components other than the coarse aggregate (≥ 4.75mm) within a compacted mix expressed as a proportion of the volume or air voids contained within the coarse aggregate in a dry rodded condition.

In effect, it is ratio of the volume of mastic (bitumen, filler and fine aggregate) to the volume of voids in the coarse aggregate skeleton. For SMA14, the ratio is specified to be less than 1.0, however for SMA10; the value is just to be recorded. By maintaining the mix volume ratio at less than 1.0, stone on stone contact for the coarse aggregate will be maintained. This requirement is based on volumes, whereas the grading specified are based on weight. Specifications such as MRS11.33 are not recipe composition specifications in the conventional sense because the producer must use skill and judgment to design a mix within these constraints that will satisfy performance requirements (Loveday and Bellin 1998).

7.8 Development of New Mix Design Methods for SMA

There is a world wide interest in the development of a rational method for the design of the grading of SMA mixes. The National Center for Asphalt Technology (NCAT) method is intended to identify the point at which addition of fine aggregate cause's expansion of the coarse aggregate structure and thus prevents the coarse aggregate stone on stone contact which gives SMA its very high rut resistance. The concept of the NCAT method is that the mix design proportion for fine aggregate is determined by adding increasing amounts of fine aggregate in a series of trial mixes and identifying the point at which further addition of the fine aggregate expands the coarse aggregate matrix. The percentage of fine aggregate which first causes the expansion is called the dilation point (Brown et al, 1997a).

The method uses two types of voids calculations. They are the conventional voids in the mineral aggregate (VMA) and the voids in the coarse aggregate (VCA). VMA is the volume of the mix which is not occupied by the mineral material (coarse and fine aggregate and filler) expressed as a percentage of the total volume of the mix. VCA is the volume of a mix not occupied by the coarse aggregate expressed as a percentage of the total mix volume. The point where a change of slope occurs on a plot of VMA and/or VCA verses Percent Passing 4.75mm sieve is interpreted as the dilation point.

Procedures have now been prepared in the United States for the design of SMA mixtures as the draft AASHTO standard. The procedure involves proportioning the percentage of aggregates by volume and mix design examples are available (NCAT 1998c).

The NCAT method also attempts to pre-estimate VCA by determining the bulk density of a sample of the coarse aggregate compacted into a large cylinder by dry rolling. ARRB (1998) research concluded that the measurement of the dry rodded VCA of the coarse aggregate, although rapid and thus an attractive and perhaps indicative procedure, may not duplicate the aggregate packing in the bitumen bound mix and thus may be unsuitable for incorporation into a design procedure.

Determination of the volume of compacted samples by mensuration is recommended by ARRB (1998). When samples are prepared by the Gyropac, the volume can be determined by measuring the sample height, as determined from the piston position, and multiplying the value by the area of the base of the sample mould. Given that the area of the base of the samples will be constant, a plot of Gyropac height verses percentage fines can be used to identify the dilation point (AAPA 2000a). Based on this concept of measuring the dilation point, ARRB (1998) proposed a procedure for the design of the aggregate skeleton based on a series of trial mixes with differing percentages of fine aggregate added to a common coarse aggregate fraction.

However, it is now recommended that … "The recipe approach to SMA design, based on the AAPA Design Guide (now AAPA 2000a), should be incorporated into the Provisional Guide (APRG 1997a) as the preferred method, with the dilation point approach reserved for special mixes" (Oliver 1999). The gradings in AAPA (2000a) are based on the Maccarrone et al (1997b) CSRE-1 gradings and are much coarser than those originally include in APRG (1997a).

Given the importance of the "undilated" stone skeleton to ensure that the good rut resistance properties of SMA are maintained, a rational method of design such as the Dilation Point Method (DPM) would appear more desirable than the return to a recipe approach. Past research has presented theory to identify a means of extending the DPM by incorporating particle packing theory and the resilient modulus test.

7.8.1 "Design of SMA Mixes" – Australia

This research undertaken by ARRB Transport Research (1998) investigated the Dilation Point of SMA mixtures by adding varying percentages of fine aggregate to the coarse aggregate skeleton. Based on this research, a draft mix design method for the aggregate grading was proposed. The report conclude that the dilation point procedure appears to have promise but it may be desirable for the concept to be more widely considered before a decision is made on its adoption. An alternative path suggested was to specify a grading envelope based on the grading produced by the identification of the dilation point.

7.9 Implications for the Design of the SMA Stone Skeleton

The collected research demonstrates that unless a singles sized coarse aggregate is used, it is not possible to design an undilated mix using the Dilation Point Method in its current form. A 10mm SMA designed by the dilation point test method will have a grading significantly different to a mix designed to DMR (Qld) MRS11.33.

Results from the various resilient modulus testing with Stephensons Report (2002) support the Dilation Point Method. For a single sized coarse aggregate, it is a simple means of verifying the dilation point. For mixes with a more complex coarse stone structure such as the DMR(Qld) MRS11.33 SMA10, the resilient modulus testing was able to identify upper grading limits to ensure an undilated “double stone skeleton: where a single dilation point could not be identified from the change in “Gyropac” height. The concept of an undilated “double stone skeleton” accounts for the good rut resistance that has been reported for the Queensland SMA in service (Hogan et al 1999). Such a comment could not be made based solely on the “Gyropac” height verses the fines content due to the complex interactions between the two stone skeletons and the mastic.

Whilst there are strong relationships between voids contents and the percentage passing the 4.75mm sieve, models developed to predict the voids content include the percentage passing the 4.75mm sieve as only one of the important factors. The models suggest that for SMA10, the percentage passing the 9.5mm, 1.18mm and 75µm are also important. The research also demonstrates that it is the relationship between percentages passing adjacent sieve sizes e.g. 4.75mm/2.36mm that is important. The filler content will also have a significant effect on the final voids in the asphalt. Consideration of packing theories has confirmed that the maximum size of the aggregate used in the mortar is dependent on the size of the coarse aggregate in the stone skeleton. It is suggested that the cut-off between coarse and fine aggregates should be 2.36mm for the SMA14 and 1.18mm for the SMA10. The 1.18mm sieve was also identified in the predictive relationships fir the SMA10.

The development of the stone on stone contact in the stone skeleton is a function of both the grading of the coarse aggregate and the maximum size of the fine aggregate used in the mortar. Particle packing theory shows that using the 4.75mm sieve as the maximum size for the mortar components, as used by many researchers (NCAT 1998c, ARRB 1998) is inappropriate. This study has found that the sieve size should be varied depending on the grading and maximum size of the coarse aggregate.

7.10 Extended Method for Design of the SMA Stone Skeleton

Based on the research of Stephenson Report (2002), incorporating additional steps into the Dilation Point Method (DPM) of SMA Mix design, provides methods for selecting the maximum size of the fine aggregate in the mortar and determining the upper grading limits of the SMA. By using these steps, the DPM can also be applied to the “double stone skeleton” SMA as used in the DMR (QLD) MRS11.33 specification. The design of SMA mixes using the extended DPM provides a rational means of selecting the combined aggregate grading to ensure that the important stone on stone contact is maintained. The extended DPM also gives a simple means of assessing the complex interactions within the combined aggregate grading which are ignored when using “recipe” methods.

7.11 Summary

The Chapter highlighted comparisons made between different compaction methods and methods of measuring voids and provides some relationships between the various methods. The method of measuring the voids content has a significant effect on the value reported. The mensuration method always gives a higher value when compared to the silicone coated method however there is a strong relationship between the two methods of measurement. The greatest difference occurs at higher voids contents. In the mensuration method, the surface voids are included in the voids content whereas with the silicone coated method, these voids would be filled with silicone and excluded from the voids content. Any comparison between reported voids content needs to consider the method used to measure the voids to ensure that appropriate comparison is made. This thesis presents the need to highlight that there are useful methods when comparing voids measurements by using the two different methods.

The experimental programme and literature review of testing undertaken for this thesis revealed the limitations of the current methods for the design of SMA mixes without trial data. Based on the empirical evidence reported in this thesis, sections of actual roadway with SMA surfaces need a continuous program for experimental research. A "cradle to grave" trial (start to finish with complete process control audits) is necessary to ensure that variables are revealed and compared with actual failures, but based on empirical evidence.

By rating certain criteria over time a relationship is developed to be able to modify to standard specifications. The critical upper limits of the grading curve can be determined by using the resilient modulus test. The extended DPM from other research data provides the means of designing complex "double stone skeleton" SMA mixes such as specified in DMR (Qld) MRS11.33.

The analysis of the actual field voids and compaction results within the chapter promote that it is worthy of future and more detailed testing of sites. The compaction will always hover around the specification, due somewhat to the commercial environment. The field voids are spread too much though per project, and as we see from the Cook Highway and Bruce Highway sections, there are changes where the results are quite different. With SMA, this can really change the appearance, surface texture, and fatigue of the mix.

With the filler/ binder ratio once again it is the variation. If the results were tighter and with less standard deviation, at least one could concentrate on the design issues. The results again from some of the sections show that is may be a more of control issue. The results would tend to support excess free binder in the mix, which is certainly evident in the field. The upper and lower limits in most cases are about the specification, and the centre line result would average high. The Gillies project though is certainly the only one on the lower side, and it is at this location where many of the trials were conducted, and they are certainly performing well in relative terms.

CHAPTER 8 – HOT WEATHER CONSIDERATIONS AND DEFORMATION RESISTANCE

8.1 Introduction

The basis of the report was particularly SMA produced and layed in a tropical climate. Developing criteria and known failure mechanisms for hot and wet weather is part of the consideration of this climate, and certainly for the zone where this Thesis is surrounded – Tropical North Queensland.

8.2 Need for consideration

It is generally recognised that weather conditions have a significant influence on the laying and compaction of asphalt. Cold, wet and windy conditions during laying can result in poor quality asphalt that will perform badly in service. Most areas with problems have occurred in the colder climatic conditions; so that laying difficulties were seen to be associated with low air temperatures. Understanding of the effects of wind speed and air temperature on the cooling of hot laid asphalt layers increased rapidly in the 1980s. The term ‘Cold Weather Working’ was replaced by ‘Adverse Weather Working’ to emphasise the influence of wind speed. In assessing adverse weather, the emphasis was still on adverse cold weather laying conditions and the effect of solar radiation was regarded as a benefit that was generally discounted in specifications.

Difficulties following the laying of asphalt in adverse hot weather became very apparent in Queensland during the very hot summer of 1995 when early failure, in the form of excessive deformation and loss of texture depth, occurred on several occasions. The problems arose from paving materials being trafficked whilst the material was still too hot, although this has been partially alleviated by the current trend to use thinner layers that cool more quickly. Nevertheless, there is a need for advice about laying bituminous materials in hot weather when the time taken to cool sufficiently to permit trafficking can be critical. If the time allowed is too short, it can result in premature rutting by the traffic. If adequate time is allowed, there may be considerable traffic congestion that will lead to pressure to open the site to traffic prematurely. Whilst this is primarily a problem associated with surface course materials, it can also affect the lower layers that are trafficked by site vehicles or are used as temporary running surfacings. Also, the temperature of lower layers will have an effect on the rate of cooling of any layer placed over it.

In order to provide authoritative advice, one needs to be able to estimate the time that a pavement course takes to cool sufficiently before it can be trafficked under various weather conditions. Hence, a model was developed by Nicholls and Carswell (TRL Report 494) to estimate the rate of cooling and a criterion established for the condition of the pavement layer that will not be detrimentally affected by trafficking. The cooling model, which incorporates the trafficking criterion, can be used as the basis for advice about whether the conditions are, or are not, suitable for laying bituminous materials within a specified time period before being exposed to traffic.

8.3 Laying asphalt in hot weather conditions

8.3.1 Potential Problems

When hot asphalt is laid during weather conditions of high ambient temperature, particularly during continuous periods of strong sunlight, it can remain workable for a considerable time. During the laying and compaction it may be difficult to maintain profile and, in the case of hot rolled asphalt surface course with added pre-coated chippings, it may be difficult to achieve adequate texture depth.

During extended periods of hot, sunny conditions, the newly-laid surfacing layers of a pavement can maintain temperatures after opening to traffic that are sufficiently high to allow excessive rutting and the rapid embedment of any chippings, with the latter causing a reduction of texture depth. The conditions are compounded in conditions where traffic intensity is high and speeds are restricted. Excessive texture depth loss and rutting may affect vehicle steering and braking.

8.3.2 Cooling of asphalt layers

Heat is lost from a hot-laid asphalt layer by conduction into the cooler substrate and by convection and radiation from the top surface. As the hot asphalt layer cools, the heat flow into the substrate will reduce as the layer approaches a condition of temperature equilibrium with the environment.

The rate at which an asphalt layer cools depends on both environmental and asphalt factors. Both higher wind speeds and lower air temperatures increase the cooling rate and, hence, reduce the time available for compaction. Increasing incident solar radiation reduces the cooling rate, thus extending the time available for compaction. Asphalt factors that affect the cooling rate include its temperature, thermal conductivity, specific heat, surface albedo (reflection coefficient) and layer thickness. The most important material factor is asphalt layer thickness, followed by its temperature. For fixed conditions, the cooling time is proportional to the asphalt layer thickness raised to the power 1.8 (Daines, 1985). Calculations describing the cooling behaviour of hot laid asphalt layers are complex (Jordan and Thomas, 1976) and require the use of computers. However, there are simplified equations that are sufficiently accurate for practical purposes to predict the time available for compaction (Daines, 1985; Nicholls and Daines, 1993).

This method of estimating the cooling behaviour of a hot-laid asphalt layer is only applicable down to a mid-layer temperature of about 80°C for constant environmental conditions. Extrapolation using this method to lower temperatures is unreliable and an asphalt layer will not cool below about 50°C, twice the air temperature in degrees Celsius, on a day that it hot, calm and sunny. This temperature can be compared to the safe temperature for trafficking of below 50°C.

Some of the heat absorbed by the pavement from a newly laid hot asphalt layer remains when the next daily temperature cycle resumes. The temperature behaviour of the pavement, from the end of the compaction period until the new layer has cooled to ambient temperature, is influenced by the cyclic effect of solar radiation. This is extremely complex and, to date has not been modeled. Nevertheless, in general a day must elapse before the heat from a 50mm thick layer is dissipated and three days for a 150mm thick layer.

8.3.3 Solar Radiation

Solar radiation is stronger and endures for longer periods of the day during the summer months, although its intensity can be reduced by cloud cover. Table 8.1 gives measurements of solar radiation at the Meteorological Office in Backnell, Central Southern England, in which the 99th and first percentile figures related to full sunshine and full cloud cover, respectively.

Table 8.1 - Total incident energy averaged between 12:00 and 13:00 EST

	Total incident energy (W/M²)	
Month	***99th Percentile (full sunshine)***	***1st Percentile (full cloud cover)***
January	900	50
April	322	20
July	550	40
October	830	70

Table 8.1 shows that high levels of solar radiation can occur in the months from October to March, and that even in winter solar radiation can be appreciable. The risk of hot weather having an adverse effect on the laying of asphalt is obviously higher during

the summer months of December to February, but it may be significant for shorter periods of the day in the spring and autumn.

New asphalt road surfacings, which can have an albedo (reflection coefficient) close to zero, are blacker than ‘weathered’ surfacings. Unfortunately, from the viewpoint of construction and early trafficking, a black new surfacing is more vulnerable to the effects of solar radiation and therefore at greater risk of deformation during the first summer compared with subsequent summers. Although, in the past, it was assumed that road surface temperatures rarely exceeded 45°C, recent hot summers have demonstrated that the asphalt surface temperature can reach, and may even exceed, 50°C, particularly for those in North Queensland.

8.4 Risk Assessment Model

With such an unforgiving type of pavement, the risk of various criteria needs to be assessed on paper. One procedure for achieving this is through modeling.

8.4.1 Requirement

The model was developed to help assess the risk of unacceptable deformation occurring when asphalt is laid in hot weather conditions.

8.4.2 Factors

The risk of a road surface deforming (and losing texture depth) is related to several parameters, of which the primary ones are:

- The temperature throughout the depth of the pavement;
- Deformation resistance of the asphalt layers at those temperatures;
- The loading on the pavement from the traffic flowing over it;
- The speed of traffic; and
- The length of time that the conditions persist.

To develop a risk assessment model, these parameters are rationalized into the following factors;

- Material & Temperature Factor:
  the wheel-tracking rate at the maximum surface temperature, in mm/h;
- Traffic Load Factor:
  the design commercial traffic flow, in cv/l/d;
- Traffic Speed Factor:
  the time that wheels load the pavement, in s;
- Optional Action Factor:
  an allowance for specific measures, in particular using light-coloured pre-coated chippings and/or cooling the surface; and
- Time Period Factor:
  The period for which the other factors remain reasonably constant, in days.

The wheel-tracking rate of the top layer of the road pavement at the maximum surface temperature, in mm/h, is taken to reflect the material and temperature parameters. This will be conservative in that there is likely to be a temperature gradient through the depth with the material being cooler further down but optimistic because deformation in lower layers will not be considered. In the event of no better information for the particular circumstances, the maximum road surface temperature is assumed to depend on the latitude and the month of construction. Other factors that can affect it are the road direction, slope and extent of shading, both from sunlight and from winds that can cool the surface.

The applied load and loading time will influence the deformation. For asphalt binders showing Newtonian behaviour, the deformation will be proportional to the load and to the loading time. Bitumen binders and more particularly modified binders, exhibit some non-Newtonian behaviour because the binder viscosity is reduced by increased shearing rate. Nevertheless, for the purposes of this risk assessment method, the risk of deformation due to traffic load is taken to be proportional to the duration of vehicle loading, estimated in terms of commercial vehicles per lane per day.

Loading time is also inversely related to the traffic speed. Deformation increases with loading time, particularly for long loading times when the visco-elastic properties of the binder are less applicable and the binder tends to behave as a Newtonian fluid. For the purposes of this method, deformation is taken to be inversely proportional to vehicle speed.

However, calculating the equivalent loading for stationary vehicle is more complicated. For the stationery phase, the loading and speed factors need to be combined into a single factor that can be regarded as the proportion of the time when a commercial vehicle is loading the area, in s per day. 7s of stationary loading is equivalent to 1 wheel-pass at 50 km/hr and that a commercial vehicle traveling at that speed has a Speed Factor of 0.08s. Therefore, the combined factor for 3-axled commercial vehicles when permanently stationary is 3,000s/day. However, this factor must be reduced by the proportion of commercial vehicles in the traffic and by the proportion of the time when the traffic is stationary.

During the flowing phase, the total traffic loading must also be applied because each vehicle has to pass across each location, even if at a closer spacing due the stationary phase. Therefore, the deformation due to any stationary phase is additional to the normal deformation and does not replace part of it.

Clearly, deformation will increase with the length of time the other factors are extant. For the purposes of the risk assessment method, the time period is measured in days. The risk for the first day after opening to traffic can be assessed as well as the risk for an extended speed restricted period in, for example, a contra flow situation. For assessing the risk over an extended period some of the risk parameters may take different values.

8.4.3 Calculation procedure

An overall indication of asphalt deformation, summed over each period when the risk factors remain constant, may be estimated from the product of the individual factors in the following Equation

Expected deformation = $k \times \sum (R_T \times T_l \times T_S \times P_d)$

Where k = the calibration coefficient;

R_T = Material & Temperature Factor (the wheel-tracking rate at the maximum surface temperature, in mm/h):

T_r = Traffic Load & Speed Factor (the commercial traffic flow, in cv/l/d, times the time that a wheel will load the pavement, in s); and

P_d = Time Period Factor (the period for which the other factors remain nearly constant, in days).

8.4.4 Calibration of relative risk factor

A wheel-tracking rate at 45°C of 2mm/h is required to achieve a deformation rate of 0.5mm per year for a traffic flow of 6000cv/l/d (Daines, 1992). In a typical year, about 90 percent of the deformation occurs during about 15 -20 hot summer days when the road surface temperature attains about 45°C. Of course this figure is obviously arguable with relation to location and some summers are hotter and longer. A typical speed for a commercial vehicle is 80 km/hr. Therefore, for the conditions in this example:

Estimated deformation

$$= k \text{ x } 2 \text{ x } 6000 \text{ x } 0.04 \text{ x } 15 = 7200k$$

Therefore, the calibration coefficient, *k*, equals 6.2 x $10^{-5.}$

8.5 Discussion

8.5.1 Laboratory trials

The laboratory trials from investigating the change in the rate of deformation with both temperature and speed provided general confirmation of the expected relationships whilst indicating that they are not strictly accurate. The findings are equally applicable to both ‘traditional’ hot rolled asphalt and SMA.

With regard to temperature, the easiest way of modeling the relationship is to take the logarithm of the wheel-tracking rate to be proportional to the temperature. However, this is not completely accurate, with the relationship flattening off to reach a plateau at higher temperatures.

The tests showed that relatively little deformation resulted from a stationery wheel load compared to a moving wheel load. However, experience shows that more rutting develops when traffic speeds are reduced. This dichotomy suggests that loading time is important but, at the same time, a dynamic component is essential for the formation of ruts.

8.5.2 Mathematical models

There is no single solution to the problems of trafficking newly laid asphalt during hot weather. Nevertheless, there are ways of reducing the risks to manageable levels by use of a procedure such as the proposed risk assessment model. By minimising the risks at all stages of the work, from mixture production to traffic control, the amount of damage induced should be within acceptable limits. Not all the measures are necessarily applicable to all schemes and, therefore, the option measures that are most appropriate and that provide the greatest cost benefit both to the Contractor and the road used should be selected.

However, in general the conditions are unlikely to justify the use of the risk assessment model. The position should be to have conservative requirements that are simple to understand and operate with the use of the risk assessment model retained for those limited cases when its used can be justified.

On the more specific aspect of limiting the permanent deformation of asphalt, the currently used model in the UKwas developed by Szatkowski and Jacobs in 1977 for hot rolled asphalt by specifying a maximum wheel-tracking rate sufficient to limit the deformation to 10mm over 20 years. However, the traffic conditions in all developed countries over the years since 1977 and more data are available for analysis. A review of the records of various road trials of rolled asphalt were used to develop an equation to predict the development of permanent deformation in an asphalt surfacing that correlates with site data with and R^2 adj value of 0.46. The relationship implies that the permanent deformation is proportional to the traffic flow, the wheel-tracking rate at 45°C and the logarithm of the age plus one. The correlation is not very good, and other relationships were developed that had higher correlations with existing data, up to and R^2adj value of 0.72. However, there were inconsistencies when the data was extrapolated to other potential situations. In particular, the site temperature was not satisfactorily incorporated.

The use of the proposed relationship should allow better estimates to be made of the extent and development of permanent deformation under typical conditions; it cannot assist in forecasting under exceptionally hot weather conditions. New data would allow the equation to be further developed with greater confidence in the results obtained, but to obtain such data should involve long-term systematic work, ideally over a twenty year period to obtain the full life history of successful surfacings.

8.5.3 Actions to minimize the potential problems

Within the research that has been undertaken, certain actions have been identified that are believed will help to minimize potential problems that can arise when laying hot asphalt material in adverse hot weather conditions. These actions are described below.

8.5.3.1 Mixture selection

The selection of deformation-resistant mixtures can mitigate, to a limited extent, the effect of premature deformation in adverse hot weather conditions, although lower stability materials are more likely to remain workable for longer periods. Rutting is often attributed just two surface courses but road bases, and particularly binder courses, can also deform significantly. The requirements of the wheel-tracking test for hot rolled asphalt binder course should be similar to those for surface course because, although the binder course will not attain temperatures as high as the surface course during service, it is expected to have doubled the life. The counter is that, when the surface course is replaced, any permanent deformation in the lower layers will be taken out. Nevertheless, the use of more deformation resistant materials, such as multi-grade and polymer bitumen’s with a high modulus base is advisable.

8.5.3.2 Delivery temperature

Asphalt mixtures delivered to site at temperatures higher than necessary not only increase the time available for compaction but can also render the asphalt too workable to lay; it also wastes energy and promotes binder hardening. The delivery temperatures during hot weather should only be high enough to achieve the required workability. Reducing the delivery temperature for hot rolled asphalt from 160°C to 140°C reduces the time available for compaction by about 30%.

8.5.3.3 Layer thickness

The laid thickness markedly effects the time available for compaction, although contract specifications will normally have stipulated thicknesses for each asphalt layer. More flexible contract specifications would allow thinner layers to be used in hot weather conditions and thicker layers in cold weather conditions.

8.5.3.4 Rollers

The use of relatively light roller for initial compaction could be considered during hot weather conditions, particularly when rolling hot rolled asphalt with pre-coated chippings. A reduction in roller mass would diminish the risk of non-compliance with texture depth requirements due to excessive embedment of the chippings.

8.5.3.5 Time of day

Laying during the evening and night has additional advantages during hot weather. The lower air temperatures, and a reduced level or absence of solar radiation, will enable the substrate to cool more rapidly. The cooling time of a subsequent hot overlay is then considerably reduced. Road closures at this time will reduce any traffic delays and deformation.

If the weather and traffic conditions are such that the risk of premature damage to a newly opened surface is unacceptable in, for example, a speed-restricted contra flow system, a cessation of laying should be considered. Cessation of laying during the hottest part of the day, say when the road surface temperature exceeds 45°C, will not only help the Contractor to minimise problems of achieving asphalt compliance in terms of profile and texture depth, but will also enable the surface to cool more rapidly in the evening when laying can be resumed. Laying can then me resumed when the surface temperature has fallen to 30°C, usually at about 20:00, when the temperature at the depth of 100mm us also likely to be less than 30°C. Laying earlier at higher surface temperatures increases the overall cooling time of the pavement and, therefore, it is not advisable.

This is good in theory but difficult in practice with most contractors, for obvious commercial reasons.

8.5.3.6 Parking restrictions

After completion of laying and prior to opening to traffic, construction traffic should not be allowed on the newly laid asphalt. The parking of construction traffic on asphalt during hot weather may cause unacceptable, saucer shaped, depressions under the wheels (Haydon, 1994).

8.6 Conclusions

The main conclusions of this section are:

- Below about 50°C the logarithm of the wheel-tracking rate is approximately proportional to the temperature; the wheel-tracking rate flattens off.

- The loading time, and not the speed, influences the resultant deformation. However, the low deformations due to static load show that dynamic effects are important.

- A risk model can be used to assess the probability of unacceptable deformation. However, economic considerations are unlikely to justify the effort required to make use of the risk assessment model in most cases.

- The best relationship found to model deformation implies that the permanent deformation is proportional to the traffic flow, the wheel-tracking rate at 45°C and the logarithm of the age plus one. However, the value of the square of the correlation coefficient (after adjustment for the degrees of freedom) is a modest 0.46.

- There are various physical actions that can be taken when laying hot asphalt in adverse hot weather conditions to minimise the potential problems.

Specification clauses have taken into account the research already developed on the findings that are evidenced in this section.

CHAPTER 9 – STIFFNESS PROPERTIES OF SMA MIXTURES

9.1 Introduction

The previous chapter presented a method to design the coarse aggregate stone skeleton of SMA. The other important component is the mastic that binds the stone skeleton. Mastic consists of the binder, fine aggregate, fibres and fillers. Because the mechanical properties of SMA rely on the stone to stone contact, it often clamed they are less sensitive to binder variations than the conventional mixes (APRG 1997a, Brown et al, 1997b, Robert et al 1996, Kandhal et al 1998b).

This chapter discusses the research undertaken to determine the influence of the binders and fillers on the elastic properties of SMA mixes.

Investigations into stiffness properties undertaken as part of this research involved typical Queensland Department of Main Roads Stone Mastic Asphalt with a Polymer Modified Binder (PMB) containing Styrene-Butadiene-Styrene (SBS) co-polymer and a conventional binder (Class 320 bitumen). In addition, the stiffness properties of SMA manufactured to the grading requirements of APRG (1997a) and AAPA (2000a) were investigated Resilient modulus testing was undertaken at a range of temperatures to provide information on the temperature susceptibility of resilient modulus.

9.2 Plant Produced SMA10 to DMR (QLD) MRS11.33

The samples prepared during the comparison of compaction methods as discussed in Chapter 5 were used to determine the Resilient Modulus at 25°C by the indirect Tensile Method (AS/NZS 2891.13.1). Sampling and testing was performed by others in the head laboratories of MRD Brisbane.

The effect of air voids on Resilient Modulus is shown in Figure 9.1, that contains data for samples of plant produced SMA10 to DMR(QLD) MRS11.33 compacted by both Marshall Hammer and Gyropac compaction to achieve a spread of void contents. Specimens with voids contents less than 7.0% were prepared with the Marshall Hammer and those with voids contents greater than 7.7% with the Gyropac compactor. The modulus remains essentially constant for voids up until about 9.0%, before reducing sharply. The data shows a rapid reduction in strength with increasing voids contents which could illustrate the inappropriateness of the test to material with void contents in excess of 9.0%. At 5% voids, the average resilient modulus is 1073 MPa.

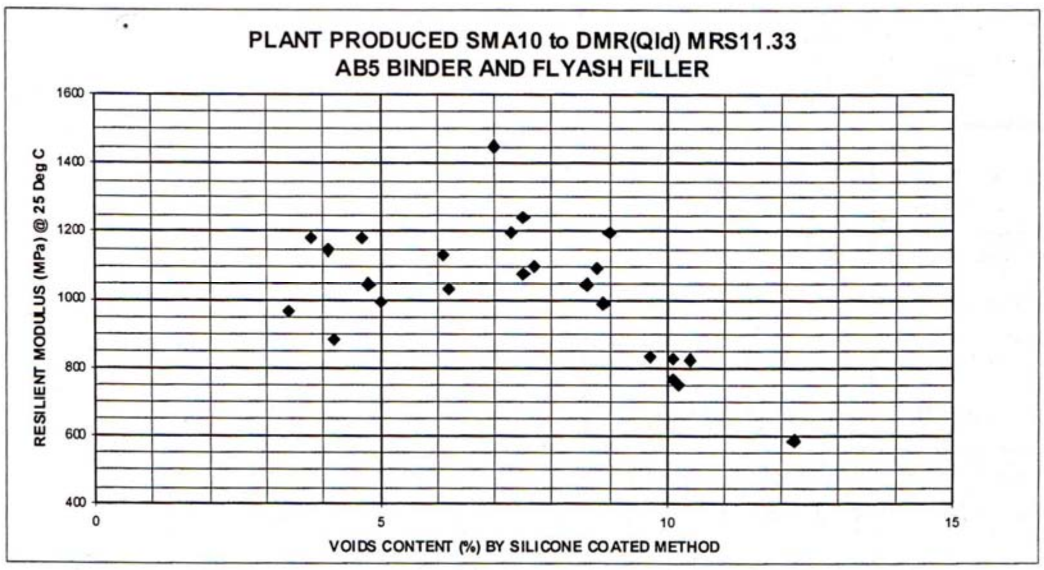

Figure 9.1 – Effect of voids content on resilient modulus

Generally the data in this and the next section is proved to be very similar whether the design mix details are taken from an approved SM 10 or SM 14, or the interim approved design for the SM 12 in North Queensland. The changes in certain criteria are followed through the whole specification to the completed stage of performance driven analysis. This ensures that so long as the total of one type of mix size is used then the sample is representative of an SMA.

9.3 Laboratory Produced Samples to DMR (QLD) MRS11.33

DMR (Qld) (2001a) reported on resilient modulus testing undertaken on laboratory produced asphalt compacted with “gyropac” compaction. A summary of the modulus values reported is given in Table 9.1 which shows that the mixes containing SBS PMB have a significantly lower resilient modulus than this made with the Class 320 bitumen binder. APRG (1997a) states that using soft grade binders such as those modified with SBS polymer will produce a very low resilient modulus of about 2,000 MPa or less. The values determined for the plant produced SMA10 and reported for the DMR (Qld) (2001a) investigations are consistent with the APRG (1997a) statement. The resilient modulus for the plant produced SMA10 (Approximately 1,073 MPa @ 5% voids) is significantly lower than that of the laboratory produced mix (1,535 MPa). The plant mix contained natural coarse sand and fly ash filler compared to all crushed rock fines and filler in the laboratory mix. The lower modulus can be attributed to the use of rounded smoother textured particles in the plant produced mix (APRG 1997a).

Mix Type	**SMA10**				**DG14N**	**SMA10**
Properties	AB5 PMB + UFD Filler	C320 Bitumen + UFD Filler	C320 Bitumen +Lime Filler	C320 Bitumen +Fly ash Filler	C320 Bitumen +UFD Filler	AB5S PMB Fly ash Filler
Voids Content	5.5%	5.3%	5.2%	5.1%	4.5%	4.1%
Bitumen Content	6.0%	6.0%	6.0%	6.0%	4.4%	5.9%
Resilient Modulus	1,535 MPa	3,247 MPa	5,918 MPa	3,400 MPa	6,100 MPa	1,691MPa

AB5 PMB contains SBS to DMR (Qld) MRS11.18 Classification A5S (AUSTROADS A15E)

Added filler is 6% by mass to give a total filler content of 10% by mass

Table 9.1 - Resilient Modulus for various Queensland mixes (After DMR (Qld) 2001a)

Similarly, it would be expected that for the SMA10 mixes with Class 320 bitumen, the mix with ultra fine dust (U.F.D.) filler would have a greater modulus than the mix with fly ash filler. The data in table 9.1 shows the reverse however that the flexural stiffness testing discussed in Chapter 7 does show the expected trend with some “scatter” in the data. Flexural testing is considered more reliable than the indirect tensile test for the measurement of stiffness (Read and Brown 1996). In this case, the flexural testing covers a greater number of samples compared to the resilient modulus testing which also contributes to its reliability. The results form the resilient modulus testing fit within the range identified from the scatter in the flexural stiffness testing. The sample with U.F.D filler has a slightly higher voids content compared to the fly ash filler sample which may have contributed to a lower modulus value.

The effect of adding lime to increase the stiffness of asphalt has been long recognised (NAASRA 1994a). The magnitude of the increase with increasing percentages of lime varies between aggregate sources (Stroup-Gardiner et al 1988). The use of lime filler has been reported to increase the stiffness by around 20% for various mix types (Baig and Wahhab 1998, Ishai and Craus 1996) whereas the results for SMA10 with Class 320 bitumen shown in Table 9.1 shown an increase of around 78% when lime filler is used in preference to the U.F.D. and fly ash fillers. The high proportion of filler in the SMA would be expected to contribute to the increased stiffness, however the flexural stiffness testing discussed in Chapter 7 shows an increase in the vicinity of 18% which is more consistent with published results. The limited resilient modulus testing reported in Table 9.1 may be giving unrepresentative differences in the stiffness values for the various mix types.

It is reported that mixes with larger maximum sized particles will tend to have greater resilient moduli than mixes with smaller particles (APRG 1997a). The modulus values from the SMA14 and SMA10 with PMB show this trend. It is also reported that the quantity of binder will also affect the resilient modulus. Within the normal range of binder contents (typically 3% to 10% by mass) the higher the bitumen content, the lower the resilient modulus of the mix will be (APRG 1997a). By comparing the SMA10 with Class 320 bitumen and U.F.D. filler with the DG14N as shown in Table 9.1, the bitumen content decreases from 6.0% to 4.4% and there is an increase in the maximum particle size. There is an increase in modulus from 3,247 MPa to 6,100 MPa due to the combined effect of these changes which is consistent with the expected trend.

Maccarrone et al (1997b) reported the following resilient modulus values for:

- AC14 (4.6% bitumen and 5.2% voids) 5,250 MPa
- SMA-14 (6.0% bitumen and 4.3% voids) 4,500 MPa
- CSRE-1 (6.8% bitumen and 5.1% voids) 3,570 MPa

Due to the low voids content for the SMA-14, a lower modulus might be expected at 5.0% voids. These results show a 370 MPa reduction in resilient modulus for each 0.5% increase in bitumen content. A typical upper limit for resilient modulus for a dense graded asphalt with Class 320 bitumen is 5,000 MPa (APRG 1997a) however the value is dependent on the mix specification used (AUSTROAS 1992a). The value of 6,100 MPa quoted in Table 9.1 is higher than typically adopted for a Queensland mix (AUSTROADS 1992a) however may be a function of the crushed rock fines an filler used for this mix. A value of 4,527 MPa has been quoted for DMR(Qld) DG14 with Class 320 bitumen however no details are given of the fine aggregate and filler type used (Frederick 1999). A typical DG14 in the Brisbane area would contain some natural sand as fine aggregate and fly ash filler which could account for the lower value. Testing of samples from North Queensland have been performed previously through the normal state wide asphalt mix design approvals. Similar results are evident.

9.4 Effect of Temperature

The resilient modulus significantly decreases as temperature increases (Barksdale et al 1997, di Benedetto and de la Roache 1998). AS/NZS 2891.13.1 states that equations are available in the AUSTROADS Pavement Design Guide (1992a) to convert the resilient modulus at the standard settings to the resilient modulus for the particular field environment in respect to temperature and load time. AUSTROADS (1992a) states that because of the many factors which may affect the result, it is recommended that whenever possible the values to be used for design be determined by testing a sample of the proposed mix, under conditions of temperatures and loading similar to those expected in the field.

However, AUSTROADS (1992a) does include the “Shell Method” nomographs to obtain an estimate of the modulus. These are included in AUSTROADS (1992a) as Figure 6.6 (Van de Poel nomograph for bitumen stiffness) and Figure 6.7 (Bonnaure et al nomograph for asphalt stiffness). It is important to note that the penetration or viscosity data used as inputs to the nomographs are the values obtained for bitumen which has been subjected to the Rolling Thin Film over (RTFO) test. Other similar methods of relating bitumen stiffness to mix stiffness are given in Roberts et al (1996).

These methods of estimating the mix stiffness use inputs such as the percentage of bitumen, aggregate and voids – all by volume. As shown in Table 9.1, the type of filler significantly impacts on the mix stiffness however is not considered by these methods. For low bitumen stiffness, e.g. higher temperature situations, the actual mix stiffness depends largely on the aggregate properties, particularly the angularity of the aggregates (AUSTROADS 1992a). In addition, Woodside et al (1998) demonstrated that the addition of 0.3% of loose fibres increased the stiffness by 48% compared to an increase of 10% when palletized fibres were used. The effects of fibres are not considered by the various nomograph methods. Use of these nomographs to estimate the effects of temperature is questionable. This is further complicated in Queensland where SMA typically contains PMB which means that the mixes would be expected to perform differently to the mixes produced with conventional binders.

9.5 Investigation of Temperature Effects

Given that the nomograph methods were developed using well compacted dense graded asphalt (Roberts et al 1996) and the effects of filler type and fibres on the modulus, suggests that the methods may not be applicable to SMA. Stephenson (2002) constructed a test plan which was designed to investigate the temperature susceptibility of three SMA mixes. Mixes were made to the centerline grading of SMA10 from DMR (Qld) MRS11.33 and APRG (1997a). The dilation point grading approximated the AAPA (2000a) SMA10 grading. Samples were prepared at four different bitumen contents, initially to investigate the sensitivity of voids content to changes in bitumen content for each mix type. After conditioning for one hour at 150°C, the samples were compacted with 120 cycles of the "Gyropac". The resilient modulus of each mix was determined at 10°C, 20°C, 30°C and 40°C so that the effects of temperature could be investigated.

Figure 9.2 summarises the resilient modulus testing for the APRG (1997a) SMA10. It would be expected that resilient modulus will decrease with increasing bitumen content and increasing voids content (APRG 1997a). For the APRG (1997a) mixes the following observations can be made. APRG1 has the highest bitumen content (6.5%) and the second highest voids content (8.4%) which is consistent with it having the lowest resilient modulus of the four APRG (1997a) samples. Of these mixes, APRG3 has the highest resilient modulus which is a reflection of it having the lowest voids content (5.4%) and the third lowest bitumen content (6.0%). The higher resilient modulus of APRG4 (5.3% bitumen, 9.2% voids) compared to APRG2 (6.2% bitumen, 8.2%voids) shows that the 0.9% reduction is bitumen content has a greater effect on increasing stiffness than the stiffness decrease due to the 1.0% increase in voids content.

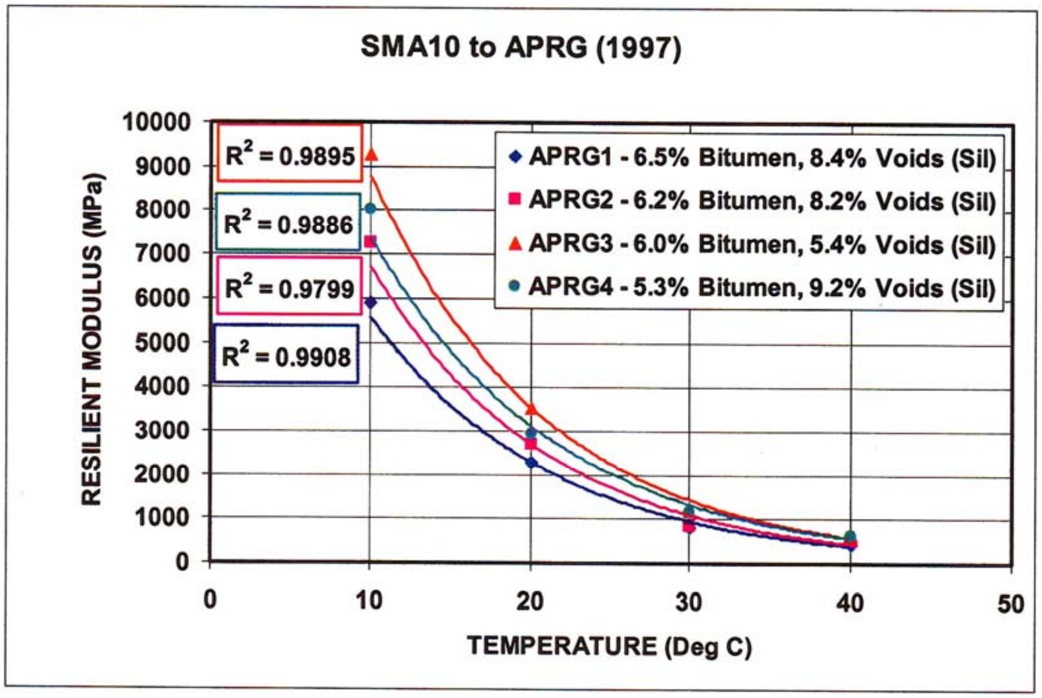

Figure 9.2 – Effect of temperature on resilient modulus of SMA 10 manufactured to APRG (1997a)

Figure 9.3 summaries the resilient modulus testing for the AAPA (2000a) SMA10. The narrow spread of results suggests that the stiffness of the AAPA (2000a) mixes are less sensitive to changes in bitumen and voids contents however, it is also a reflection of the small range of bitumen contents (5.8% to 6.75%) and voids contents (6.3% to 7.8%) tested. Samples AAPA1 and AAPA3 both achieved 6.3% voids content. As expected, AAPA1 with 6.75% bitumen has a lower resilient modulus than AAPA3 with 6.0% bitumen content. AAPA3 (6.0% bitumen, 6.3% voids) and AAPA4 (5.8% bitumen, 7.8% voids) have similar bitumen contents and the lowest resilient modulus value is from the sample with the highest voids content.

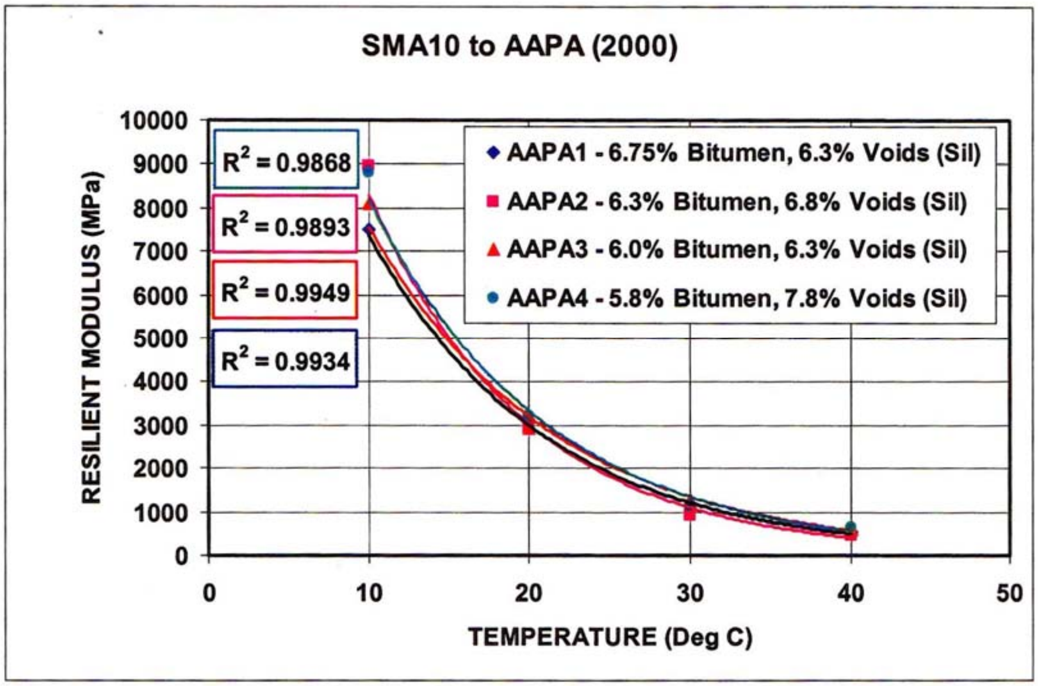

Figure 9.3 – Effect of temperature on resilient modulus of SMA 10 manufactured to APRG (2000a)

Figure 9.4 summaries the resilient modulus testing for the DMR (Qld) MRS11.33 SMA10. For the DMR(Qld) MRS 11.33 mixes at 20°C there is very little difference in resilient modulus for samples QLD2 (5.9% bitumen, 9.3% voids) and QLD1 (6.6% bitumen, 8.0% voids) suggesting that the softening effect of increased bitumen is balanced by the stiffening effect of reduced voids. QLD4 (5.3% bitumen, 11.3% voids) has the lowest resilient modulus which is a reflection of the high voids content however, it would have been expected that the low bitumen content may have resulted in a higher resilient modulus particularly when compared to QLD3 (5.5% bitumen, 10.3% voids) which has the highest resilient modulus.

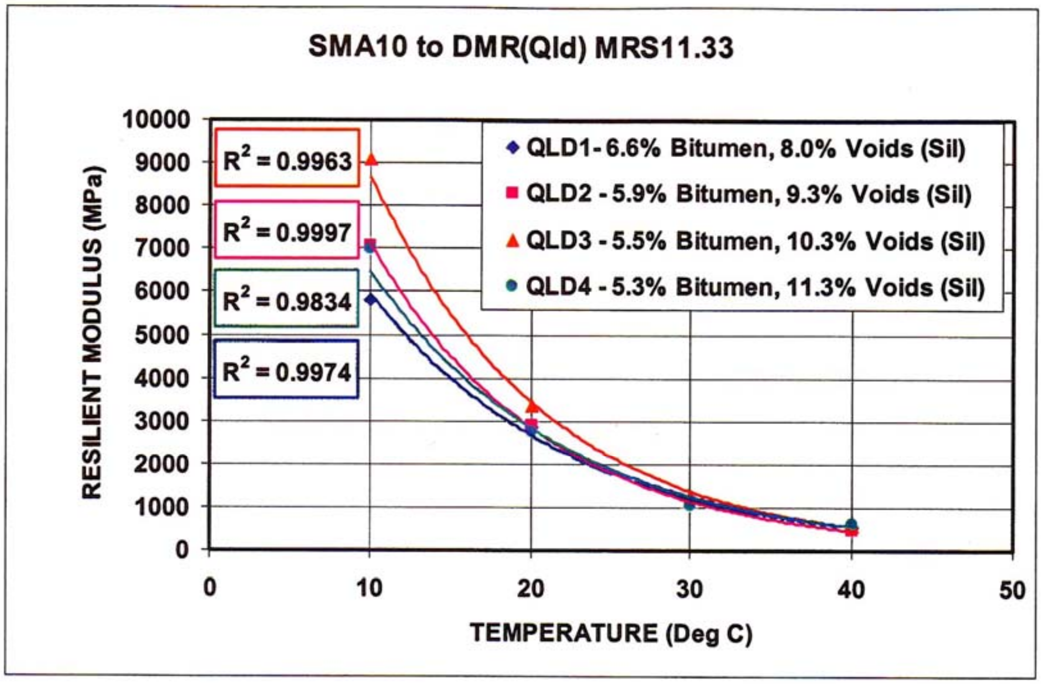

Figure 9.4 – Effect of temperature on resilient modulus of SMA 10 manufactured to DMR (Qld) MRS 11.33

From Figures 9.2, 9.3 and 9.4, it can be seen that there is a very strong relation between resilient modulus and temperature with r^2 values of 0.98 or greater. At a common temperature (say 20°C) for a given mix (say APRG 1997a), there are dual effects on the resilient modulus of varying bitumen contents and voids contents.

To assess the effect of voids on stiffness, the mixes were grouped based on bitumen content as shown in Figure 9.5. Figure 9.5(a) shows mixes with 6.5% to 6.7% bitumen content and the resilient modulus values are ranked in reverse order to the voids content. Similar trends are shown in Figure 6.5(b) (5.9% to 6.0% bitumen) and Figure 6.5(c) (5.3% bitumen). The influence of the aggregate grading on resilient modulus, at constant bitumen content, is in its affect on the voids content of the compacted mix.

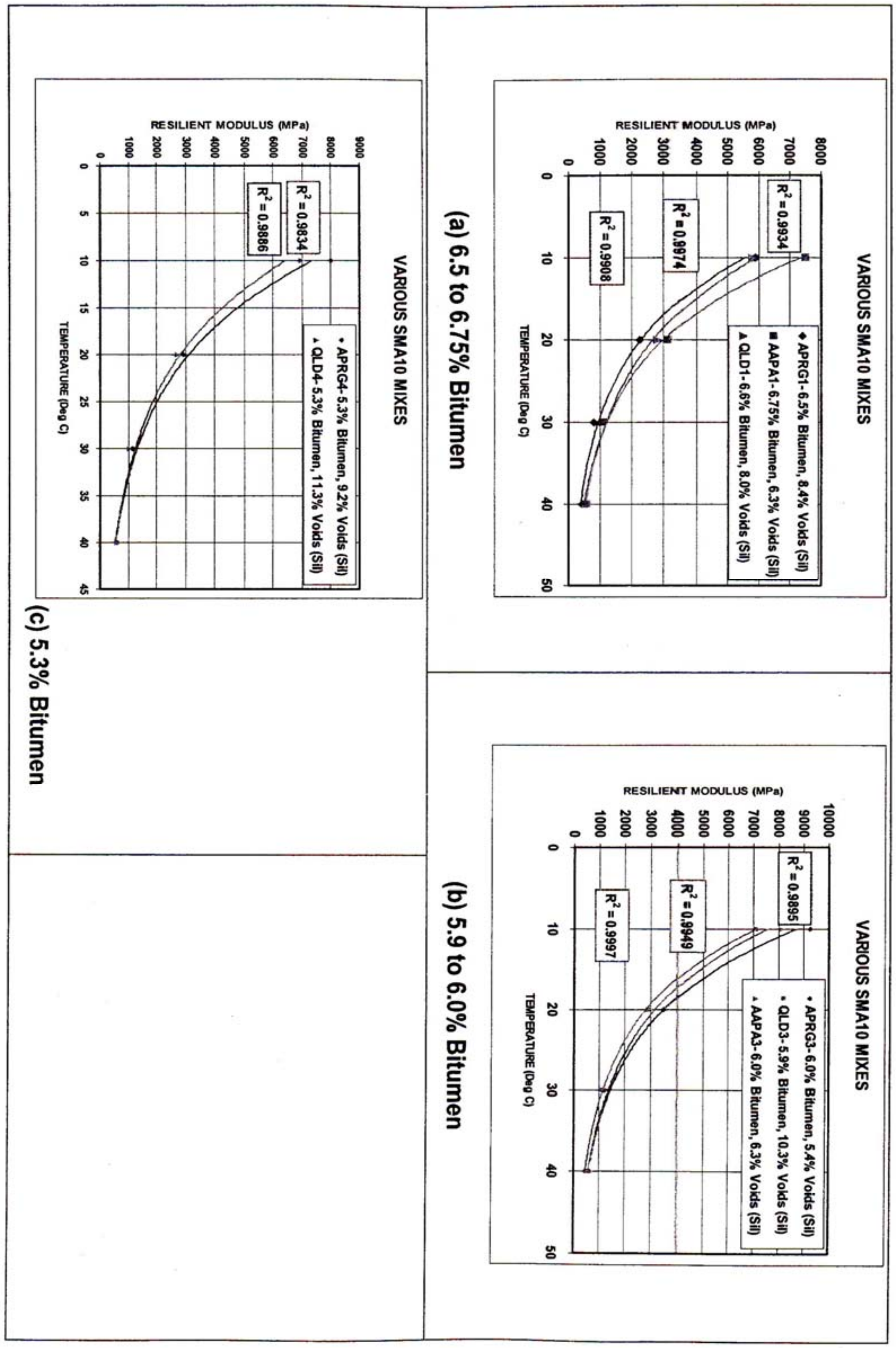

Figure 9.5 – Effect of voids content on different SMA10 mixes with similar bitumen content.

The statistical analysis software SPSS (SPSS Inc 2000) was used by Stephenson 2002 to investigate relationships between resilient modulus as the dependant variable and the independent variables of bitumen content, voids content and temperature for the three mixes as well as combined data set made up of the three mixes. The models returned from this analysis and their R^2 values are shown in Table 9.2. Separate models were also investigated using temperature as the single independent variable and are shown in Table 9.3. Comparison of the R^2 values between the models in Tables 9.2 and 9.3 shows that temperature is the dominant independent variable in predicting resilient modulus and the inclusion of additional independent variables does not significantly improve the precision of the predicted resilient modulus values.

Mix Types	**Regression Equation**	**R^2 value**
APRG (1997a)	Log_{10} RM = 4.847 – 0.0388 Temp – 0.102 % Bit	0.967
AAPA (2000a)	Log_{10} RM = 4.631 – 0.040 Temp – 0.0546% Bit	0.981
DMR(Qld) MRS11.33	Log_{10} RM = 4.4.56 – 0.0377 Temp – 0.041% Bit	0.981
Combined Data	Log_{10} RM = 4.569 – 0.0388 Temp – 0.0536% Bit	0.973

Table 9.2- Resilient Modulus regression equations incorporating temperature and bitumen content as independent variables

Mix Type	**Regression Equation**	**R^2 value**
APRG (1997a)	Log_{10} RM = 4.232 – 0.0388 Temp	0.956
AAPA (2000a)	Log_{10} RM = 4.292 – 0.0400 Temp	0.979
DMR(Qld) MRS11.33	Log_{10} RM = 4.217 – 0.0377 Temp	0.978
Combined Data	Log_{10} RM = 4.247 – 0.0388 Temp	0.970

Table 9.3 - Resilient Modulus regression equations incorporating temperature as the independent variable

The equations included in Table 9.3 have been used to predict the resilient modulus of each mixes at a range of temperatures as shown in Table 9.4. These results are also shown in Figure 9.6.

TEMP (°C)	APRG (1997a)	AAPA (2000a)	DMR (Qld)	ALL DATA
10	6,982	7,798	6,918	7,228
20	2,858	3,105	2,904	2,958
30	1,169	1,236	1,219	1,211
40	479	492	512	495

Table 9.4 - Predicted Resilient Modulus based on temperature

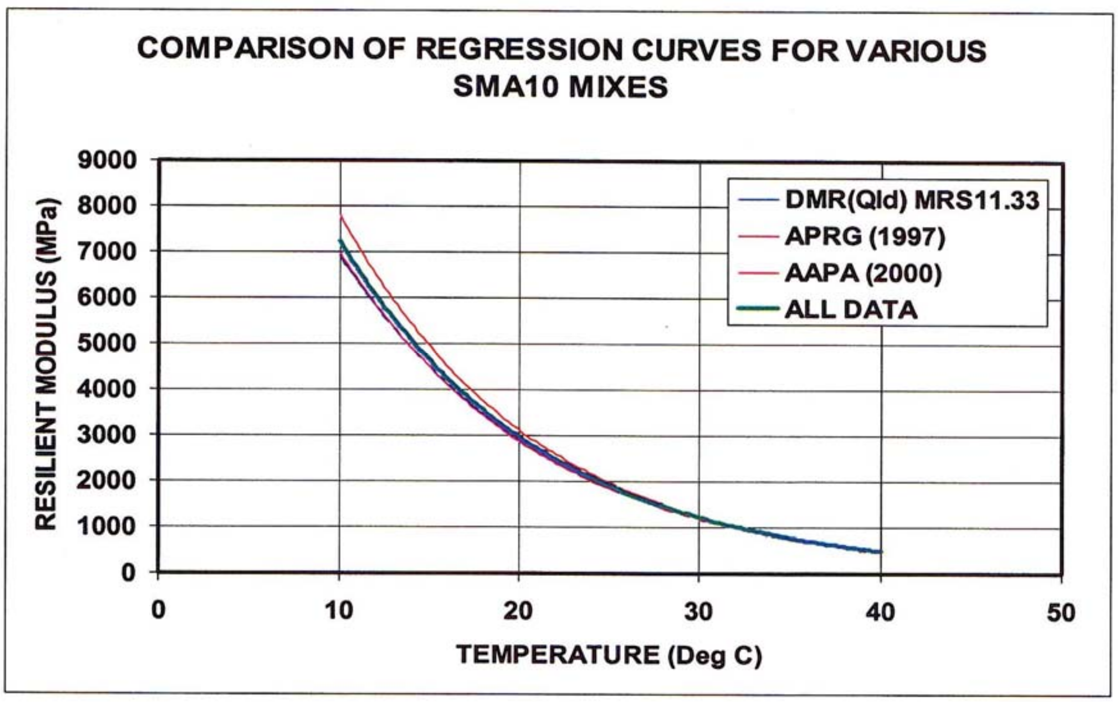

Figure 9.6 - Predicted Resilient Modulus based on temperature

From Table 9.4 and Figure 9.6, it can be seen that for typical pavement temperatures in Queensland (>20°C) all equations predict similar values of resilient modulus. It is proposed that the equation prepared using all data be used to predict the resilient modulus of SMA10 manufactured using Class 320 bitumen, 0.3% cellulose fibres and fly ash filler. Given that higher fibre contents and alternative fillers will result is stiffer mixes, the proposed relationship will give a lower bound estimate of the effect of temperature on resilient modulus.

Stephenson (2002) proposed a relationship as follows:

$$RM = 10^{(4.247-0.0388\ Temp)}$$

Similar testing is being performed in a continuous effort to endorse the use of SMA 12mm mix in North Queensland. The results of testing will use the format of the research data from above, and will be discussed in Chapter 7.

9.6 Summary

Testing of a range of laboratory and plant produced SMA10 highlighted the effect of filler type, incorporation of fibres and type of binder on the mix stiffness. The use of simple nomographs such as included in AUSTROADS (1992a) are not appropriate for estimating SMA stiffness as the effects of these significant mix component are not included. The effects of temperature on mix stiffness can be determined using equipment such as the MATTA.

An investigation was undertaken into the effects of changes in the SMA10 grading by considering samples manufactured to the gradings of APRG (1997a), APPA (2000a) and DMR (Qld) MRS11.33. It was found that temperature has the most significant effect on the asphalt stiffness however, for constant bitumen content, the increase in stiffness was proportional to a reduction in voids content. The type of aggregate grading chosen does not greatly affect the stiffness rather, the influence of the aggregate grading on resilient modulus, at constant bitumen content, is in its affect on the voids content of the compacted mix. At typical Queensland in-service pavement temperatures, all SMA10 types with Class 320 binder produced similar resilient modulus values.

CHAPTER 10 – FATIGUE PROPERTIES OF SMA MIXTURES

10.1 Introduction

This chapter discusses the research undertaken to determine the influence of the binder and filler components of the mastic on the fatigue properties of SMA mixes. Much of the research has been developed by AAPA and their associated senior officers, other related technical boards, and other Papers such as Stephenson's Report.

Investigations into fatigue life were undertaken and the research involved a typical Queensland Department of Main Roads Stone Mastic Asphalt with a Polymer Modified Binder (PMB) containing Styrene-Butadiene-Styrene (SBS) co-polymer and a conventional binder (Class 320 bitumen). For the SMA with Class 320 bitumen binder three different mineral fillers were used. These were Ultra Fine Dust, Hydrated Lime and Fly ash which are typical of the fillers used in Queensland; the actual choice normally being made on regional availability. The investigation also included a typical Dense Graded Asphalt mixture with Class 320 bitumen binder as this is the mix routinely used through out the State of Queensland.

This chapter also presents new derived fatigue relationships that can be used for the design of pavements incorporating SMA layers.

10.2 Experimental Work

To investigate the fatigue properties of Stone Mastic Asphalt, the prior research involved the designing of a laboratory test program. The mix investigated was a centre-line grading complying with Queensland Department of Main Roads Standard Specification MRS11.33 "Stone Mastic Asphalt Surfacing" (DMR (Qld) MRS11.33). The binder content remained constant at 6% by mass of the total mix including the binder and 0.3% by mass of cellulose fibres were added. To assess the impact of the binder on fatigue, a polymer modified binder and a conventional binder (Class 320 bitumen) were used with the same grading and mineral filler. The polymer modified binder selected was BP "AB5" incorporating Styrene-Butadiene-Styrene (SBS) co-polymer. It was manufactured to comply with the A5S Classification of DMR (Qld) MRS11.18.

To assess the impact of the filler on fatigue, three different mineral fillers were used with the same binder (Class 320 bitumen) and grading. The fillers used were Ultra Fine Dust – quarry manufactured fine material – Hydrated Lime and Fly ash. These fillers were selected because they are typical of those used in Queensland with the actual choice depending on regional availability. All mixes used the same quantity (6% by mass) of added filler. To allow comparison of the fatigue of SMA with a conventional Queensland Department of Main Roads mix, a 14mm Dense Graded mix to Standard Specification MRS11.30 "Dense Graded Asphalt Pavements" was included in the fatigue programme. The DG14 mix was chosen because it has historically been used in the applications where SMA10 is now specified. Figure 10.1 shows the centerline grading curves for the SMA10 and DG14 mixtures used in the fatigue testing comparisons.

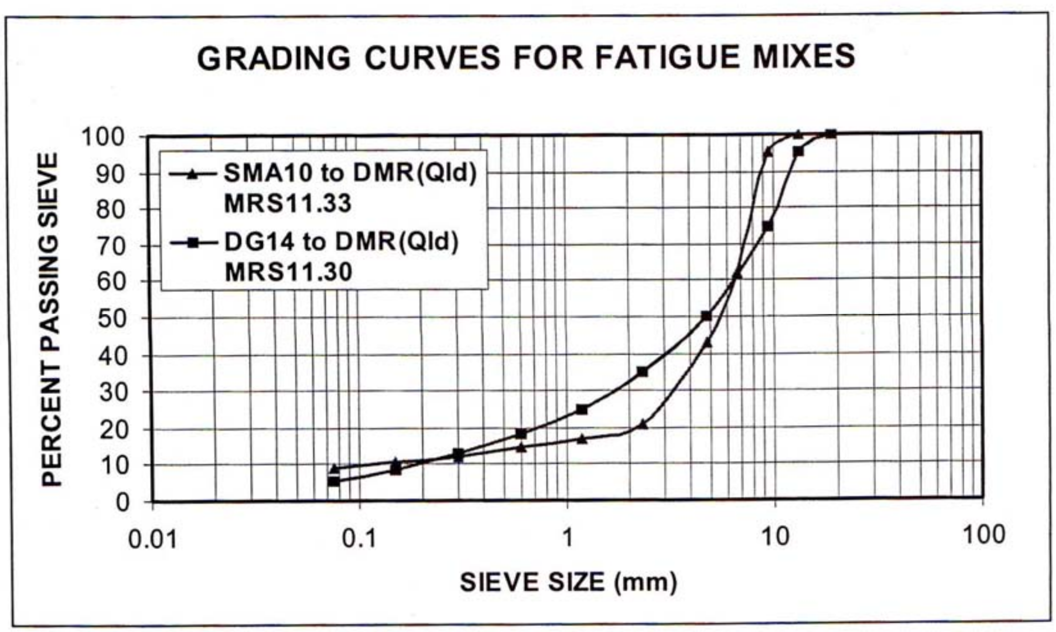

Figure 10.1 – Grading Curves for SMA10 and DG14 Mixtures

The mixes were produced at the Herston laboratory of the Queensland Department of Main Roads. After mixing, the mix was conditioned for 60 minutes at 150°C before slabs were compacted using a BP Slab Compactor. After the compacted mix cooled, 50mm high x 65mm wide x 390mm long beams were cut from the slabs. These are nominal measurements and actual dimensions were recorded and used as the input into the fatigue testing software. The bulk density of the beams was determined by the water immersion method (DMR (Qld) Q319, Q320) which allowed the voids content (DMR (Qld) Q311) to be calculated (AUSTROADS AST03:1999).

The beams were tested in a Four Point Bending Fatigue Test Apparatus, manufactured by the Australian company Industrial Process Controls (IPC). Test procedures were generally in accordance AUSTROADS Provisional Method AST03:1999 however a range of strain levels were investigated so that robust fatigue relationships could be developed. All testing was undertaken at 20°C ± 0.5°C and used the controlled strain mode of loading with a haversine wave form selected. Initial stiffness was taken as the stiffness after 50 loading cycles and fatigue defined as the point where the stiffness reduced to 50% of the initial value.

Series	**Type**	**Binder**	**Filler**	**Number of Beams**	**Voids content**
I	SMA10	AB5 PMB	U.F.D.	17	3.5% to 7.3%
II	SMA10	C320 Bitumen	U.F.D.	24	3.3% to 7.7%
III	SMA10	C320 Bitumen	Lime	24	4.6% to 8.4%
IV	SMA10	C320 Bitumen	Fly ash	30	2.8% to 8.1%
V	DG14	C320 Bitumen	U.F.D.	12	4.2% to 5.7%

Note: U.F.D. = Ultra Fine Dust

Table 10.1 - Schedule of Fatigue tests showing binder and filler types

As shown in Table 10.1, beams with a range of air voids contents were tested so that the influence of voids on fatigue life could be examined. The range of voids contents came from the beam cut from the compacted slabs. The outer beams exhibited the highest voids contents as was identified by Maccaronne et al (1997a). Given that the materials used in each mix type remained constant, the different air voids contents are a result of different compaction levels and can be used to gain an appreciation of the effect of compaction on the fatigue properties.

10.3 Test Results

10.3.1 Analysis of Test Results

Plots were produced for tensile strain, flexural stiffness, cumulative dissipated energy and core temperature. The tensile strain and core temperature plots were produced to ensure that the specification conditions were being satisfied. Some plots were also produced for phase angle. The flexural stiffness and cumulative dissipated energy plots were used to estimate the fatigue properties of the mixes.

10.3.2 Effect of Voids Contents

The influence of the mix design parameters on the fatigue life has been the subject of many studies. The feature common to these studies has been the attempt to predict the fatigue behavior of the mixes from their compositions. An analysis of the influence of the each of the parameters is difficult due to their interdependencies (di Benedetto and de la Roche 1998). Due to the influence of flexural stiffness on the fatigue life, those parameters that have the most influence (air voids, binder type and content/film thickness and its rheological properties, and aggregate type and grading (Baburamani 1999)). The literature reviews of the detailed research in this area demonstrates the relationship between initial flexural stiffness and air voids, binder type (Class 320 bitumen and SBS PMB), filler type, and aggregate grading (SMA10 and DG14). The results for the fatigue life production models were prepared on the basis of incorporating the testing of all voids contents.

10.4 Implications for the Design of the SMA Mastic

Given the strong relationship between stiffness and fatigue life, the change in stiffness makes significant differences in fatigue life. The testing programme has shown that the type of filler and binder used in the SMA significantly affects the elastic properties and hence the fatigue life of the mix.

For the mixes containing Class 320 bitumen, the higher fatigue life measured for the SMA mixes confirms the claim that the higher bitumen content of SMA10 compared to DG14 results in greater fatigue life (AAPA 2000a). The change in grading to allow the higher bitumen content of SMA10 compared to DG14 results in greater fatigue life (AAPA 2000a). The change in grading to allow the higher bitumen content resulted in a 10 fold increase in fatigue life.

The choice of binder has dramatic implications on the fatigue life. For the PMB used in this research, it resulted in a 100 fold increase in fatigue life. Of the SMA10 mixes containing the same binder, the fatigue life is a function of the filler type. Lime filler produced the shortest fatigue life, followed by ultra fine dust and fly ash.

The testing machine requirements to measure phase angle as contained in AUSTROADS AST 03:1999 are not appropriate for high strain levels e.g. greater than 600$\mu\varepsilon$ or for higher bitumen contents such as typically encountered in SMA.

10.5 Summary

The results of the experimental programme to investigate the fatigue life of SMA has been summarised and discussed. Strain based fatigue life prediction equations have been analysed by others for a number of SMA10 mixes manufactured to DMR (Qld)

MRS11.33 as well as DG14 manufactured to DMR (Qld) MRS 11.30.

The testing programme researched showed the fatigue life benefits of using SMA in place of DG asphalt however it must be appreciated there will be a corresponding reduction in the stiffness of the mix. The fatigue life benefits of using the SBS-PMB were clearly demonstrated however this comes with the sacrifice of significantly reduced stiffness. The reduction in stiffness when using the SMA10 and/or SBS-PMB will need to be considered as part of any pavement design process.

The testing has also quantified the effects on stiffness and fatigue life of commonly used fillers. This has implications for generic type specifications where a variety of fillers are permitted in nominally the “Same” mix.

CHAPTER 11 – CONTINUED USE OF SMA

11.1 Introduction

The previous two Chapters considered aspects of SMA that are important for the analysis and structural design of pavement layers. One of the fundamental serviceability requirements of asphalt mixes, especially surfacing layers, is resistance to permanent deformation (rutting) under applied traffic loads. The investigation into deformation resistance comprised of two parts namely:

- Part A Development of new fundamental test method to assess the effects of the stone skeleton; and

- Part B Assessment of the effects of different fillers and binder

11.2 Advantages and Disadvantages of SMA

ARRB Transport Research and Austroads reported the advantages and disadvantages of SMA in a Technical Note 16. The advantages were listed as:

- SMA provides a textured, durable and rut resistant wearing course.

- The surface texture characteristics of SMA were similar to OGA so that noise generated is lower than that on DGA but equal to or slightly higher than OGA.

- SMA can be produced and compacted with the same plant and equipment available for normal hot mix (DGA), using the above procedure modifications.

- SMA may be used at intersections and other high traffic stress situations where OGA is unsuitable.

- SMA surfacing may provide reduced reflective cracking from underlying cracked pavements due to its flexible mastic.

- The durability of SMA should be equal, or greater than, DGA and significantly greater than OGA."

Point three, in the above list, states that a modified procedure should be used. For SMA, the recommended method of compaction is to use heavy steel rollers with limited vibration. If a significant amount of vibration were to be used, then the stone and the mastic would separate with the mastic coming to the surface.

The majority of respondents in the asphalt industry provided the same list of advantages and emphasised an effective texture together with a long life and durability as the prime benefits. In addition, other respondents stated that:

- There us a reduction in the water spray from a wet surface with SMA when compared with the spray from a DGA.

- The greater durability enables SMA to have a longer life and reduced maintenance and whole-of life costs.

- When it comes to be refurbished, provided the SMA surfacing has not been infiltrated with water, it need not be removed when resurfacing. In contrast, and OGA is designed to be porous and will generally always be removed before the pavement is resurfaced.

The disadvantages of SMA were stated in the ARRB Transport Research – Austroads Technical Note 14 as: · Increased material cost associated with higher binder and filler content.

- Increased mixing time and time taken to add extra filler may result in reduced productivity.

- Possible delays in opening (the roads) to traffic as the SMA should be cooled at 40 degrees C to prevent flushing of the binder to the surface.

- Initial skid resistance may be low until the thick binder film is work off the surface by traffic. In critical situations, a small, clean grit may need to be applied before opening to traffic."

Point four above indicates that the initial skid resistance could be low. SMA when constructed has a film of mastic over its upper surface. Under the action of traffic this film is removed and the skid resistance increases. Typically a lower speed limit and warning signs are installed during this period.

Again the respondents provided other disadvantages. These were:

- SMA requires more attention to detail when mixing and being produced transported and placed in the field. However, once the expertise and capability is obtained, the surfacing is no more difficult to place than other pavements.

- The higher temperatures of the asphalt, when polymers are used, may limit the distances the material can be effectively transported.

11.3 Summary of Requirements

With there being so many possibilities with a mix design for asphalt we need to continually focus on the specific directions of what we need to achieve.

The most important performance requirements are:

- Resistance to deformation
- Resistance to fatigue cracking
- Resistance to reflection cracking
- Durability – resistance to fretting

Deformation resistance of asphalt can be achieved through a number of different ways:

- Mix design (mechanical interlock of aggregate skeleton)
- Binder type (binder grade / PMB)
- Binder Quantity
- Aggregate type (mechanical strength)
- Aggregate fines (internal friction of mixture)
- Filler (quantity and type)

Fatigue and cracking resistance of asphalt mixtures are mainly controlled by characteristics associated with the binder:

Volume of binder in the mixture (Vb) The greater the quantity of binder, the greater the resistance to crack propagation

Elasticity of the binder, PMBs made with elastomeric polymers can have an enormous influence on improving fatigue characteristics

Fretting distress is normally associated with open grade type mixtures and thin surfacings.

Fretting is usually a function of the adhesion between binder and aggregate, or the cohesive properties of the binder.

Both attributes can be greatly improved by the use of PMBs

Open grade and thin surfacings normally use modified binders

11.4 Where SMA should or should not be used?

SMA is not considered to be the only surfacing alternative and respondents considered that the pavement selection guidelines should be consulted.

Generally, SMA is ideal for highways as its strength makes it more resilient to rutting. It is particularly useful when there has been some cracking of a previous (lower) surfacing. SMA with its higher bitumen content is a more flexible surfacing and this allows it to accommodate more movement. As a result SMA decreases the tendency of cracking in the lower pavement layer reflecting (or propagating) through and affecting the SMA surface. Other surfacings like DGA do not have this capability. The ability to reduce cracking provides for a longer pavement life.

SMA is not suitable for small areas or for areas where the asphalt laying plant has restricted access. SMA is very stiff and less workable than other asphalts. This stiffness makes it harder to compact particularly if modified binders have been used. The addition the use of modified binder decrease the time available for compaction. Construction crews, plant and supervision must develop skills for the effective placement and compaction of SMA. If these skills are not available then SMA should not be used. Because SMA is difficult to compact by hand, it is not suitable for small or restricted areas.

SMA is constructed with the significant stone on stone contact providing considerable compressive strength. It is also constructed with a textured surface where the stones on the upper surface stand proud of the mastic. This means that the stones are less resistant to sideway (shear) forces on the surface caused by trucks making tight turns. In these cases the tyre – surface forces can cause the "proud" stones to "roll out" and leave the binder behind. SMA is often not recommended at smaller roundabouts and DGA may be more appropriate. It is emphasised that any surfacing should not be considered to be a "universal" solution for all conditions and local knowledge and experience should be used to select the most appropriate mix.

SMA and OGA have been developed to provide an effective surface texture. This is a prime safety requirement and helps to maintain skid resistance at the higher speeds. The texture is also useful in decreasing the water depth on the surface. These qualities make for safer roads. Skids resistance is a function of the mircotexture (or the roughness of the individual pieces of exposed aggregate) and the macrotexture (developed from the arrangement of the aggregate on the surface). These textures are shown diagrammatically in Figure 11.1 from the Austroads "Guidelines for the Management of Road Surface Skid Resistance". Both qualities are required to produce effective skid resistance at highway speeds.

Figure 11.1 – Microtexture and Macrotexture (Source: Austroads)

11.5 Modifications of the SMA to suit Queensland conditions

SMA is a generic name for a type of asphalt and is therefore not the same product for all

counties and regions. Small changes are possible and needed to enhance some characteristics and to make adjustment for differences in material properties and climates. The SMA used in Queensland was similar but not identical to the design used in Germany and other parts of the world.
The Department of Main Roads has pursued SMA pavements with increased average surface texture depth. This feature enhances the safety of the motorist, as the texture depth increase macrotexture, which helps to maintain skid resistance at higher speeds and better controls spray from vehicles. These features constitute significant benefits to the community.

Main Roads pavement specialist sought industry and District involvement in the development of the specifications. It was deemed essential that the industry could provide a mix that met the specifications and had an opportunity to provide technical input to its development.

The modification and development of SMA requires studies of the performance of small specimens in the laboratory under controlled tests and also studies of the surfacing on the road. These field trials are important to understand the performance of the SMA surfacings and its associated material properties. Important aspects of these field trials are:

- That they are undertaken at sites similar to proposed installation;
- That there is a benchmark surfacing installed at the same time on the same section of road, and
- That the performance of the road sections be monitored closely.

Respondents from the industry, the specialist groups and the Districts have emphasised the importance of field trials in the development of SMA surfacings.

11.6 Consequential Effects

- The development of North Queensland Specifications that is different to the National Specifications, particularly due to the climatic conditions.
- Manufacturing differences due to the difficulty in maintaining source rock specifications.
- Due to the necessary requirement to continue using SMA, we need to justify any changes for the North Queensland area.

11.7 Results and Correlations

The outcome of the result from the testing and analysis is which design criteria leads to which properties in service, and what is the failure mechanism of the surfacing. This is difficult to quantify is some respects due to variables such as climate, loading and existing surface which change with location.

The following be considered as a part of the Supplementary Procedure for Volumetric Design of SMA Based on "Determination of Dilation Point" contained in AAPA (2000a):

- The Aberg (1992a) particle packing equation be applied to the coarse aggregate grading as the first stage of the extended DPM process. The series of trial mixes be manufactured with increasing percentages of fine aggregate of this maximum size rather the 4.75mm maximum size used in the current procedure.

- After the trial mixes are compacted, each sample to be tested for resilient modulus and a plot of sample resilient modulus against % fines be prepared vs. the % passing 4.75 mm sieve. Separate the data into the two distinct groups through which straight lines can be drawn. The intersection point of these two straight lines becomes the dilation point.

- Similar plots of resilient modulus can be produced for % passing the other sieve sizes as shown in Figure 5.14 and used to determine the upper limits of the SMA grading curve.

The results from the filler/ binder ratio testing in Chapter 7 seem to correlate with where there are visual problems in the field. The excess free binder is more about the filler than merely just reducing the limits of bitumen content. SMA manufacture is such that the producing, including, and trapping of fines/ filler is an issue. It becomes a greater issue is North Queensland due to the nature of the existing older plants available for production.

With the field voids, the spread is too great per the project. Of the trial sections where there are problems, the areas of concern are patchy and not across the scope of the roadway. This may coincide with the fact that the ranges of air voids percentages are large. The exact locations of the results are not specific enough though to compare to where there is a high or low result from the filler/ binder ratio.

The correlation to be drawn is one of fines and binder. Binder is good for fatigue, but excess creates a 'fatty' mix. Not enough binder and the mix will internally unravel even if the field voids are within design range. Too much fines, and the air voids increase and this creates excess binder. The stone skeleton design of SMA certainly requires an intricate element of design testing.

11.8 Economic Analysis

There are several methods for economic comparison of alternative treatment types. The

Present Worth methods is used as it effectively allows for both uniform series and sporadic events, (e.g. routine and period maintenance) which will occur during the service life of the pavement. With the Present Worth method, all costs are converted into capital sums of money which, invested now for an analysis period, would provide the sums necessary for construction of a project and subsequent maintenance during that period. (AUSTROADS 1992a).
The criterion of Nett Present Worth (NPW) can be used to compare mutually exclusive projects with unequal time spans. However, NPW amounts indicate the projects’ costs and earnings over their duration, and where these durations are different, NPW are not directly comparable. An assumption must be made, that investment opportunities equivalent to those under comparison will continue to be available and be accepted, and at least until a common multiple of project durations is reached. Repetitions of the project cash flows must then be simulated until this common multiple of project durations is reached, and the NPW of these multiple projects, of equal duration, can then be compared (McAnally and Iliff, 1984).

The concept of “whole of life” costing (WHOLC) using the NPW of the construction, maintenance and road user costs and any residual (salvage) value is gaining increased acceptance. As an example, the review of pavement types for Pacific Highway Upgrade (Wallce et al 1996) used a similar approach which has now been extended into the DMR (Qld) (1998) WHOLC guidelines for heavy duty pavements. Joseph and Bullen (1999) incorporated the probability estimates for all alternatives being investigated. The process is referred to as P-WHOLC.

Whilst construction, maintenance and salvage cost are readily identifiable and can be easily quantified, road user costs are more intangible. Wallace et al (1996) found that the road user costs are by far the largest cost in any economic analysis of different pavement alternatives, as only a 1% difference in these user costs can be equivalent to the present worth of the full construction or maintenance cost. DMR(Qld) (1998) takes the approach that road user costs (Vehicle operating cost and normal travel time) for routine road operations may be excluded from the analysis as they are essentially similar for heavy duty pavement alternatives, provided minimum levels of serviceability are maintained.

By assuming that the road user costs are the same or negligible for the alternative pavement types, the concept of whole-of-life agency cost (WHOLC) is introduced (DMR(Qld) 2001c) which only includes the discounted sum of construction, widening and maintenance costs, minus salvage value. Whilst this approach is convenient for pavement designers, it does not appear to have total support within the Department (Vos 2002). The approach ignores operational impacts such as variable noise levels, skid resistance and slash and spray effects, such issues are gaining greater public attention (Jones 2002). It also ignores impacts and costs to the road users such as the frequency of required maintenance treatments and increased user costs due to reduced serviceability of the pavement over time.

One of the issues associated with WHOLC is the source of funding. Construction and maintenance costs are met from road authority budgets which are normally inadequate to satisfy all competing demands often leading to the choice of 'the lower cost", higher coverage option as the best network-level solution" (Vic Roads 1998). The realties of a limited budget can often mean that the best economic solution for an individual pavement section is not the lowest cost solution for the whole road network. Because the road user costs are borne by the individual user, they are obscured and any savings are not reflected as an increase in the road authority budgets. To transfer these savings to the road authority budget would mean an increase in either direct or indirect taxation which would be expected to be politically unpalatable.

The most obvious of road user costs is the delays associated with maintenance or rehabilitation. These costs may be significant on roads with high traffic volumes with the additional road user costs being estimated from road works delay modeling. As an example, for a 4 lane section of the Bruce Highway, DMR (Qld) (2001c) assumed that night maintenance would be undertaken between the hours of 6pm and 6am which would result in insignificant maintenance delay costs so they were not considered further in their analysis. This simplistic assumption may be inappropriate for the concrete pavements option because typical maintenance diaries for concrete pavements (Wallace et al 1996) including digging out, replacing and curing time for slab replacements would be expected to require closure for longer periods.

The complete economic analysis of all alternative pavement types is beyond the scope of this thesis. The foregoing discussion has highlighted some of the principles that have been used and items that can be considered. For the purpose of this thesis, the life cycles aspects are discussed in the context of substituting SMA for conventional AG asphalt as a surfacing.

11.9 Summary

The continued use of SMA in the Northern region under tropical climatic conditions will depend on the balance of alliance type discussions between DMR (MRD) Peninsula and the local manufacturing and laying companies. The guidelines are becoming clearer and more identifiable as we progress, and even though criteria remain the same the limits are being tightened. Methods to convert the stiffness and fatigue relationships derived under standard test conditions to actual pavement loading and temperature regimes were developed.

It was found that the design stiffness values currently adopted to DMR (Qld) for their SMA14 are significantly higher than those reported elsewhere and are inappropriate for the service temperature conditions. This is of concern for any pavement studies because the pavement design relies on regular overlays to increase the pavement strength during the life of the pavement. This will not be such an issue where the surfacing layer is considered as a non-structural layer.

The difference between overlay design by the AUSTROADS (1992a) and mechanistic methods is an issue that remains unresolved at the National level (Jameson 1996). The AUSTROADS (1992a) method is applicable to dense graded asphalt overlays and the development of appropriate curvature functions for SMA/PMB for overlay design is an area where further research is justified.

The economic analysis shows that where increased life of the total pavement structure can be expected from the overlay using SMA/SBS, then there is sound justification for using the more expensive product.

To summarise the filler/ binder and voids testing part of the Chapter, we need to reflect on the actual level of current testing. Analysis tables such as the one in section 7.5 give us a clear indication of the sections and areas where there are problems, and allow the correlation of these distinct locations against other design criteria.

CHAPTER 12 – CONCLUSIONS AND RECOMMENDATIONS

12.1 Introduction and Overview

The introduction of SMA into many marketplaces has been undertaken against a background of the change from empirical mix design methods to performance – based and performance – related mix design methods. Whilst the US SHRP programme has had a major influence throughout the world, other influences such as the desire to harmonize standards within the European Union have re-awakened the interest in asphalt mix design. This has also been a period of the implementation of a range of new bitumen and asphalt test procedures particularly in the USA as a result of SHRP. Most of the validation of this new equipment and design methods has been undertaken on dense graded asphalt or the US Superpave™ mixes.

In Australia, a new asphalt mix design method (APRG 1997a) has been introduced along with a new range of test equipment (Wonson and Bethune 2000). The validation has been undertaken on dense graded asphalt and many of the proposed specification limits based on the performance of DG asphalt. This investigation of SMA has provided additional information that can be used in the on-going development of these test methods. An example was the fundamental research undertaken into the use of an applied vacuum to provide sample confinement in the dynamic creep test. This work needed to incorporate DG asphalt as well as SMA as there was no body of research findings to make comparisons between the performances of the two material types.

Within the design process there are two distinct phases. In the asphalt mix design process, the proportions of the various components of the mix to achieve the structural properties are determined. The mix design process must ensure that a range of serviceability requirements are satisfied such as rut resistance, durability and surface texture properties. The structural design of the pavement uses the stiffness and fatigue properties of the asphalt layers to ensure that the imposed loads can be resisted for the design life. The structural and serviceability requirements often produce conflicting requirements to the mix design process consists of compromises to optimize these requirements.

This thesis has concentrated on the SMA Mix design process and then assessed the implications of the chosen components on the varying conditions of a Tropical Climate in North Queensland. Stone Mastic Asphalt can be considered as a stone skeleton made up of the coarse aggregate which is held together by the mastic made up of binder, fine aggregate, filler and fibres. The mix design can be considered as a two stage process – one for the coarse aggregate skeleton and the other for the mastic.

12.2 Response to Aims of the Research

12.2.1 Developing a Design Method

Developing a design method to ensure that the important features of the coarse aggregate stone skeleton are attained is important. The reported rut resistance of SMA is generally attributed to the stone skeleton however the current design methods do not contain methods to assess whether this stone skeleton is achieved. With reference to the "Wheel Tracking Test Data", the four SMA10 mixes investigated exhibited the same deformation resistance irrespective of the binder type and filler type. Because all the SMA10 mixes contained the same coarse aggregate, it was shown that all the SMA's coarse aggregate skeleton provides the rut resistance of SMA. All combinations of binder and filler types produced a mortar that provided sufficient lateral restraint to the stone skeleton. The SMA14 mix, containing SBS/PMB and fly ash filler would be expected to produce a mix with a low stiffness. It has a similar rut rate to the SMA10 mixes which further confirms that SMA's stone skeleton is the important parameter for deformation resistance.

The Vacuum Confined Dynamic Creep testing undertaken by Stephenson (2002) in conjunction with DMR (QLD) showed that the deformation resistance is dependent on the stone skeleton being established and the mix remaining "undilated" i.e. ensuring coarse stone on stone contact is not interrupted by over filling the voids with mastic. Small increases in the quantity of fine aggregate above the dilation point significantly lowered the long term deformation resistance. This highlighted the importance of determining the actual dilation point and ensuring that the appropriate specification limits are achieved.

Whilst most authorities use a demarcation between coarse and fine aggregate of either the 2.36mm or 4.75mm sieve, the choice appears somewhat arbitrary and based on the raw material classifications. The demarcation point is the same for all mix types and maximum sizes. Consideration of particle packing theory shows that the demarcation between coarse and fine aggregate is a function of the grading and the maximum size of the coarse aggregate. For different size mixes, the demarcation points occur on different size sieves. For SMA, the design criterion is to ensure that the voids in the coarse aggregate are not overfilled by the mastic. The effects on deformation resistance as demonstrated by the Vacuum Confined Dynamic Creep Test showed that this is the paramount consideration. Particle packing theory shows that using the 4.75mm sieve as the maximum size for the mortar components, as used by many researchers (NCAT 1998c, ARRB 1998) is inappropriate. The sieve size should be varied depending on the grading and maximum size of the coarse aggregate. It is suggested that the cut-off between coarse and fine aggregate should be 2.36mm for the SMA14 and 1.18mm for the SMA10. The 1.18mm sieve was also identified in the predictive relationships for the SMA10.

This research demonstrated that unless a single sized coarse aggregate is used, it is not possible to design an undilated mix using the Dilation Point Method (NCAT 1998c, ARRB 1998) in its current form. A 10mm SMA designed by the dilation point test method will have a grading significantly different to a mix designed to DMR (QLD) MRS11.33.

The results from the resilient modulus testing support the Dilation Point Method. For a single sized coarse aggregate, it is a simple means of verifying the dilation point. For mixes with a more complex coarse stone structure such as the DMR (Qld) MRS11.33 SMA10, the resilient modulus testing was able to identify upper grading limits to ensure an undilated “double stone skeleton” where a single dilation point could not be identified from the change in “Gyropac” height. The concept of an undilated “double stone skeleton” accounts for the good rut resistance that has been reported for the Queensland SMA in service (Hogan et al 1999). Such a comment could not be made based solely on the “Gyropac” height verses the fines content due to the complex interactions between the two stone skeletons and the mastic.

12.2.1.1 Extended Method of Design for the SMA Stone Skeleton

Further development work should be incorporated into the specifications with regards to the Determination of Dilation Point AAPA (2002a). It has been proposed in past years that the process of using the DPM to ascertain the maximum particle size of the fine aggregate be adopted. Using the dilation point which is developed by plotting the resilient modulus should now be employed. The grading curve limits can be determined and altered with accuracy compared to the various parameters in source material and production limits.

By incorporating these additional steps into the extended DPM of SMA mix design, methods are provided for selecting the maximum size of the fine aggregate in the mortar and determining the upper grading limits of the SMA. By using these steps, the extended DPM can also be applied to SMA with a “double stone skeleton” such as used in the DMR (Qld) MRS11.33 specification. The design of SMA mixes using the extended DPM provides a rational means of selecting the combined aggregate grading to ensure that the important stone on stone contact is maintained. The extended DPM also gives a simple means of assessing the complex interactions within the combined aggregate grading which are ignored when using “recipe” methods.

12.2.2 Impacts of the Filler and Binder

12.2.2.1 Elastic Properties

An analysis of the investigation was undertaken into the effects of changes in the SMA10 grading by considering samples manufactured to the gradings of APRG (1997a), AAPA (2000a) and DMR (Qld) MRS11.33. It was found that temperature has the most significant effect on the asphalt stiffness however, for constant bitumen content, the increase in stiffness was proportional to a reduction in voids content. The type of aggregate grading chosen does not greatly affect the stiffness rather, the influence of the aggregate grading on resilient modulus, at constant bitumen content, is in its affect on the voids content of the compacted mix. It was shown that the resilient modulus is inversely proportional to the voids content. Optimisation of the bitumen content for each mix type will eliminate this effect.

At typical Queensland in-service pavement temperatures (>20°C), all SMA10 types with Class 320 binder produced similar resilient modulus values. Even in the hotter temperatures of North Queensland where pavement can reach 50°C, the dependency of aggregates on stiffness remain similar. It is the mastic content and hence bitumen content required to fill the voids in the aggregate skeleton that causes the temperature dependency.

The type of filler and binder used in the SMA effect the elastic properties of the mix. The choice of filler makes an impact on the stiffness with the lime filler having the greatest stiffening effect. The choice of binder has the greatest impact on the stiffness. As expected the incorporation of the PMB/SBS significantly reduced the stiffness. In applications where the load spreading ability of SMA needs to be considered, the effects on stiffness of the mix of the chosen filler and binder should also be considered. Testing of a range of laboratory and plant produced SMA10 highlighted the effect of filler type, incorporation of fibres and type of binder on the mix stiffness. The use of simple nomographs such as included in AUSTROADS (1992) are not appropriate for estimating SMA stiffness as the effects of these significant mix components are not included. The effects of temperature on mix stiffness can be readily determined using equipment such as the MATTA.

12.2.2.2 Fatigue Properties

The testing programme has shown the fatigue life benefits of using SMA in place of DG asphalt however it must be appreciated there will be a corresponding reduction in the stiffness of the mix. The fatigue life benefits of using the SBS-PMB were clearly demonstrated however this comes with the sacrifice of significantly reduced stiffness. The reduction in stiffness when using the SMA10 and/or SBS-PMB will need to be considered as part of any pavement design process. Given the strong relationship between stiffness and fatigue life, the change in stiffness makes significant differences in fatigue life. The testing programme has shown that the type of filler and binder used in the SMA significantly affects the elastic properties and hence the fatigue life of the mix.

For the mixes containing Class 320 bitumen, the higher fatigue life measured for the SMA mixes confirms the claim that the higher bitumen content of SMA10 compared to DG14 results in greater fatigue life (AAPA 2000a). The change in grading to allow the higher bitumen content resulted in a 10 fold increase in fatigue life. The choice of binder has dramatic implications on the fatigue life. For the PMB used and discussed in this research, it resulted in a 100 fold increase in fatigue life of the SMA. Of the SMA10 mixes containing the same binder, the fatigue life is a function of the filler type. Lime filler produced the shortest fatigue life, followed by ultra fine dust and fly ash.

12.2.2.3 Rut Resistance

The analysis on the various SMA mixes investigated exhibited the same deformation resistance when reviewing data from the measurements using the wheel tracking test. There was no influence of the binder type and filler type on the rut resistance of the SMA10 mixes tested. All the SMA10 mixes contained the same coarse aggregate and it was shown that it is the SMA’s coarse aggregate skeleton that provides the rut resistance of SMA, if produced to design mix tolerances.

12.2.2.4 Implications for the Choice of Mastic Materials

Typically, for dense graded asphalt good rut resistant mixes would have high stiffness values. As explained in Chapter 10, fatigue life is inversely proportional to mix stiffness, therefore the stiff; rut resistance mix would result in lower fatigue life. By using the SMA concept, rut resistant mixes can be produced with high fatigue life especially by incorporating the PMB/SBS. The testing programme has also quantified the effects on stiffness and fatigue life of commonly used fillers. This has implications for generic type specifications where a variety of fillers are permitted in nominally the "same" mix because the differing properties of the mix and the implications on pavement performance are not normally considered as part of the pavement design process.

From the wheel tracking test, it was found that all combinations of binder and filler types produced a mortar that provided sufficient lateral restraint to the stone skeleton. The selection of binder and filler needs to be made on the basis of the other required mix properties. Binders and filler may be selected to increase stiffness or fatigue life without compromising the rut resistance of SMA. This has proved similar with DMR (QLD Peninsula) testing over the last 6 years, where lime is the predominate filler and PMB for the binder.

12.3 Further Research and Recommendations

12.3.1 Skid Resistance

If public perception and general news is worthy of concern, the latest is all about safety and further work could be placed on Skid Resistance and how the problems can be addressed in the design and construction process. This could have a very meaningful outcome from the point of view of the supplier, road authority and hence the public. It would address a topic currently blown out of all proportions and seemingly poorly understood by many, including people within the industry.

The skid resistance parameters are specified using a range of different criteria – speed environment, rainfall events, and designed crossfall of the roadway, actual tyre and vehicle performance. The issue with the publicised crash data is that these items need to be reviewed in detail before commenting on the skid resistance. Was it raining greater than normal? Was the vehicle speeding? Is the roadway to a designed crossfall? Was the vehicle roadworthy? The issue becomes a media display and the road is termed slippery before any real analysis.

In North Queensland the initial failure mechanism has been listed as coming from early life skid resistance. This may in fact have come from flushing through which is an entirely different problem, but one that obviously causes the initial failure, even though indirectly.

12.3.2 Continued Monitoring

The trial sections identified previously in the paper comprised significant changes in the graduation and the type of binder used in the SMA.

The performance of this SMA trial will need to be evaluated over time, and will go past the timing for this paper. A continued monitoring approach should be employed, and the testing proposed is identified.

The following testing is to be evaluated over time:

- Texture;
- Skid resistance;
- Permeability;
- Stripping potential by extraction of cores;

- Deflection testing (if early cracking is observed).

The analysis charts and graphs from Chapter 7 are not normally produced per project, and certainly not per quarter or year against all projects. This would not be difficult to administer, and would give the specifiers a guide as to where minor changes are necessary to enhance the design of SMA. The element of production and field testing provides most of the data necessary, although it is recommended that it be better identified from a location perspective. Definitely the current projects should continue to form a data-base for future analysis.

12.3.3 Texture

The drawback with using negatively or porous textured materials is that the high proportion of coarse aggregate requires a modified binder to ensure the materials integrity. This may be achieved by the used of an additive such as cellulose fibers to create a thicker binder film on the aggregate, modification of the binder using a polymer. This is a fundamental change in the type of surfacing material i.e. from positive texture to smoother negatively textured. The addition of fibers or polymer modification of the bitumen produces either a thicker binder film on the aggregate or stronger bond between binder and aggregate. Either way this will affect the rate at which trafficking can remove the binder and expose the aggregate. This seems to be a continued point of discussion.

12.3.4 Underlying Layers

The SMA layers throughout the State are generally used as the final wearing course. Preceding this layer is a number of prior layers that are of a bituminastic, cementicious or aggregate nature. One of the latest specifications in pavements is the using of foamed bitumen as a stabilising agent for gravel layers. In North Queensland where the water table sits at around 1 m above sea level, and the pavements are generally at 2 m, there is a metre of air in between. The movement of water within the ground we believe is affecting the performance of the pavement designs. The layers are also under traffic and sealed with a C170 primer seal at an early stage which traps water within the gravel.

The issue which would create a further research study is on the basis of a vapourisation affect of the water table to drag the bitumen from the pavement to the surface. This then affects the seal layers and eventually the SMA layer on top. The ‘blotchy and almost volcanic’ nature of the excess bitumen on the surface of a number of sections in North Queensland may actually be caused from such an action.

12.3.5 North Queensland Approach

The North Queensland approach to local issues has been towards “overdesign”. The original SMA mixes were designed with good stone on stone contact, but the aggregate size changed due to availability. The reduction to a 10mm saw flushed areas and early failure. It is known that flushed areas tend to promote rutting as the next failure mechanism, but this is not always the case. This concept was taken from the knowledge in Dense Graded Asphalt.

With SMA, the flushing has not lead to rutting, which has assisted in proving the stone to stone contact of the design is what makes SMA a rut resitanct asphalt. In the process a full 100% hydrated lime was introduced as a reaction to the possible rutting. A full polymer modified binder was also directed as mandatory for stiffness reasons.

Over time, we have seen areas where the traffic has worn away the binder on the surface, and generally thay have remained rut resistant. The “belts and braces” approach has come with economic implications, and certainly caused the failure mechanisms to have many variables. The harder binder has assisted with internal raveling, but the existing SMA mix design is still very applicable.

12.4 Conclusions

Promoted as having high rut resistance, SMA is typically used in heavy and very heavy traffic conditions. In Queensland, where thin surfacing layers (<40mm for SMA10) are used, often over old and weak pavements with high deflection, fatigue failure of SMA rather than rutting may be the limiting design factor. This research has provided a rational basis for comparing the fatigue life benefits of using SMA in place of the traditional dense graded asphalt. By combining the rut resistant stone skeleton with fatigue performance of the SBS/OMB, SMA is able to satisfy two conflicting requirements.

This research has demonstrated that it is the coarse stone skeleton of SMA that provides the rut resistant properties and endorses the notion of using an extended Dilation Point Method to ensure that the stones skeleton remains “undilated”. By considering the effects of different binders and fillers used in the mastic of SMA, comprehensive material performance data has been proposed for SMA mixes. It has been utilised to both explain and predict the future performance of the road pavements under a range of operating conditions. This thesis presents stiffness, fatigue and deformation performance data which can be used as inputs for the mechanistic approach to the design of pavements incorporating SMA and PMBs.

This research has identified the various considerations in Production and Laying of SMA in a Tropical Climate, and the necessity for complete quality control. SMA can remain the preferred asphalt of choice in a climate such as North Queensland, but under strict guidance control.

This research has reinforced the major works undertaken by DMR (QLD Peninsula) in the standards for SMA, and the localised and unique modifications of the specifications. Adherence by all concerned will ensure that SMA provides the properties necessary for a long life pavement.

Whilst conventional DG asphalt will remain adequate for many asphalt paving situations, SMA fulfils a niche role for surfacing in the heavy duty, high traffic volume pavement situations where DG asphalt does not perform adequately. Whilst SMA is a premium asphalt product, it is not a panacea for all pavement construction and maintenance situations. The level of research into SMA technology has been great and needs to continue on the basis of developing new longer life pavements with a minimum cost and greater economic benefits to the consumer, and reduced production issues and a minimum of cost.

The continued testing of SMA is proving to be more about free binder and the combination of fines to binder, but the analysis over time is vital to performance. Stone Mastic Asphalt is just one of the many surfacing types in the arsenal of pavement designers. Its use and specification requires exercising sound engineering judgment. This research has provided a rational basis for such judgment, and this knowledge needs to extend throughout the network to all the specifiers of asphalt on our roadways.

Appendix A – Copy of Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project

PROJECT SPECIFICATION

Issue A, 27th March 2006

STUDENT: Glen Allen

TOPIC: Problems of Stone Mastic Asphalt Use in North Queensland.

SUPERVISOR: Ron Ayers – University of Southern Queensland

David Hamilton – Queensland Department of Main Roads

PROJECT AIM: This project seeks to produce best practice guidelines for the design and use of stone mastic asphalt in the north Queensland area.

SPONSORSHIP: Queensland Department of Main Roads

PROGRAM:

1. Review existing literature from Australia and overseas regarding the design and use of asphalt surfacings in tropical areas, and the problems encountered with such surfacings.
2. Assess the extent of use of stone mastic asphalt in north Queensland, and the problems currently encountered with the material.
3. Compile and analyse data on trials with stone mastic asphalt in north Queensland (including associated data such as traffic volumes and composition, and rainfall).
4. In conjunction with Main Roads and/or local authorities, select various trial sections for testing or re-testing to determine current data and problems. Such testing may include skid resistance, surface texture, voids and flushing.
5. Analyse data and investigate correlations between parameters such as asphalt properties, design criteria and failure mechanisms.
6. Establish recommendations for best practice design and use of stone mastic asphalt in north Queensland.
7. Report findings through oral presentation at the project conference and in the required written format.

As time permits:

8. Perform a risk assessment to consider the risks involved in design, construction and maintenance issues associated with the use of stone mastic asphalt.

AGREED:

________ (Student) ________, R.A. Ayers (Supervisors)

4 / 4 / 06   4 / 4 / 06   10 / 4 / 06

28 March 2006

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Appendix B – Media Releases

Cheap asphalt on state death roads

THE Queensland Government has been accused of hiding a secret list of nearly 80 roads, including the Captain Cook and Bruce highways, surfaced with the controversial stone mastic asphalt.

State Opposition Leader Lawrence Springborg said his office had uncovered a matrix of the SMA-covered roads throughout Queensland which the Government had not admitted to, including four which were under investigation.

"What we have in there is a whole range of particular problem sites which have not been exposed to date," Mr Springborg said in Brisbane yesterday.

The SMA surface was used as a cheaper and longer wearing alternative to the traditional asphalt surface, but was later found to become slippery and treacherous in the wet.

The Queensland Government has admitted it made a mistake using the asphalt and has begun plans to resurface a section of the Bruce Highway on the Sunshine Coast, linked to four deaths since December last year.

Mr Springborg said the "secret" list had identified 78 problem sites throughout Queensland, including Gympie, Bundaberg, Toowoomba, Cairns and Townsville. It included sections of the Warego, Captain Cook and Bruce highways.

Mr Springborg said the Government knew of problems with the surfacing as far back as 2002 and they were "far more significant" than what it had confessed to. He said Transport Minister Paul Lucas had to "fess up" to a number of SMA covered roads throughout Queensland. "The Labor Government in Queensland is actually killing Queenslanders, not only in our hospitals, but also on our roads," Mr Springborg said.

The claim comes as an independent expert hired to review Queensland's use of the SMA surface was fired just hours after he was hired.

Paul Hillier, from the Sydney office of UK-based Transport Research Laboratories, was stood down on Thursday night after it was revealed he had lectured road authorities on how they could defend themselves against crash victim lawsuits that blamed the road surface.

— AAP, Brisbane

Alerts on asphalt date back 3 years

By DARRELL GILES
political editor

THE State Government knew there were problems with stone mastic asphalt on Queensland highways three years ago, according to an internal document.

A Main Roads investigation into sections of the Bruce Highway revealed significant damage to the surface and recommended immediate remedial work.

The May 2002 report contradicts claims by Transport and Main Roads Minister Paul Lucas this week that the government only learnt of problems with this asphalt in December last year and April this year.

On Thursday, the department continued to deny it had any prior knowledge of a technical fault on a stretch of the highway south of Gympie, where four people have died this year.

The controversial asphalt has been banned in parts of Europe. The Brisbane City Council stopped using it four years ago.

Mr Lucas admitted the government had made a mistake using the asphalt to surface a 1.4km stretch of the highway at Federal, south of Gympie, and will now replace it.

The asphalt has also been used to surface another 1280km of roads across the state and a clearly concerned Mr Lucas said these would be closely examined.

PROBLEMS: A section of road sealed with the asphalt

Mr Lucas, who complained that his department had withheld two emails on the problems in the past six months, was forced on Friday to sack the independent expert he had just appointed to review of the use of asphalt.

The May 2002 report, "Investigation of the observed distress on the Stone Mastic Asphalt surfacing – Bruce Highway North Coast Hinterland District", is likely to further embarrass him.

Opposition Leader Lawrence Springborg produced the document yesterday and accused the government of a major cover-up.

"They have known about the problem for over three years and have done nothing," he said. "In that time, people have been killed and seriously injured."

Experts from the Pavement Rehabilitation Section and the Pavements, Materials and Geotechnical Division found significant problems on the Bruce Highway north of Gympie.

They inspected 13km north of Gympie, 5.5km near the Sunshine Motorway interchange and a section of the Eumundi-Noosa road.

Defects in the asphalt included cracking, depression, flushing of surface, wet areas and materials coming through the surface. Some damage was described as significant.

They suggested various options, including repair of the roads with a different asphalt or complete replacement.

Some options were ruled out because they were too costly.

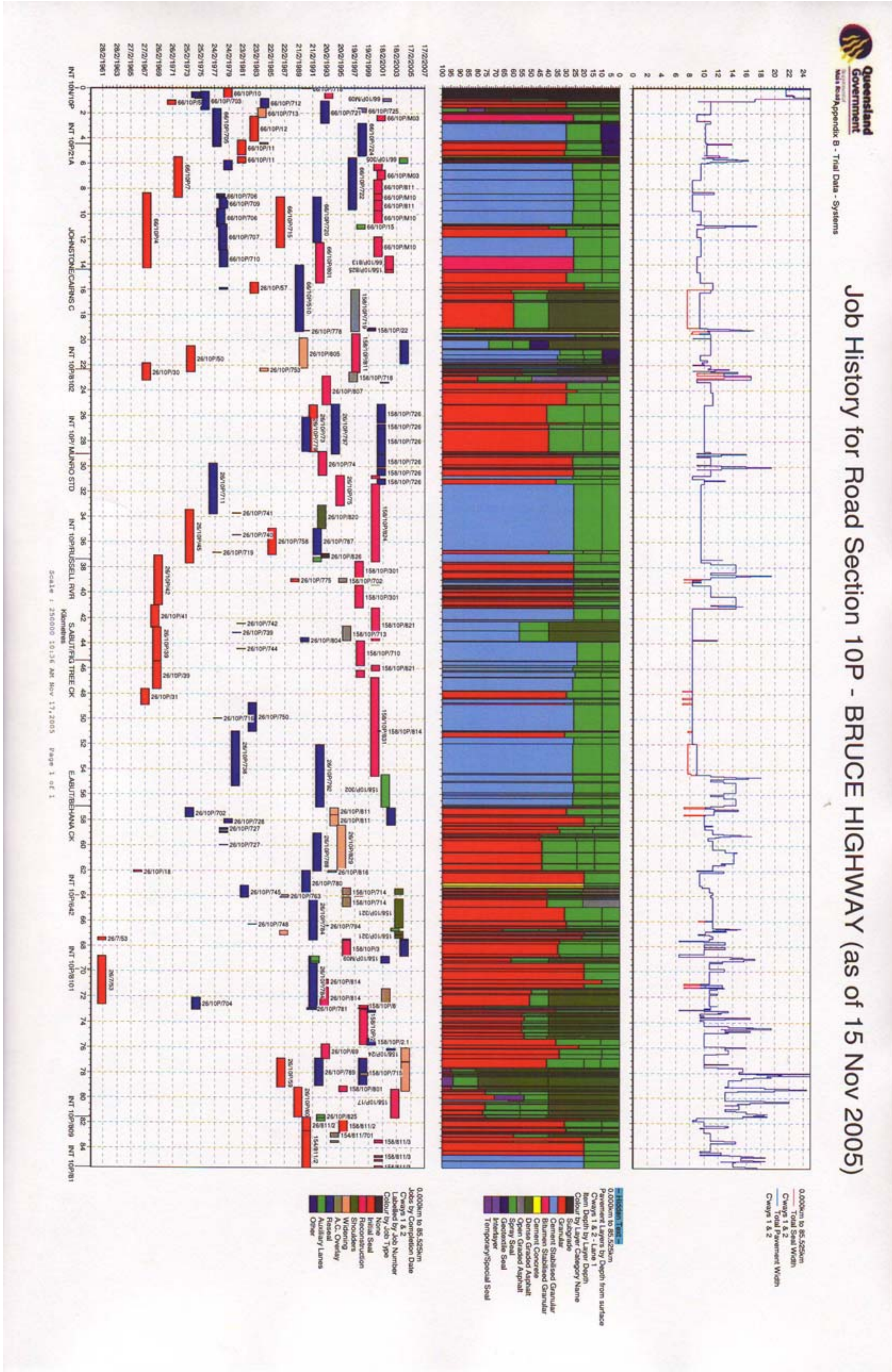
Mr Lucas said Main Roads engineers had told him yesterday that the problems on the highway north of Gympie were "issues of durability, not safety".

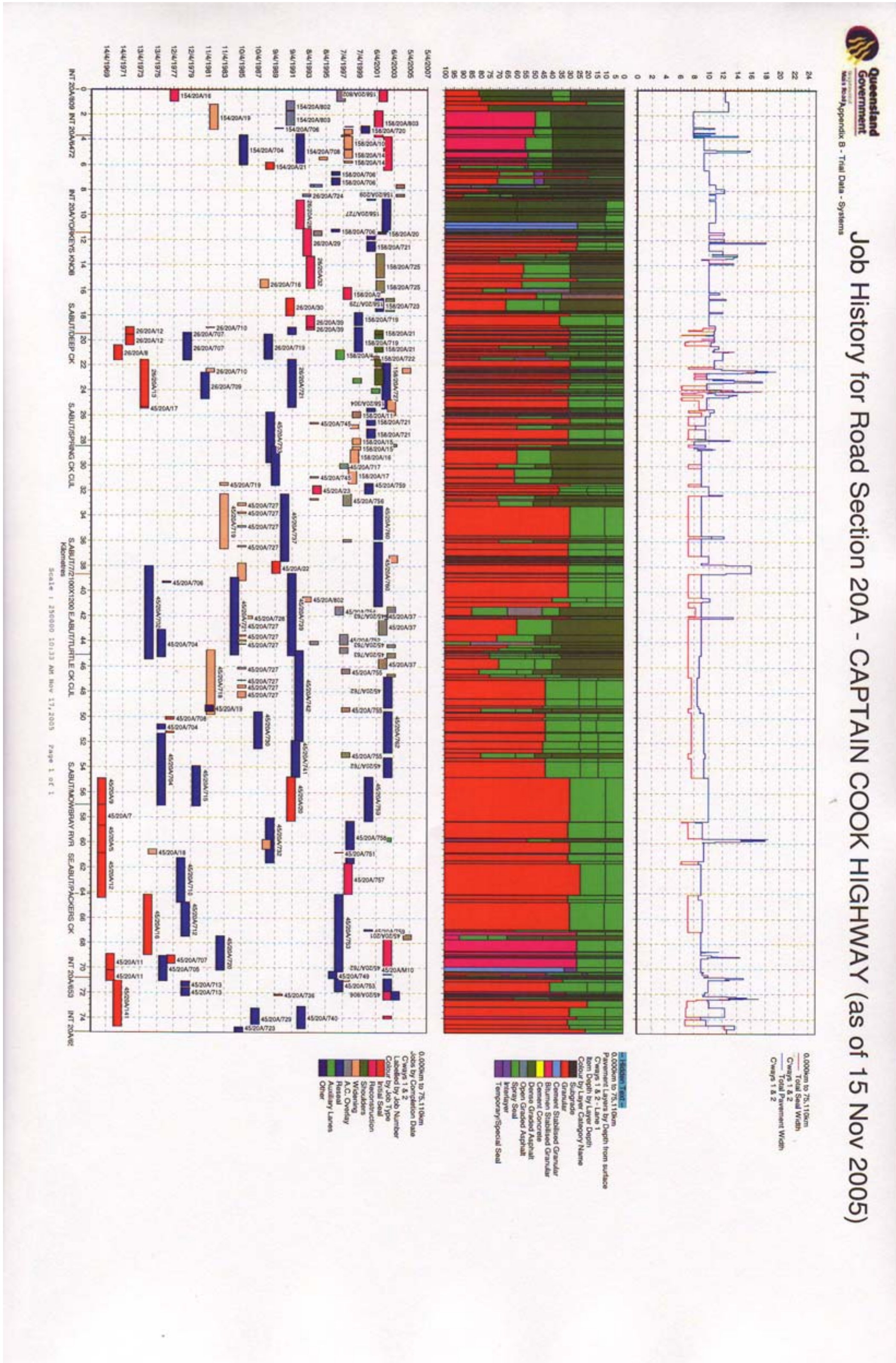
He said they could not be compared to the stretch near Federal.

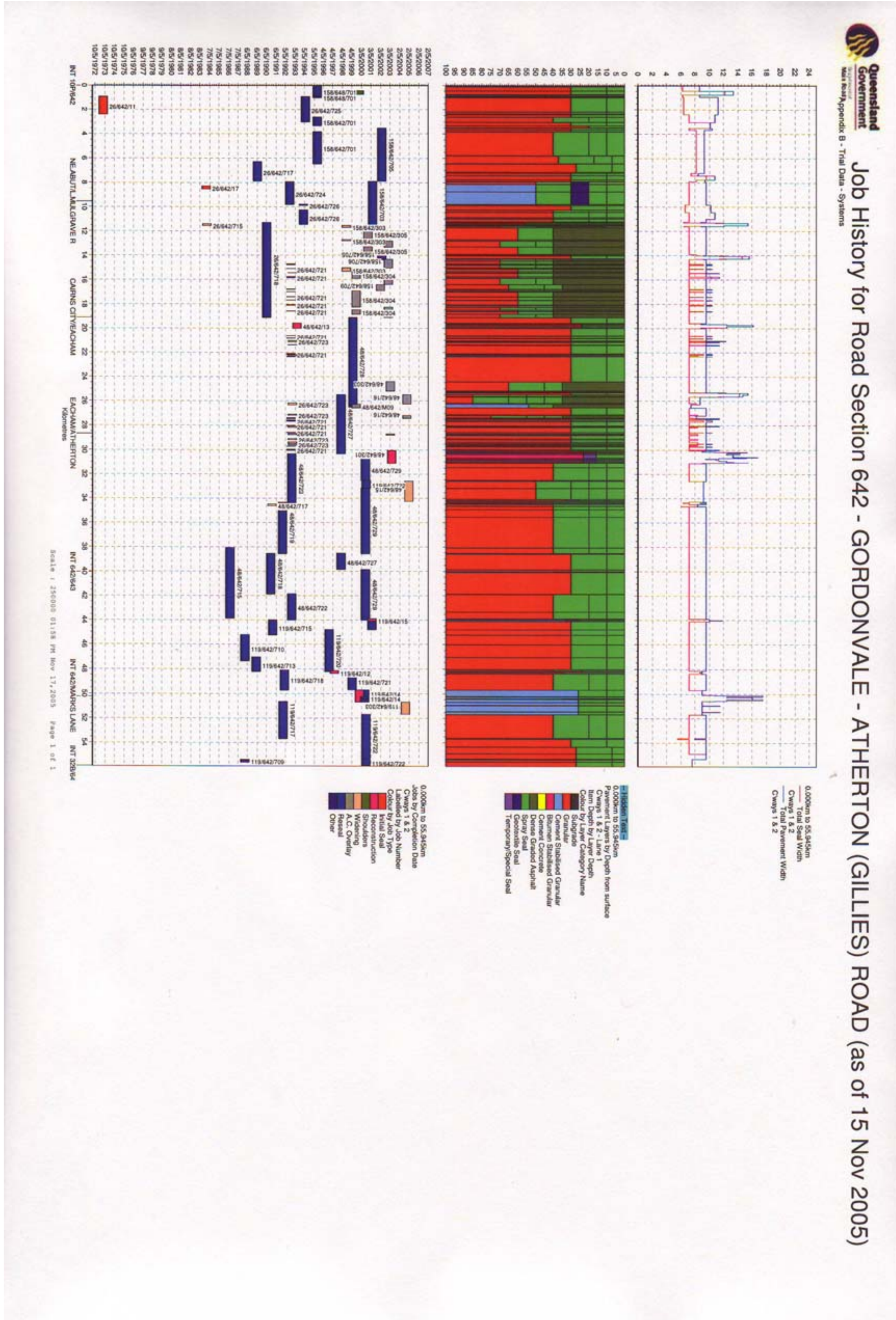
Mr Lucas said an independent reviewer would look at all aspects of the asphalt since it was first laid in Queensland in 1996 and would re-examine the 2002 report.

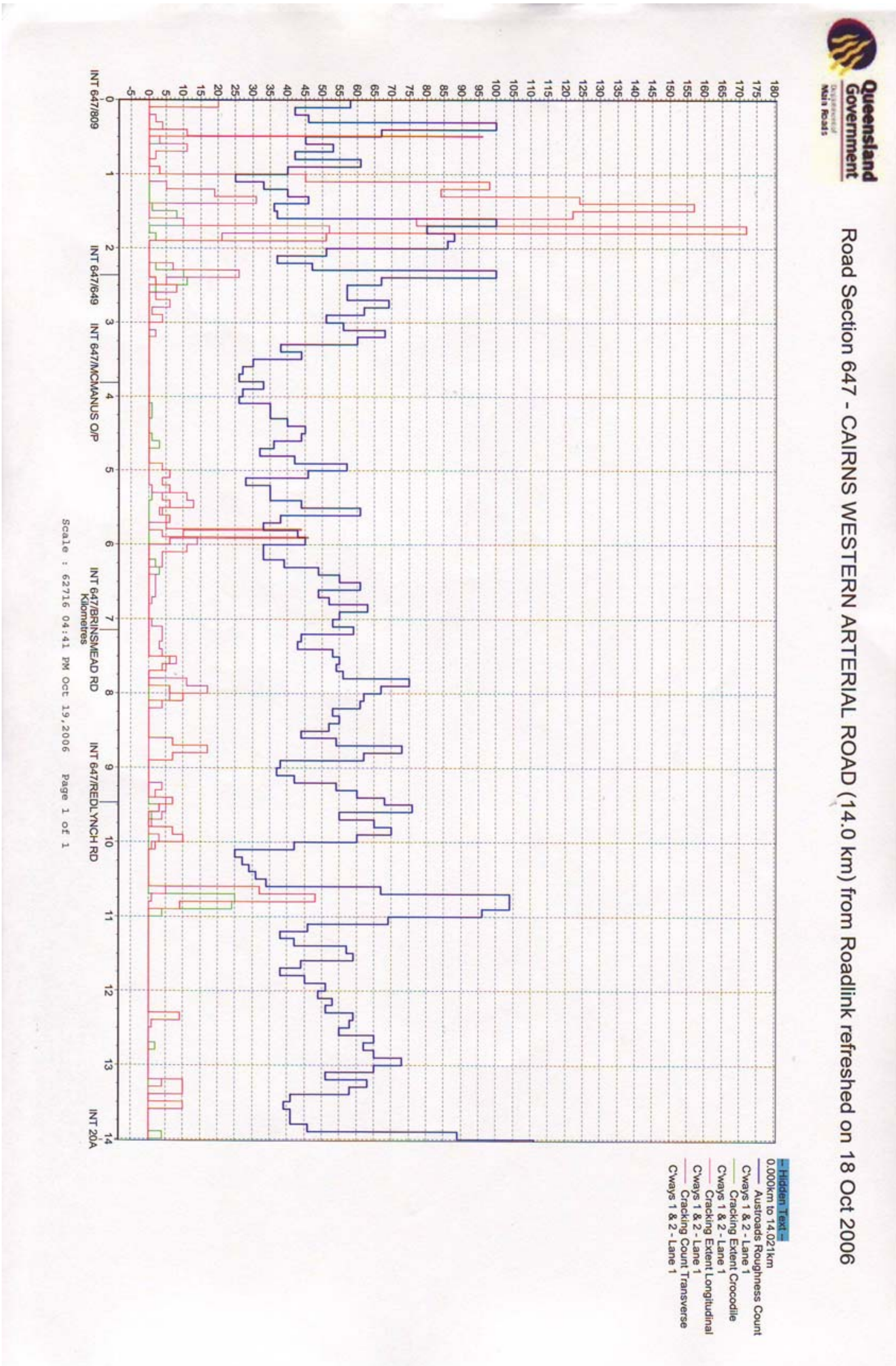
Source: Sunday Mail 26/6/2005

Appendix C – Trial Data – Systems


Appendix B - Trial Data - Systems
Job History for Road Section 10P - BRUCE HIGHWAY (as of 15 Nov 2005)


Job History for Road Section 20A - CAPTAIN COOK HIGHWAY (as of 15 Nov 2005)
Queensland Government
Appendix B - Trial Data - Systems


Queensland Government
Appendix B - Trial Data - Systems
Job History for Road Section 642 - GORDONVALE - ATHERTON (GILLIES) ROAD (as of 15 Nov 2005)


Queensland Government
Main Roads
Road Section 647 - CAIRNS WESTERN ARTERIAL ROAD (14.0 km) from Roadlink refreshed on 18 Oct 2006
- Hidden Text -
0.000km to 14.021km
Austroads Roughness Count
C'ways 1 & 2 - Lane 1
Cracking Extent Crocodile
C'ways 1 & 2 - Lane 1
Cracking Extent Longitudinal
C'ways 1 & 2 - Lane 1
Cracking Count Transverse
C'ways 1 & 2 - Lane 1
INT 647/809
INT 647/649
INT 647/MCMANUS O/P
INT 647/BRINSMEAD RD
INT 647/REDLYNCH RD
INT 20A
Kilometres
Scale : 62716 04:41 PM Oct 19,2006 Page 1 of 1

As discussed I have tabled the PAFV results for the asphalt mixes used in the District

QUARRY SOURCE	MIX DESIGN TYPE	DATE	PAFV	PAFV SPECIFICATION (MIN)
PNQ EDMONTON	SM12	2001	54	45 current- (Proposed > 50)
PNQ EDMONTON	SM14	2001	54	45
PNQ EDMONTON	SM10	2001	54	45
PNQ EDMONTON	DG14	1998	47	45
PNQ EDMONTON	OG14	1995	48	45
BORAL TICHUM CREEK	SM12	2002	49	45 current - (Proposed> 50)
BORAL TICHUM CREEK	SM14	2002	49	45
BORAL REDLYNCH	DG7	2003	53	45
BORAL REDLYNCH	DG10	1999	49	45
BORAL REDLYNCH	SM10	1998	48	45

Appendix D – Trial Data – Actual Sites

Site_ID	Rank by change in all	Rank by change in wet	District	Carriageway Code	District Name	Road Section ID	Road Section Name	SMA Install Date	Tdist Start	Tdist End	Length	Pre SMA Crashes/Yr	Post SMA Crashes/Yr	% Change Crashes/Yr
551	408	309	11	1	Peninsula	627	Innisfail - Jap	08-Aug-03	0.00	0.07	0.07	0.6	0.0	-100%
342	426	506	11	1	Peninsula	642	Gordonvale -	12-Jun-01	11.73	12.72	0.99	1.2	0.5	-58%
114	113	72	11	1	Peninsula	642	Gordonvale -	24-Feb-99	12.72	12.84	0.12	0.0	0.4	Large
545	133	52	11	1	Peninsula	642	Gordonvale -	07-Aug-03	12.84	13.29	0.45	0.2	0.6	176%
343	101	324	11	1	Peninsula	642	Gordonvale -	12-Jun-01	13.29	13.93	0.64	0.2	0.8	278%
546	106	101	11	1	Peninsula	642	Gordonvale -	07-Aug-03	14.34	15.03	0.69	0.4	1.1	176%
115	70	110	11	1	Peninsula	642	Gordonvale -	24-Feb-99	15.03	15.33	0.30	0.2	1.2	500%
169	421	400	11	1	Peninsula	642	Gordonvale -	17-Mar-00	15.33	15.77	0.44	0.8	0.0	-100%
547	254	262	11	1	Peninsula	642	Gordonvale -	07-Aug-03	15.77	15.85	0.08	0.0	0.0	0%
170	310	282	11	1	Peninsula	642	Gordonvale -	17-Mar-00	15.85	15.89	0.04	0.2	0.0	-100%
548	134	370	11	1	Peninsula	642	Gordonvale -	07-Aug-03	15.89	16.41	0.52	0.2	0.6	176%
481	248	256	11	1	Peninsula	642	Gordonvale -	16-Oct-02	16.41	16.93	0.52	0.0	0.0	0%
171	363	360	11	1	Peninsula	642	Gordonvale -	17-Mar-00	16.93	18.26	1.33	1.4	1.4	0%
549	84	161	11	1	Peninsula	642	Gordonvale -	07-Aug-03	18.26	18.46	0.20	0.0	0.6	Large
172	327	422	11	1	Peninsula	642	Gordonvale -	17-Mar-00	18.46	18.82	0.36	0.4	0.2	-50%
550	255	263	11	1	Peninsula	642	Gordonvale -	07-Aug-03	18.82	19.11	0.29	0.0	0.0	0%
565	358	390	11	1	Peninsula	642	Gordonvale -	30-Oct-03	24.39	25.16	0.77	0.4	0.0	-100%
173	347	294	11	1	Peninsula	642	Gordonvale -	17-Mar-00	26.28	26.53	0.25	0.4	0.0	-100%
566	359	444	11	1	Peninsula	642	Gordonvale -	30-Oct-03	28.73	28.83	0.10	0.4	0.0	-100%
264	270	271	11	1	Peninsula	642	Gordonvale -	23-Dec-00	39.97	40.06	0.09	0.2	0.2	13%
227	216	224	11	1	Peninsula	642	Gordonvale -	25-Jul-00	50.22	50.27	0.05	0.0	0.0	0%
228	217	225	11	1	Peninsula	642	Gordonvale -	25-Jul-00	50.41	50.46	0.05	0.0	0.0	0%
263	222	230	11	1	Peninsula	643	Malanda - La	23-Dec-00	12.05	12.09	0.04	0.0	0.0	0%
512	39	150	11	2	Peninsula	647	Cairns Weste	16-Apr-03	0.00	0.07	0.07	1.0	3.3	230%
513	336	291	11	3	Peninsula	647	Cairns Weste	16-Apr-03	0.00	0.03	0.03	0.6	0.5	-21%
151	20	68	11	3	Peninsula	647	Cairns Weste	10-Sep-99	5.91	7.42	1.51	0.6	3.6	500%
150	57	100	11	2	Peninsula	647	Cairns Weste	10-Sep-99	5.91	7.42	1.51	0.6	2.2	266%
152	5	27	11	1	Peninsula	647	Cairns Weste	10-Sep-99	7.42	9.78	2.36	0.0	5.2	Large
338	298	351	11	1	Peninsula	647	Cairns Weste	24-May-01	9.78	10.23	0.45	1.2	1.5	24%
385	138	371	11	1	Peninsula	647	Cairns Weste	19-Dec-01	10.23	10.67	0.44	0.4	0.9	118%
482	89	136	11	1	Peninsula	647	Cairns Weste	31-Oct-02	11.50	11.62	0.12	1.4	2.7	94%
537	262	268	11	1	Peninsula	647	Cairns Weste	08-Jul-03	11.62	12.43	0.81	0.4	0.5	32%
538	77	131	11	1	Peninsula	647	Cairns Weste	08-Jul-03	13.59	14.02	0.43	1.2	2.6	120%
372	493	333	11	2	Peninsula	649	Anderson Str	05-Dec-01	0.30	1.69	1.39	4.4	3.4	-22%
373	504	524	11	3	Peninsula	649	Anderson Str	05-Dec-01	0.30	1.69	1.39	5.2	4.0	-23%
163	212	220	11	3	Peninsula	649	Anderson Str	11-Dec-99	1.69	1.77	0.08	0.0	0.0	0%
164	213	221	11	2	Peninsula	649	Anderson Str	11-Dec-99	1.69	1.73	0.04	0.0	0.0	0%
376	139	184	11	1	Peninsula	649	Anderson Str	05-Dec-01	2.19	2.37	0.18	0.4	0.9	115%
523	7	38	11	1	Peninsula	649	Anderson Str	25-May-03	2.37	2.51	0.14	2.2	7.4	238%
558	189	374	11	1	Peninsula	653	Mossman - N	26-Aug-03	2.46	3.56	1.10	0.4	0.6	42%
140	206	214	11	1	Peninsula	653	Mossman - N	04-Jun-99	4.25	4.36	0.11	0.0	0.0	0%
141	207	215	11	1	Peninsula	653	Mossman - N	04-Jun-99	4.55	4.65	0.10	0.0	0.0	0%
460	391	486	11	1	Peninsula	653	Mossman - N	04-Jul-02	7.85	9.06	1.21	0.8	0.3	-57%
552	256	264	11	1	Peninsula	662	Mareeba Con	08-Aug-03	0.00	0.03	0.03	0.0	0.0	0%
442	242	250	11	1	Peninsula	664	Mareeba - Di	07-Jun-02	0.00	0.03	0.03	0.0	0.0	0%
553	375	392	11	1	Peninsula	664	Mareeba - Di	08-Aug-03	0.34	1.63	1.29	3.0	3.3	11%
444	457	414	11	2	Peninsula	664	Mareeba - Di	07-Jun-02	1.79	2.72	0.93	5.2	5.0	-3%
445	386	513	11	3	Peninsula	664	Mareeba - Di	07-Jun-02	1.79	2.72	0.93	2.8	3.0	8%
554	356	300	11	2	Peninsula	664	Mareeba - Di	08-Aug-03	2.72	2.96	0.24	0.4	0.0	-100%
555	409	310	11	3	Peninsula	664	Mareeba - Di	08-Aug-03	2.72	2.96	0.24	0.6	0.0	-100%
556	257	265	11	1	Peninsula	664	Mareeba - Di	08-Aug-03	2.96	3.02	0.06	0.0	0.0	0%
446	301	279	11	1	Peninsula	664	Mareeba - Di	07-Jun-02	3.27	3.39	0.13	0.6	0.7	12%
12	115	171	11	1	Peninsula	664	Mareeba - Di	30-Jun-97	3.93	4.59	0.66	0.0	0.4	Large
508	470	449	11	2	Peninsula	809	Mulgrave Rd	19-Mar-03	0.00	0.05	0.05	1.6	0.5	-72%
509	519	451	11	3	Peninsula	809	Mulgrave Rd	19-Mar-03	0.00	0.03	0.03	3.2	0.0	-100%
514	59	153	11	3	Peninsula	809	Mulgrave Rd	16-Apr-03	0.03	0.72	0.69	0.2	1.4	607%
515	159	194	11	2	Peninsula	809	Mulgrave Rd	16-Apr-03	0.05	0.72	0.67	0.8	1.4	77%
527	13	83	11	3	Peninsula	809	Mulgrave Rd	29-Jun-03	0.72	1.36	0.64	2.0	6.3	213%
526	513	463	11	2	Peninsula	809	Mulgrave Rd	29-Jun-03	0.72	1.36	0.64	4.4	2.6	-41%
516	16	354	11	2	Peninsula	809	Mulgrave Rd	16-Apr-03	2.36	2.94	0.58	6.8	11.8	73%
517	482	462	11	3	Peninsula	809	Mulgrave Rd	16-Apr-03	2.68	2.96	0.28	3.0	1.9	-37%
256	221	229	11	2	Peninsula	809	Mulgrave Rd	01-Dec-00	2.94	3.24	0.30	0.0	0.0	0%
257	345	293	11	3	Peninsula	809	Mulgrave Rd	01-Dec-00	2.96	3.22	0.26	0.6	0.4	-26%
518	140	478	11	3	Peninsula	809	Mulgrave Rd	16-Apr-03	3.22	4.62	1.40	5.2	7.1	36%
519	282	518	11	2	Peninsula	809	Mulgrave Rd	16-Apr-03	3.24	4.61	1.37	8.8	10.8	23%
249	219	227	11	1	Peninsula	6504	Port Douglas	20-Oct-00	0.03	0.27	0.24	0.0	0.0	0%
250	325	471	11	1	Peninsula	6504	Port Douglas	20-Oct-00	2.79	2.97	0.18	1.8	2.2	21%
500	353	297	11	1	Peninsula	6504	Port Douglas	30-Jan-03	5.18	5.38	0.20	0.4	0.0	-100%
153	209	217	11	1	Peninsula	8101	Pine Creek -	22-Oct-99	17.07	17.09	0.02	0.0	0.0	0%
165	214	222	11	1	Peninsula	8101	Pine Creek -	15-Jan-00	21.16	21.34	0.18	0.0	0.0	0%
154	210	218	11	1	Peninsula	8101	Pine Creek -	22-Oct-99	22.63	22.65	0.02	0.0	0.0	0%
456	407	308	11	1	Peninsula	8101	Pine Creek -	28-Jun-02	23.86	25.51	1.65	0.6	0.0	-100%
457	305	280	11	1	Peninsula	8101	Pine Creek -	28-Jun-02	25.88	27.13	1.25	0.4	0.3	-14%
557	474	406	11	1	Peninsula	10N	Ingham - Inni	08-Aug-03	146.66	146.90	0.24	1.8	0.6	-69%
426	75	436	11	1	Peninsula	10P	Innisfail - Cai	11-May-02	0.80	1.04	0.24	0.8	2.0	146%
75	466	78	11	1	Peninsula	10P	Innisfail - Cai	28-Oct-98	1.59	1.92	0.33	2.0	1.0	-50%
16	172	15	11	1	Peninsula	10P	Innisfail - Cai	30-Sep-97	16.00	19.02	3.02	1.0	1.6	60%
166	160	343	11	1	Peninsula	10P	Innisfail - Cai	21-Feb-00	19.02	19.27	0.25	0.4	0.8	100%
238	496	467	11	1	Peninsula	10P	Innisfail - Cai	30-Aug-00	42.26	43.83	1.57	2.2	0.6	-71%
525	121	176	11	1	Peninsula	10P	Innisfail - Cai	20-Jun-03	66.54	66.81	0.27	0.4	1.0	157%
392	420	399	11	1	Peninsula	10P	Innisfail - Cai	13-Feb-02	68.79	69.31	0.52	1.0	0.3	-70%
399	484	503	11	1	Peninsula	10P	Innisfail - Cai	27-Feb-02	71.34	72.80	1.46	2.0	0.6	-69%
79	271	272	11	1	Peninsula	10P	Innisfail - Cai	28-Nov-98	72.90	72.94	0.03	0.2	0.2	0%

80	33	44	11	2	Peninsula	10P	Innisfail - Cai	28-Nov-98	73.20	75.32	2.11	0.2	2.2	999%
81	36	69	11	3	Peninsula	10P	Innisfail - Cai	28-Nov-98	73.20	75.11	1.91	0.0	1.8	Large
111	204	212	11	2	Peninsula	10P	Innisfail - Cai	29-Jan-99	78.01	78.04	0.02	0.0	0.0	0%
112	273	273	11	3	Peninsula	10P	Innisfail - Cai	29-Jan-99	78.01	78.07	0.05	0.2	0.2	0%
510	175	353	11	3	Peninsula	10P	Innisfail - Cai	19-Mar-03	79.35	80.49	1.14	3.2	4.5	42%
511	528	534	11	2	Peninsula	10P	Innisfail - Cai	19-Mar-03	79.35	81.60	2.25	10.4	6.4	-39%
271	360	434	11	3	Peninsula	10P	Innisfail - Cai	12-Feb-01	80.49	81.90	1.41	3.2	3.7	16%
272	37	149	11	3	Peninsula	10P	Innisfail - Cai	12-Feb-01	82.65	84.20	1.55	0.2	2.1	948%
273	348	295	11	2	Peninsula	10P	Innisfail - Cai	12-Feb-01	83.40	83.56	0.17	0.4	0.0	-100%
274	97	111	11	3	Peninsula	10P	Innisfail - Cai	12-Feb-01	84.55	85.53	0.98	0.0	0.5	Large
275	349	296	11	2	Peninsula	10P	Innisfail - Cai	12-Feb-01	84.61	84.71	0.10	0.4	0.0	-100%
276	340	443	11	2	Peninsula	10P	Innisfail - Cai	12-Feb-01	84.91	85.04	0.12	0.6	0.5	-22%
277	440	315	11	2	Peninsula	10P	Innisfail - Cai	12-Feb-01	85.44	85.53	0.08	1.0	0.0	-100%
459	450	472	11	3	Peninsula	20A	Cairns - Mos	29-Jun-02	0.00	0.92	0.92	5.2	5.1	-1%
458	511	481	11	2	Peninsula	20A	Cairns - Mos	29-Jun-02	0.00	0.92	0.92	6.2	4.8	-23%
374	462	433	11	2	Peninsula	20A	Cairns - Mos	05-Dec-01	1.67	3.56	1.89	10.6	11.2	6%
375	415	522	11	3	Peninsula	20A	Cairns - Mos	05-Dec-01	1.67	3.56	1.89	8.8	9.8	11%
377	438	96	11	2	Peninsula	20A	Cairns - Mos	05-Dec-01	3.59	3.77	0.18	1.8	1.1	-36%
378	261	267	11	3	Peninsula	20A	Cairns - Mos	05-Dec-01	3.59	4.80	1.21	0.6	0.9	44%
505	488	470	11	2	Peninsula	20A	Cairns - Mos	03-Feb-03	3.77	6.38	2.61	3.0	1.7	-43%
506	161	372	11	3	Peninsula	20A	Cairns - Mos	03-Feb-03	4.80	6.13	1.33	1.6	2.6	62%
450	510	531	11	2	Peninsula	20A	Cairns - Mos	13-Jun-02	11.35	11.54	0.19	4.6	3.0	-34%
451	520	536	11	3	Peninsula	20A	Cairns - Mos	13-Jun-02	11.35	11.54	0.19	5.8	3.0	-48%
400	38	91	11	3	Peninsula	20A	Cairns - Mos	28-Feb-02	13.05	15.04	2.00	0.6	2.8	362%
401	30	56	11	2	Peninsula	20A	Cairns - Mos	28-Feb-02	13.09	15.04	1.95	3.2	6.5	102%
402	306	381	11	3	Peninsula	20A	Cairns - Mos	28-Feb-02	15.25	16.17	0.91	0.4	0.3	-23%
403	406	397	11	2	Peninsula	20A	Cairns - Mos	28-Feb-02	15.25	16.15	0.90	0.6	0.0	-100%
521	401	352	11	2	Peninsula	20A	Cairns - Mos	02-May-03	16.64	17.68	1.04	1.0	0.5	-52%
520	266	375	11	3	Peninsula	20A	Cairns - Mos	02-May-03	16.64	17.68	1.04	0.4	0.5	20%
389	234	242	11	1	Peninsula	20A	Cairns - Mos	18-Jan-02	20.36	20.62	0.25	0.0	0.0	0%
350	337	441	11	1	Peninsula	20A	Cairns - Mos	30-Jul-01	21.27	21.55	0.28	0.6	0.5	-13%
131	326	385	11	1	Peninsula	20A	Cairns - Mos	10-May-99	25.70	25.75	0.05	0.4	0.2	-50%
535	253	261	11	1	Peninsula	20A	Cairns - Mos	30-Jun-03	25.75	25.81	0.06	0.0	0.0	0%
132	205	213	11	1	Peninsula	20A	Cairns - Mos	10-May-99	25.81	25.83	0.02	0.0	0.0	0%
536	502	504	11	1	Peninsula	20A	Cairns - Mos	30-Jun-03	25.83	25.98	0.15	1.8	0.0	-100%
133	403	424	11	1	Peninsula	20A	Cairns - Mos	10-May-99	25.98	26.18	0.20	0.8	0.2	-75%
113	381	446	11	1	Peninsula	20A	Cairns - Mos	19-Feb-99	26.76	27.07	0.31	0.6	0.2	-67%
90	476	466	11	1	Peninsula	20A	Cairns - Mos	17-Dec-98	28.75	29.80	1.05	2.0	0.8	-60%
20	514	498	11	1	Peninsula	20A	Cairns - Mos	28-Nov-97	29.80	30.15	0.35	2.8	0.6	-79%
91	517	529	11	1	Peninsula	20A	Cairns - Mos	17-Dec-98	30.15	31.40	1.25	4.2	1.6	-62%
21	518	508	11	1	Peninsula	20A	Cairns - Mos	03-Dec-97	32.24	33.21	0.97	4.0	1.4	-65%
22	379	147	11	1	Peninsula	20A	Cairns - Mos	03-Dec-97	35.89	36.10	0.20	0.8	0.4	-50%
539	477	519	11	1	Peninsula	20A	Cairns - Mos	15-Jul-03	41.20	41.90	0.70	1.8	0.5	-70%
454	307	382	11	1	Peninsula	20A	Cairns - Mos	21-Jun-02	41.90	42.00	0.10	0.4	0.3	-15%
455	527	537	11	1	Peninsula	20A	Cairns - Mos	21-Jun-02	42.16	43.45	1.29	6.4	2.0	-68%
23	475	511	11	1	Peninsula	20A	Cairns - Mos	11-Dec-97	43.45	44.26	0.81	3.6	2.8	-22%
540	423	487	11	1	Peninsula	20A	Cairns - Mos	15-Jul-03	44.26	44.47	0.21	0.8	0.0	-100%
24	321	123	11	1	Peninsula	20A	Cairns - Mos	11-Dec-97	44.47	44.94	0.47	0.6	0.6	0%
541	123	438	11	1	Peninsula	20A	Cairns - Mos	15-Jul-03	44.94	45.35	0.41	0.8	1.6	100%
214	279	357	11	1	Peninsula	20A	Cairns - Mos	30-Jun-00	45.35	46.19	0.84	1.2	1.6	36%
28	308	281	11	1	Peninsula	20A	Cairns - Mos	13-Feb-98	46.19	46.60	0.41	0.2	0.0	-100%
542	96	166	11	1	Peninsula	20A	Cairns - Mos	15-Jul-03	46.60	46.82	0.22	0.0	0.5	Large
29	102	168	11	1	Peninsula	20A	Cairns - Mos	13-Feb-98	49.28	49.56	0.28	0.2	0.8	300%
30	162	195	11	1	Peninsula	20A	Cairns - Mos	13-Feb-98	52.82	53.20	0.39	0.4	0.8	100%
507	79	159	11	1	Peninsula	20A	Cairns - Mos	03-Feb-03	59.60	59.89	0.29	0.4	1.3	223%
98	272	344	11	1	Peninsula	21A	Innisfail - Rav	20-Dec-98	42.04	42.16	0.13	0.2	0.2	0%
483	442	403	11	1	Peninsula	21A	Innisfail - Rav	04-Nov-02	42.16	44.41	2.25	1.0	0.0	-100%
371	233	241	11	1	Peninsula	21A	Innisfail - Rav	19-Nov-01	44.84	44.96	0.12	0.0	0.0	0%
37	370	17	11	1	Peninsula	32A	Cairns - Mare	16-Apr-98	1.43	2.64	1.22	3.2	3.6	12%
74	286	54	11	1	Peninsula	32A	Cairns - Mare	09-Sep-98	2.93	3.19	0.26	0.8	1.0	25%
107	529	535	11	1	Peninsula	32A	Cairns - Mare	29-Jan-99	3.19	4.26	1.07	6.2	1.6	-74%
427	501	527	11	1	Peninsula	32A	Cairns - Mare	20-May-02	4.26	4.88	0.62	2.0	0.3	-83%
108	485	514	11	1	Peninsula	32A	Cairns - Mare	29-Jan-99	4.88	5.31	0.43	2.0	0.6	-70%
428	185	373	11	1	Peninsula	32A	Cairns - Mare	20-May-02	5.31	5.56	0.26	0.2	0.3	65%
429	239	247	11	1	Peninsula	32A	Cairns - Mare	20-May-02	5.65	5.71	0.06	0.0	0.0	0%
430	8	5	11	1	Peninsula	32A	Cairns - Mare	20-May-02	5.97	8.00	2.03	3.6	9.2	157%
109	344	348	11	1	Peninsula	32A	Cairns - Mare	29-Jan-99	9.78	10.18	0.40	0.8	0.6	-25%
110	74	48	11	1	Peninsula	32A	Cairns - Mare	29-Jan-99	10.53	10.85	0.32	0.0	0.8	Large
504	287	144	11	1	Peninsula	32A	Cairns - Mare	31-Jan-03	11.04	11.22	0.18	0.4	0.4	7%
447	244	252	11	1	Peninsula	32A	Cairns - Mare	07-Jun-02	49.16	49.17	0.02	0.0	0.0	0%
441	241	249	11	1	Peninsula	32B	Mareeba - Ra	07-Jun-02	0.00	0.02	0.02	0.0	0.0	0%
232	312	284	11	1	Peninsula	32B	Mareeba - Ra	02-Aug-00	12.63	12.69	0.06	0.2	0.0	-100%
319	168	119	11	1	Peninsula	32B	Mareeba - Ra	23-Apr-01	29.98	30.05	0.07	0.4	0.7	83%
369	132	182	11	1	Peninsula	32B	Mareeba - Ra	15-Nov-01	65.84	66.05	0.21	0.2	0.6	183%
370	232	240	11	1	Peninsula	32B	Mareeba - Ra	15-Nov-01	66.21	66.35	0.14	0.0	0.0	0%
443	243	251	11	1	Peninsula	90A	Mareeba-Mt I	07-Jun-02	0.00	0.05	0.05	0.0	0.0	0%

Road Section ID	Road Section Name	SMA Install Date	Data analysis start	Data analysis end	Pre SMA Time	Post SMA Time	Tdist Start	Tdist End	Length
32A	Cairns - Mare	20-May-02	19-May-97	30-May-05	5.00	3.03	5.97	8.00	2.03
10P	Innisfail - Cai	30-Sep-97	29-Sep-92	01-Oct-02	5.00	5.00	16.00	19.02	3.02
32A	Cairns - Mare	16-Apr-98	15-Apr-93	17-Apr-03	5.00	5.00	1.43	2.64	1.22
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	7.42	9.78	2.36
649	Anderson Str	25-May-03	24-May-98	30-May-05	5.00	2.02	2.37	2.51	0.14
10P	Innisfail - Cai	28-Nov-98	27-Nov-93	29-Nov-03	5.00	5.00	73.20	75.32	2.11
32A	Cairns - Mare	29-Jan-99	28-Jan-94	30-Jan-04	5.00	5.00	10.53	10.85	0.32
642	Gordonvale -	07-Aug-03	06-Aug-98	30-May-05	5.00	1.81	12.84	13.29	0.45
32A	Cairns - Mare	09-Sep-98	08-Sep-93	10-Sep-03	5.00	5.00	2.93	3.19	0.26
20A	Cairns - Moss	28-Feb-02	27-Feb-97	30-May-05	5.00	3.25	13.09	15.04	1.95
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	5.91	7.42	1.51
10P	Innisfail - Cai	28-Nov-98	27-Nov-93	29-Nov-03	5.00	5.00	73.20	75.11	1.91
642	Gordonvale -	24-Feb-99	23-Feb-94	25-Feb-04	5.00	5.00	12.72	12.84	0.12
10P	Innisfail - Cai	28-Oct-98	27-Oct-93	29-Oct-03	5.00	5.00	1.59	1.92	0.33
809	Mulgrave Rd	29-Jun-03	28-Jun-98	30-May-05	5.00	1.92	0.72	1.36	0.64
20A	Cairns - Moss	28-Feb-02	27-Feb-97	30-May-05	5.00	3.25	13.05	15.04	2.00
20A	Cairns - Moss	05-Dec-01	04-Dec-96	30-May-05	5.00	3.48	3.59	3.77	0.18
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	5.91	7.42	1.51
642	Gordonvale -	07-Aug-03	06-Aug-98	30-May-05	5.00	1.81	14.34	15.03	0.69
642	Gordonvale -	24-Feb-99	23-Feb-94	25-Feb-04	5.00	5.00	15.03	15.33	0.30
10P	Innisfail - Cai	12-Feb-01	12-Feb-96	30-May-05	5.00	4.29	84.55	85.53	0.98
32B	Mareeba - Ra	23-Apr-01	22-Apr-96	30-May-05	5.00	4.10	29.98	30.05	0.07
20A	Cairns - Moss	11-Dec-97	10-Dec-92	12-Dec-02	5.00	5.00	44.47	44.94	0.47
647	Cairns Weste	08-Jul-03	07-Jul-98	30-May-05	5.00	1.89	13.59	14.02	0.43
647	Cairns Weste	31-Oct-02	30-Oct-97	30-May-05	5.00	2.58	11.50	11.62	0.12
32A	Cairns - Mare	31-Jan-03	30-Jan-98	30-May-05	5.00	2.33	11.04	11.22	0.18
20A	Cairns - Moss	03-Dec-97	02-Dec-92	04-Dec-02	5.00	5.00	35.89	36.10	0.20

Road Section ID	Road Section Name	SMA Install Date	Data analysis start	Data analysis end	Pre SMA Time	Post SMA Time	Tdist Start	Tdist End	Length	
10P	Innisfail - Cai	28-Oct-98	27-Oct-93	29-Oct-03	5.00	5.00	1.59	1.92	0.33	
10P	Innisfail - Cai	30-Sep-97	29-Sep-92	01-Oct-02	5.00	5.00	16.00	19.02	3.02	
10P	Innisfail - Cai	28-Nov-98	27-Nov-93	29-Nov-03	5.00	5.00	73.20	75.32	2.11	
10P	Innisfail - Cai	28-Nov-98	27-Nov-93	29-Nov-03	5.00	5.00	73.20	75.11	1.91	Delete
10P	Innisfail - Cai	12-Feb-01	12-Feb-96	30-May-05	5.00	4.29	84.55	85.53	0.98	
20A	Cairns - Moss	05-Dec-01	04-Dec-96	30-May-05	5.00	3.48	3.59	3.77	0.18	
20A	Cairns - Moss	28-Feb-02	27-Feb-97	30-May-05	5.00	3.25	13.09	15.04	1.95	Delete
20A	Cairns - Moss	28-Feb-02	27-Feb-97	30-May-05	5.00	3.25	13.05	15.04	2.00	
20A	Cairns - Moss	03-Dec-97	02-Dec-92	04-Dec-02	5.00	5.00	35.89	36.10	0.20	
20A	Cairns - Moss	11-Dec-97	10-Dec-92	12-Dec-02	5.00	5.00	44.47	44.94	0.47	
32A	Cairns - Mare	16-Apr-98	15-Apr-93	17-Apr-03	5.00	5.00	1.43	2.64	1.22	
32A	Cairns - Mare	09-Sep-98	08-Sep-93	10-Sep-03	5.00	5.00	2.93	3.19	0.26	
32A	Cairns - Mare	20-May-02	19-May-97	30-May-05	5.00	3.03	5.97	8.00	2.03	
32A	Cairns - Mare	29-Jan-99	28-Jan-94	30-Jan-04	5.00	5.00	10.53	10.85	0.32	
32A	Cairns - Mare	31-Jan-03	30-Jan-98	30-May-05	5.00	2.33	11.04	11.22	0.18	
642	Gordonvale -	24-Feb-99	23-Feb-94	25-Feb-04	5.00	5.00	12.72	12.84	0.12	
642	Gordonvale -	07-Aug-03	06-Aug-98	30-May-05	5.00	1.81	14.34	15.03	0.69	
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	5.91	7.42	1.51	Delete
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	5.91	7.42	1.51	
647	Cairns Weste	10-Sep-99	09-Sep-94	10-Sep-04	5.00	5.00	7.42	9.78	2.36	
647	Cairns Weste	31-Oct-02	30-Oct-97	30-May-05	5.00	2.58	11.50	11.62	0.12	
647	Cairns Weste	08-Jul-03	07-Jul-98	30-May-05	5.00	1.89	13.59	14.02	0.43	
649	Anderson Str	25-May-03	24-May-98	30-May-05	5.00	2.02	2.37	2.51	0.14	
809	Mulgrave Rd	29-Jun-03	28-Jun-98	30-May-05	5.00	1.92	0.72	1.36	0.64	

The SMA trials which have been conducted by the district comprise the following.
This contents of this table will be used in the prioritisation of the current trial areas in section 5.1 of this report.

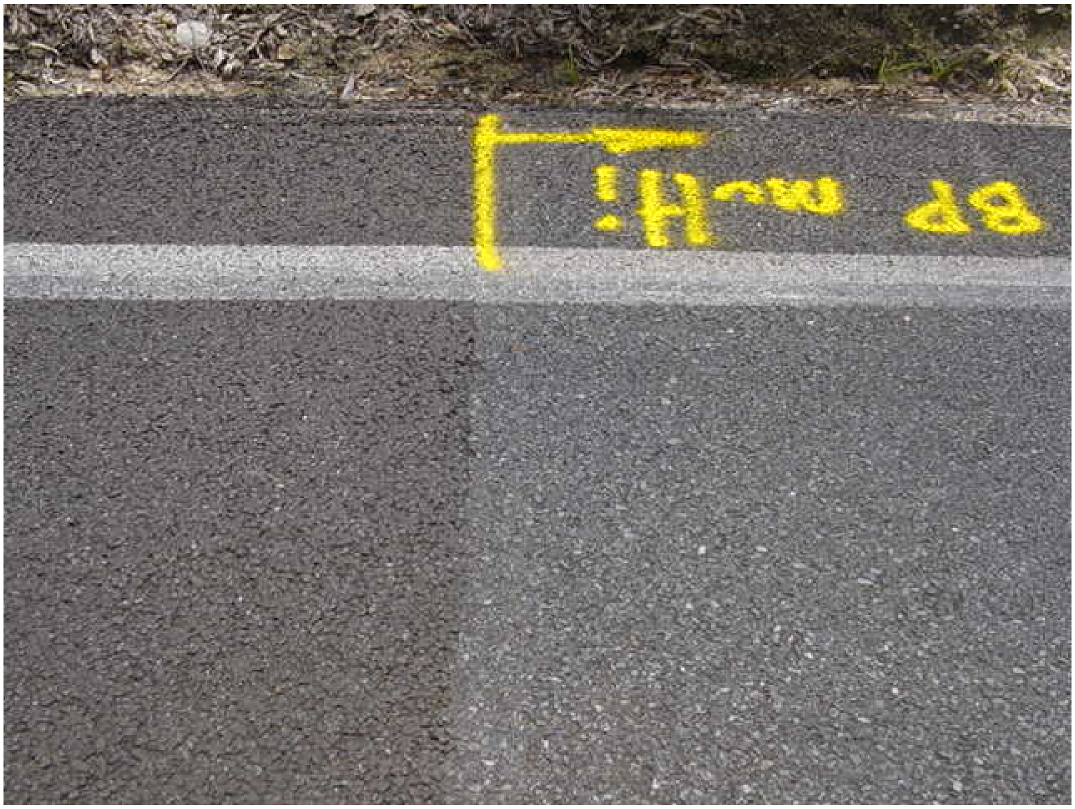
Trial Dates	Location	Trial Type	Suppliers
1997	Eubenangy Swamp	SM14	
2002 - October	Gilles Range	SM12	
2001 - August	Gilles Range	SM14	
2002 - June	Rex Lookout	SM14	
2002 - July	Rex Range	SM14	
2002 - January	Cook Highway Smithfield	SM10	Boral Asphalt
2002- June, July	Yarrabah Range	SM14	Boral Asphalt
	Mulgrave Road	SM12	Pioneer Nth Qld
2003- May	Cook Highway Trinity Beach	SM12	Pioneer Nth Qld
2002 -Feb	Bruce Hwy Wrights Creek	SM14	Pioneer Nth Qld
	Sheraton Street	SM10	Pioneer Nth Qld
2002 - June	Yorkeys Knob		Boral
2002 - June	Main Street Mareeba	SM14, SM10	Boral
2002 - April, May	Kuranda Range		
2003	Bruce Hwy - Gordonvale	SM12	Boral
2004	Brinsmead - Kamerunga Road	SM12	
2005	Cairns Airport		

Priority Sites - listed as all Boral Asphalt Projects

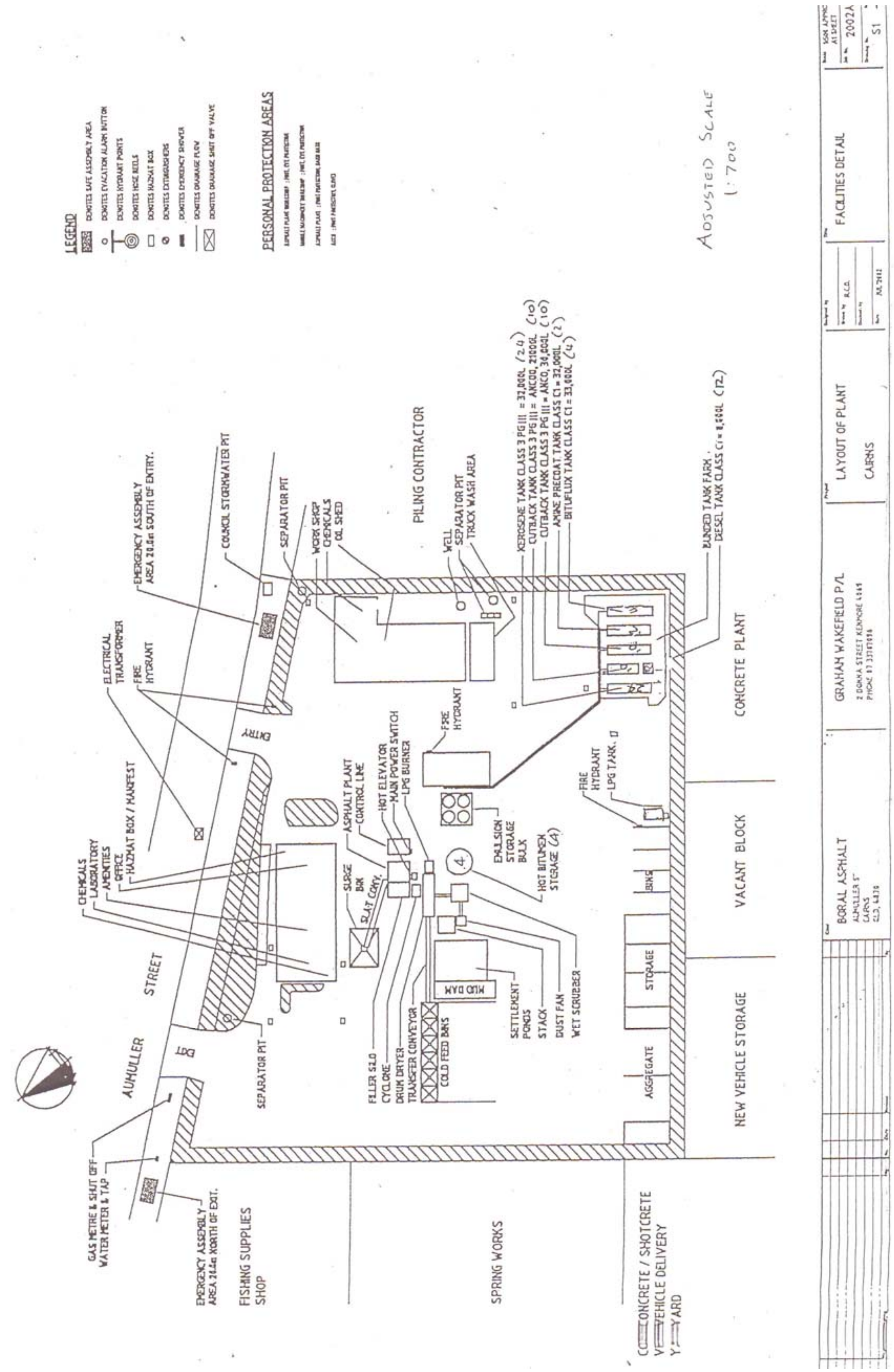
Trial Location	Chainage	Trial Type	Comments
Gillies Range		SM14	Performing well
Brinsmeand - Kamerunga	Section 1	SM12	Flushing, blotchy
Road			
	Section 2		
Rex Range		SM14	Performing well
Cook Highway - Smithfield		SM10	Bleeding
Bruce Highway - Wrights Ck.		SM14	Cracking, under distress

Appendix E – Site Photos


BP AOG
Shell A15



Appendix F – Specific Results


LEGEND
PERSONAL PROTECTION AREAS
AUMULLER STREET
EMERGENCY ASSEMBLY AREA 20.0m SOUTH OF ENTRY.
COUNCIL STORMWATER PIT
SEPARATOR PIT
ELECTRICAL TRANSFORMER
FIRE HYDRANT
PILING CONTRACTOR
WELL
SEPARATOR PIT
TRUCK WASH AREA
BUNDED TANK FARM
CONCRETE PLANT
VACANT BLOCK
NEW VEHICLE STORAGE
FISHING SUPPLIES SHOP
SPRING WORKS
GAS METRE & SHUT OFF
WATER METER & TAP
EMERGENCY ASSEMBLY AREA 20.0m NORTH OF EXIT.
ENTRY
EXIT
CHEMICALS
LABORATORY
AMENITIES
OFFICE
HAZMAT BOX / MANIFEST
ASPHALT PLANT CONTROL LINE
HOT ELEVATOR
MAIN POWER SWITCH
LPG BURNER
FIRE HYDRANT
LPG TANK
EMULSION STORAGE BULK
HOT BITUMEN STORAGE
SURGE BIN
FILLER SILO
CYCLONE
DRUM DRYER
TRANSFER CONVEYOR
COLD FEED BINS
MUD DAM
SETTLEMENT PONDS
STACK
DUST FAN
WET SCRUBBER
STORAGE
AGGREGATE
BINS
WORK SHOP
CHEMICALS
OIL SHED
ADJUSTED SCALE 1:700
BORAL ASPHALT
GRAHAM WAKEFIELD P/L
LAYOUT OF PLANT
CAIRNS
FACILITIES DETAIL
2002A
S1

26/10/2006 4:10 PM

PENINSULA DISTRICT CAIRNS SM12 MRS11.33B MIX DESIGN 01.06.2004

Queensland Government
Department of Main Roads

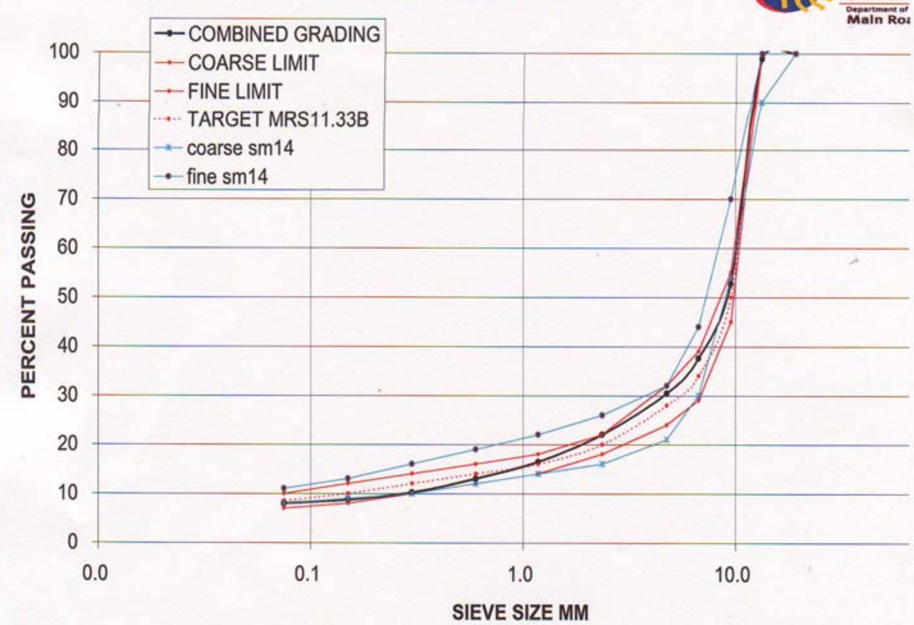
PENINSULA DISTRICT SM12 MIX DESIGN

MEAN AGGREGATE GRADINGS 01.06.04
BORAL CAIRNS MATERIALS

DESIGN CHECKED BY R.LOWE
01.06.04

SEIVE MM SIZE	Mix proportions% / TARGET MRS11.33B	DESIGN 100.0 / COMBINED GRADING	BORAL Tichum Creek 14MM SEAL AGG	BORAL Redlynch 10MM AGGREGATE	BORAL Redlynch 7MM AGGREGATE	BORAL Tichum Ck CRUSHER DUST	NORTHERN SANDS CM SAND	HYD/LIME	FLY ASH	VIATOP PUR FIBRES A02-1103	AB5 BP AUSTRALIA NOMINAL BITUMEN % 6.00 MRS11.33.B Spec lim COARSE LIMIT	FINE LIMIT
			53.00	15.00	5.00	14.00	6.00	3.50	3.50	0.3		
19.0	100	100	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100	100
13.2	100	99	98.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100	100
9.5	50	53	13.5	90.4	100.0	100.0	100.0	100.0	100.0	100.0	45	55
6.7	34	38	1.1	34.3	97.7	100.0	100.0	100.0	100.0	100.0	29	39
4.75	28	30	0.0	7.9	47.5	100.0	98.1	100.0	100.0	100.0	24	32
2.36	20	22	0.0	1.0	4.2	67.9	86.4	100.0	100.0	100.0	18	22
1.18	16	16	0.0	0.0	0.0	38.8	66.9	100.0	100.0	100.0	14	18
0.600	14	13	0.0	0.0	0.0	24.6	43.6	100.0	100.0	100.0	12	16
0.300	12	10	0.0	0.0	0.0	15.2	18.8	100.0	100.0	100.0	10	14
0.150	10	9	0.0	0.0	0.0	10.5	5.7	100.0	98.0	100.0	8	12
0.075	9	8.0	0.0	0.0	0.0	7.8	1.7	100.0	95.0	100.0	7	10

BORAL CAIRNS SM12 MRS11.33B MIX DESIGN



Road System & Engineering Group
Pavements, Materials & Geotechnical Division
35 Butterfield Street Herston Qld 4006

REPORT ON ASSESSMENT OF ASPHALT MIX DESIGN

COMPANY: Boral Asphalt (Cairns) **REPORT NO:** A3246
MIX TYPE: SM12 (A5S) **DATE:** 26/09/02
CODE NO: **PAGE:** 2 of 3

MIX PROPERTIES	SPEC LIMITS	TARGET MIX		TOLERANCE MIXES (Q309)				JOB LIMITS
				COARSE GRADING		FINE GRADING		
		DESIGN	TEST	LOW BINDER	HIGH BINDER	LOW BINDER	HIGH BINDER	
Sample Number			A02-1163	A02-1262			A02-1261	
Binder Content (%) (Q308A)		6.0	6.0#	5.8#			6.2#	5.80-6.20
Grading (% Passing) (Q308A) 37.5 mm								
26.5 mm								
19.0 mm								
13.2 mm		100	100	100			100	100
9.50 mm		49	49	43			55	43-55
6.70 mm		34	34	29			39	29-39
4.75 mm		29	29	25			33	25-33
2.36 mm		22	22	19			25	19-25
1.18 mm		17	17	14			20	14-20
0.600 mm		13	13	11			15	11-15
0.300 mm		11	11	9			13	9-15
0.150 mm		9.5	9.5	8.0			11.0	8.0-11.0
0.075 mm		8.0	8.0	7.0			9.0	7.0-9.0
Stability (kN) (Q305)	6.0min		9.7	8.3			9.8	
Flow (mm) (Q305)	2.0min		3.4	3.6			3.5	
Stiffness (kN/mm) (Q305)	2.0min		2.9	2.3			2.8	
Air Voids (%) (Q311)	1-6**		3.4	6.0			2.2	
VMA (%) (Q311)*	14min		16.8	18.3			15.8	
Voids Filled (%) (Q311)*	-		80.0	67.0			86.0	
Binder Film Thickness (μm) (Q317)	-		10.4					
Effective Binder Volume* (Unabsorbed) (%) (Q311)	12min		13.4	12.3			13.6	
Compacted Density (t/m^3) (Q306C)	-		2.490	2.450			2.506	
Maximum Density (t/m^3) (Q307A)	-		2.578	2.607			2.561	2.541-2.627##
Binder Absorption (%)*	-		0.72	0.72			0.72	

* Determined using an established binder absorption/water absorption relationship.

Variation to Test Method(s)/Remark(s):

Theoretical binder contents used in the calculation of the voids relationships.
Includes ±0.020 t/m^3 tolerance
**As authorised by Peninsula District

CHECKED BY: ______________ **SIGNATORY:** ______________

R.Lowe **I.Berghofer**

F:BAC023/6

The following comments are made regarding the existing SM12 mix designs compliance with MRS11.33B SMART 21_11_03 Supplimentary Specification.

1/ . BORAL

Table 1 Boral SM12 Mix design proportions

Component	Source	Old Mix Proportion %
14mm Seal Aggregate	Boral Tichum Creek Quarry	53
10mm Aggregate	Boral Redlynch Quarry	15
7mm Aggregate	Boral Redlynch Quarry	5
Crusher Dust	Boral Tichum Creek Quarry	14
C/M Sand	Northern Sands	6
Hydrated Lime	Pozzolanic Woodstock	7
Cellulose Fibre	Nutritional Specialists	0.3
A5S Polymer Modified Binder	BP Bitumen	6.0

Note : A minimum Hydrated Lime content of 2% is specified , fly ash and baghouse fines could be introduced .

Table 2 Raw Material Properties

Property	Test Method	Existing Test Results	Comply	Spec Limits MRS11.33 B SMART 21_11_03
Flakiness Index (%)	Q201A	* 12mm =11 * 10mm =12	**Combined grading FI ?**	15 max
Ten Percent Fines Value (Wet) (kN)	Q205B	353	**YES**	150 min
Wet/Dry Strength Variation (%)	Q205C	8	**YES**	35 max
Degradation Factor	Q208B	78	**YES**	40 min
Water Absorption (%)	Q214B	0.95	**YES**	2 max
Weak Particles (%)	Q217	0.4	**YES**	1 max
Polished Aggregate Friction Value	Q203	49	**NO**	50 min
Voids in Dry Compacted Filler (%)	AS 1141.17	46	**YES**	38 min

Note : Boral Tichum Creek PAFV is due for retesting check with quarry if they have done this yet other wise Redlynch Quarry current PAFV result is 54.

The following comments are made regarding the existing SM12 mix designs compliance with MRS11.33B SMART 21_11_03 Supplimentary Specification.

1/ . BORAL

Boral SM12 Mix design proportions

Component	Source	Old Mix Proportion %
14mm Seal Aggregate	Boral Tichum Creek Quarry	53
10mm Aggregate	Boral Redlynch Quarry	15
7mm Aggregate	Boral Redlynch Quarry	5
Crusher Dust	Boral Tichum Creek Quarry	14
C/M Sand	Northern Sands	6
Hydrated Lime	Pozzolanic Woodstock	7
Cellulose Fibre	Nutritional Specialists	0.3
A5S Polymer Modified Binder	BP Bitumen	6.0

Note : A minimum Hydrated Lime content of 2% is specified , fly ash and baghouse fines could be introduced .

Raw Material Properties

Property	Test Method	Existing Test Results	Comply	Spec Limits MRS11.33 B SMART 21_11_03
Flakiness Index (%)	Q201A	* 12mm =11 * 10mm =12	**Combined grading FI ?**	15 max
Ten Percent Fines Value (Wet) (kN)	Q205B	353	**YES**	150 min
Wet/Dry Strength Variation (%)	Q205C	8	**YES**	35 max
Degradation Factor	Q208B	78	**YES**	40 min
Water Absorption (%)	Q214B	0.95	**YES**	2 max
Weak Particles (%)	Q217	0.4	**YES**	1 max
Polished Aggregate Friction Value	Q203	49	**NO**	50 min
Voids in Dry Compacted Filler (%)	AS 1141.17	46	**YES**	38 min

Note : Boral Tichum Creek PAFV is due for retesting check with quarry if they have done this yet other wise Redlynch Quarry current PAFV result is 54.

Mix Design Properties

Mix Properties		Spec Limits		Target Mix Properties	Job Limits	Comply
Binder Content (%) (Q308A)				6.0 (AB5)	5.80 - 6.20	Revise
Grading (% Passing)						
	37.5 mm					
	26.5 mm		MRS11.33B 21_11_03			
	19.0 mm		Grading Limits		Grading Job Limits	
	13.2 mm		100	100	100	YES
	9.50 mm		45-55	49	43-55	Revise to 44-54
	6.70 mm		29-39	34	29-39	YES
	4.75 mm		24-32	29	25-33	YES
	2.36 mm		18-22	22	19-25	YES
	1.18 mm		14-18	17	14-20	YES
	0.600 mm		12-16	13	11-15	YES
	0.300 mm		11-15	11	9-15	YES
	0.150 mm		8-12	9.5	8- 11	Revise to 8 -10 or 9-11
	0.075 mm		7-11	8	7- 9	YES
Stability (kN) (Q305)		6.0 min		9.7	8.3-9.8	YES
Flow (mm) (Q305)		2.0min		3.4	3.5-3.6	YES
Stiffness (kN/mm) (Q305)		2.0min		2.9	2.3-2.8	YES
Air Voids (%) (Q311)		1-6**	2.5-3.5**	3.4	2.2-6.0	YES
VMA (%) (Q311)*		16 min**		16.8	15.8-18.3	YES
Voids Filled (%) (Q311)*		-		80	67-86	-
Binder Film Thickness (µm) (Q317)		-		10.4		-
Effective Binder Volume* (Unabsorbed) (%) (Q311)		14 min**		12.9		NO
Compacted Density (t/m3) (Q306C)		-		2.490		2.450-2.506
Maximum Density (t/m3) (Q307A)		-		2.578		2.541-2.627
Binder Absorption (%)*		-		0.72		

**As specified in MRS11.33B Draft 21/11/2003 Supplimentary Specification for Job Number 160/104/711 .

Note : An increased binder content or lower filler content in the mix design is required to increase binder volume and it is possible to revise the grading however the 0.075mm sieve should remain unchanged, this may compensate for fines generated during manufacture even though the filler content can be lowered by 1% in the mix design.

				100	100	43	29	25	19	14	11	9	8	7	5.80								
SPECIFICATION				100	100	55	39	33	25	20	15	13	11	9	6.20	PROPERTIES							
DATE	LAB No.	37.5	27	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.300	0.150	0.075	% BIT	CD	STAB.	FLOW.	STIFF.	VMA	VFB	VOIDS	MD
CARAVONICA OVERLAY 61330A 2003																							
	CNS03-																						
1/06/03	185			100	100	45	33	30	21	16	13	11	10.0	9.0	6.17								2.547
1/06/03	186			100	100	45	33	29	21	16	13	11	9.7	8.8	5.81								2.546
3/06/03	192			100	99	52	34	29	21	16	13	11	10.0	8.8	5.92								2.538
3/06/03	193			100	99	54	35	28	20	15	12	10	9.4	8.6	6.02								2.543
4/06/03	195			100	99	54	33	27	20	16	13	11	9.8	8.6	6.22	2.420				18.6	78.5	4.0	2.521
4/06/03	196			100	99	62	35	27	20	15	13	11	10.0	8.9	5.95								2.529
5/06/03	197			100	99	50	31	26	19	14	11	9.6	8.5	7.6	5.72	2.446				17.3	78.4	3.7	2.541
5/06/03	198			100	100	54	34	28	21	16	12	10	8.8	7.8	6.09								2.523
6/06/03	203			100	100	51	35	29	21	15	12	9.4	8.2	7.4	6.01								2.549
GILLIES RANGE 61330A 2003																							
	CNS03-																						
30/07/03	317			100	99	50	32	26	24	19	16	11	8.6	6.8	6.22	2.442				18.1	81.6	3.3	2.526
30/07/03	318			100	100	55	35	28	25	19	17	13	9.3	7.4	5.94								2.532
31/07/03	320			100	100	52	33	26	24	22	15	12	8.2	6.4	6.10	2.455				16.8	86.3	2.3	2.512
31/07/03	321			100	99	49	31	26	20	17	14	11	8.9	7.6	6.12								2.533
1/08/03	323			100	99	44	33	28	21	15	12	10	9.1	7.9	6.04	2.458				18.3	78.6	3.9	2.555
1/08/03	324			100	98	52	34	28	22	17	14	12	10.0	8.9	6.25								2.543
2/08/03	326			100	99	47	32	27	20	15	12	11	9.2	7.9	6.25	2.446				18.4	80.7	3.5	2.536
2/08/03	327			100	99	47	33	27	21	16	13	11	9.2	7.9	6.14								2.546
4/08/03	329			100	100	47	32	26	20	15	12	11	9.2	8.0	6.22	2.452				18.3	81.0	3.5	2.540
4/08/03	330			100	99	48	32	26	21	16	12	11	9.2	8.1	6.22								2.542
6/08/03	334			100	98	45	32	27	20	16	13	11	9.3	8.2	6.07	2.465				17.8	81.5	3.3	2.549
6/08/03	335			100	99	47	33	27	21	16	13	11	9.2	8.0	6.19								2.544
7/08/03	344			100	100	47	33	28	21	16	13	11	9.2	8.1	6.13	2.446				18.3	79.4	3.8	2.542
7/08/06	345			100	99	47	32	28	20	17	13	11	9.0	7.9	6.12								2.538
BRINSMEAD - KAMERUNGA RD 61480A 2004																							
	CNS04-																						
8/06/04	99			100	99	57	39	31	23	16	15	12	10.0	8.8	6.03								2.548
8/06/04	100			100	99	56	39	30	22	17	13	11	9.0	7.3	5.80								2.558
9/06/04	101			100	98	49	36	28	20	15	13	11	9.2	7.9	5.94								2.544
9/06/04	102			100	96	51	37	28	21	16	14	13	11.0	9.0	6.03								2.530
10/06/04	109			100	100	53	37	28	21	16	14	11	9.6	8.2	6.10								2.540
10/06/04	110			100	97	47	36	28	21	17	14	12	10.0	8.7	5.93								2.533
				100	100	43	29	25	19	14	11	9	8	7	5.80								
SPECIFICATION				100	100	55	39	33	25	20	15	13	11	9	6.20	PROPERTIES							
DATE	LAB No.	37.5	27	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.300	0.150	0.075	% BIT	CD	STAB.	FLOW.	STIFF.	VMA	VFB	VOIDS	MD
BRINSMEAD - KAMERUNGA RD 61480A 2004 (CONT.)																							
10/06/04	111			100	98	48	36	28	21	17	13	12	11.0	8.9	5.91								2.539
11/06/04	112			100	95	49	35	26	20	15	12	11	9.6	8.4	6.05								2.535
11/06/04	113			100	96	49	35	27	21	16	12	11	9.7	8.2	6.05								2.541
15/06/04	115			100	98	52	35	27	20	15	13	11	9.7	8.3	6.14								2.541
15/06/04	116			100	97	52	35	28	21	15	13	11	9.7	8.2	6.07								2.539
16/06/04	117			100	98	54	37	28	20	15	13	11	9.7	8.5	6.16								2.529
16/06/04	118			100	97	52	37	27	21	15	13	11	9.1	8.2	6.08								2.543
16/06/04	121			100	97	52	36	26	21	16	13	11	9.3	8.2	6.07								2.534
16/06/04	122			100	97	51	36	26	21	15	13	11	9.3	8.2	6.04								2.539
17/06/04	124			100	99	52	36	27	20	16	13	12	10.0	8.6	6.13								2.529
17/06/04	125			100	97	51	36	27	21	16	13	11	9.7	8.2	6.12								2.538
18/06/04	126			100	100	50	35	27	19	16	13	12	10.0	8.8	6.12								2.525
18/06/04	127			100	97	51	35	28	21	16	13	11	9.3	8.2	6.03								2.521
21/06/04	130			100	97	48	34	26	19	15	12	10	9.0	7.8	6.14								2.533
21/06/04	131			100	97	50	35	27	20	16	13	11	9.2	8.3	6.14								2.528
22/06/04	132			100	98	50	36	28	21	17	14	12	11.0	8.9	6.19								2.539
22/06/04	133			100	98	51	36	27	21	16	13	11	9.5	8.1	6.16								2.534
24/06/04	138			100	96	50	35	27	20	16	14	11	10.0	8.3	6.19								2.525
24/06/04	139			100	97	51	36	27	21	16	13	11	9.6	8.0	6.18								2.529
25/06/04	140			100	95	48	36	28	19	14	12	10	9.5	8.4	6.16								2.525
25/06/04	141			100	98	52	35	27	21	15	13	11	9.6	8.2	6.16								2.533
REX RANGE FQA00056 2004																							
	CNS04-																						
14/07/04	166			100	96	50	35	27	20	16	13	11	9.5	8.3	5.98								2.523
14/07/04	167			100	97	50	36	27	21	16	13	11	9.5	8.2	6.03								2.526
15/07/04	168			100	98	45	34	27	20	16	13	11	9.7	8.5	6.00								2.524
15/07/04	169			100	99	43	34	28	20	16	13	11	10.0	8.7	5.90								2.530
16/07/04	170			100	98	48	37	28	21	17	14	12	10.0	8.5	6.06								2.537
16/07/04	171			100	97	50	34	27	21	16	13	11	9.6	8.1	6.03								2.528

SPECIFICATION				100	100	43	29	25	19	14	11	9	8	7	5.80								
				100	100	55	39	33	25	20	15	13	11	9	6.20	PROPERTIES							
DATE	LAB No.	37.5	27	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.300	0.150	0.075	% BIT	CD	STAB.	FLOW.	STIFF.	VMA	VFB	VOIDS	MD
McCOOMBE ST OVERLAY FQA00478 2006																							
	CNS06-																						
31/05/06	185			100	94	55	34	25	19	15	12	10	8.4	7.4	5.98	2.422				17.7	70.4	5.2	2.555
31/05/06	186			100	97	55	30	26	20	16	12	10	8.5	7.4	5.89								2.562
1/06/06	187			100	99	55	33	25	20	16	14	11	9.2	8.3	6.14	2.440				17.6	73.6	4.7	2.559
1/06/06	188			100	100	52	32	26	21	17	14	12	9.8	8.6	5.80								2.520
2/06/06	191			100	99	55	33	26	20	16	14	11	9.0	8.1	6.17	2.431				17.3	75.1	4.3	2.541
2/06/06	192			100	99	51	32	25	19	16	13	10	8.5	7.5	5.90								2.544
5/06/06	194			100	98	47	32	25	19	15	12	10	8.4	7.6	5.85	2.444				15.9	77.2	3.6	2.536
5/06/06	195			100	97	53	33	26	21	16	13	12	10.0	8.8	5.92								2.522
6/06/06	197			100	97	45	31	25	20	16	13	11	9.8	8.7	6.06	2.444				16.1	79.2	3.4	2.528
6/06/06	198			100	98	47	32	26	21	16	13	11	9.8	8.6	6.02								2.527
7/06/06	207			100	98	48	33	27	22	19	14	12	10.0	8.4	5.83	2.441				16.0	76.1	3.8	2.538
7/06/06	208			100	99	47	33	26	22	18	13	11	10.0	8.5	5.85								2.535
13/06/06	217			100	97	47	32	25	19	14	11	9.1	8.3	7.6	5.87	2.432				16.0	76.6	3.8	2.527
13/06/06	218			100	99	54	34	25	21	17	14	11	8.7	7.7	6.03								2.529
14/06/06	220			100	97	49	31	25	20	16	13	11	9.1	8.1	5.96	2.451				15.6	80.6	3.0	2.527
14/06/06	221			100	97	47	30	25	20	16	13	11	9.0	7.8	5.95								2.529
15/06/06	224			100	98	47	32	27	22	18	13	11	10.0	8.1	6.13	2.431				17.5	73.8	4.6	2.548
15/06/06	225			100	99	49	32	27	21	16	13	11	9.3	7.8	5.90								2.533
16/06/06	226			100	98	54	35	27	21	17	14	11	9.1	8.0	6.12	2.417				16.5	77.4	3.7	2.511
16/06/06	227			100	99	49	30	26	19	15	12	10	8.7	7.8	6.08								2.530
17/06/06	229			100	97	48	30	25	19	15	12	9.9	8.1	7.2	6.15	2.442				16.7	77.7	3.7	2.536
17/06/06	230			100	100	54	33	25	20	16	13	11	8.9	7.8	6.11								2.531
10/07/06	281			100	97	51	32	25	19	15	12	9.5	8.3	7.5	6.03	2.424				16.4	77.2	3.7	2.518
10/07/06	282			100	98	54	33	25	19	16	13	11	9.4	8.4	6.07								2.533
11/07/06	284			100	97	55	33	26	20	16	13	11	9.3	8.4	6.10	2.446				16.8	76.6	3.9	2.546
11/07/06	285			100	97	55	33	25	19	174	12	9.9	8.7	7.8	6.17								2.530
18/07/06	292			100	99	53	34	25	19	15	12	9.6	8.7	7.5	6.10	2.439				16.4	81.0	3.1	2.518
18/07/06	293			100	99	53	33	25	20	16	13	10	8.8	7.7	6.12								2.521
19/07/06	296			100	97	55	33	26	19	15	12	9.8	8.4	7.6	6.10	2.431				17.2	77.3	3.9	2.530
19/07/06	297			100	97	54	33	25	19	15	12	9.9	8.3	7.7	6.12								2.526
14/08/06	328			100	98	53	33	27	20	16	13	10	8.0	7.2	5.98								2.527
14/08/06	329			100	97	53	33	25	19	15	12	10	8.9	8.0	6.03								2.534

SPECIFICATION					100	100	68	47	25	22	18	14	12	10	6.30								
					100	90	58	37	17	14	12	10	9	8	5.90	PROPERTIES							
DATE	LAB No.	37.5	27	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.300	0.150	0.075	% BIT	CD	STAB.	FLOW	STIFF.	VMA	VFB	VOIDS	MD
COOK HIGHWAY SMITHFIELD 61070A 2002 **SM10**																							
15/01/2002	08				100	95	59	39	26	20	15	11	9.1	8.0	5.90	2.368				17.4	78.0	3.8	2.462
15/01/2002	09				100	96	61	41	25	22	17	13	10.0	8.1	5.95								2.440
16/01/2002	11				100	97	64	40	24	18	15	13	10.0	8.3	6.05	2.330				18.2	75.4	4.5	2.439
16/01/2002	12				100	96	64	40	25	18	14	12	9.9	8.3	5.95								2.437
17/01/2002	20				100	97	64	41	26	19	15	12	10.0	8.3	6.15	2.379				16.7	85.2	2.5	2.439
17/01/2002	21				100	98	65	42	25	18	14	13	10.0	8.4	6.15								2.441
17/01/2002	22				100	97	63	42	25	19	14	12	11.0	8.3	6.05								2.443
21/01/2002	24				100	98	65	41	27	21	17	12	9.5	8.0	5.90	2.376				17.1	79.4	3.5	2.463
21/01/2002	25				100	96	64	41	26	20	16	11	10.0	8.3	5.90								2.437
22/01/2002	27				100	97	65	42	26	22	17	12	9.7	8.2	5.90	2.370				16.6	87.8	3.0	2.444
22/01/2002	28				100	97	64	42	26	21	17	12	9.9	8.5	6.00								2.441
23/01/2002	30				100	98	65	41	26	22	16	12	10.0	8.7	5.90	2.399				15.6	88.0	1.9	2.445
23/01/2002	31				100	97	64	41	25	23	15	12	10.0	8.4	6.00								2.442
24/01/2002	33				100	97	67	41	26	21	17	13	10.0	8.7	5.95	2.358				17.1	78.8	3.6	2.447
24/01/2002	34				100	97	67	41	26	20	16	12	10.0	8.5	5.95								2.447
24/01/2002	35				100	97	66	41	26	21	17	12	10.0	8.4	6.05								2.448
25/01/2002	37				100	97	66	42	26	20	16	12	10.0	8.4	6.05	2.377				16.6	84.0	2.7	2.442
25/01/2002	38				100	97	67	41	26	20	16	13	10.0	8.5	6.05								2.443
05/02/2002	55				100	94	61	41	25	21	17	13	10.0	8.6	5.95								2.440
05/02/2002	56				100	94	61	41	25	21	17	13	10.0	8.6	5.95								2.441
06/02/2002	57				100	92	62	41	27	21	16	11	9.3	8.4	5.95	2.382							2.460
06/02/2002	58				100	97	65	41	26	21	16	12	10.0	8.5	5.95								2.446
06/02/2002	59				100	97	66	42	26	21	15	11	9.9	8.3	6.05								2.445
06/02/2002	60				100	96	65	41	25	21	16	12	9.9	8.6	6.05								2.435
07/02/2002	63				100	98	65	41	26	21	17	12	9.5	8.3	5.95	2.377						2.9	2.449
07/02/2002	64				100	97	64	41	26	21	17	12	9.8	8.5	6.10								2.443
08/02/2002	65				100	96	63	41	26	20	16	12	9.5	8.8	5.95								2.463
08/02/2002	66				100	97	64	41	26	21	16	12	9.9	8.5	5.95								2.448
28/02/2002	96				100	96	59	40	24	18	14	11	9.5	8.1	5.90	2.321				19.1	69.6	5.8	2.464
28/02/2002	97				100	96	60	41	22	18	14	11	9.6	8.6	5.90								2.457
01/03/2002	99				100	97	60	40	25	22	17	13	11.0	9.2	6.00	2.404				16.5	85.0	2.5	2.465
01/03/2002	100				100	97	61	41	26	21	16	13	11.0	9.3	5.95								2.495

SPECIFICATION				100	100	66	41	32	23	18	16	14	11.5	10	6.30								
				100	90	54	31	24	17	12	12	10	8.5	8.0	5.90	PROPERTIES							
DATE	LAB No.	37.5	27	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.600	0.300	0.150	0.075	% BIT	CD	STAB.	FLOW	STIFF.	VMA	VFB	VOIDS	MD
WRIGHTS CREEK 61070A 2002 **SM14**																							
13/02/02	74			100	97	61	38	30	20	14	12	10	8.7	8.3	5.90	2.383				20.3	67.1	6.7	2.554
13/02/02	75			100	97	62	37	30	20	14	12	10	8.6	8.0	6.00								2.536
14/02/02	78			100	98	65	40	31	21	15	13	11	10.0	9.0	5.95	2.393				19.2	72.0	5.4	2.529
14/02/02	79			100	98	66	40	30	21	16	13	11	9.6	8.5	6.00								2.533
21/02/02	82			100	97	54	34	30	20	16	14	12	9.7	8.2	5.90	2.490				17.3	82.6	3.0	2.567
21/02/02	83			100	97	55	34	30	19	15	14	12	9.7	8.3	5.90								2.550
25/02/02	87			100	97	64	37	28	20	14	12	10	8.6	8.0	6.00	2.372				19.6	70.6	5.8	2.517
25/02/02	88			100	97	64	37	27	20	14	12	11	9.5	8.9	5.90								2.524
26/02/02	90			100	98	65	36	29	21	14	12	10	8.5	8.1	6.20	2.359				21.8	65.0	7.6	2.554
26/02/02	91			100	99	66	37	27	20	13	12	10	9.0	8.3	6.05								2.547
27/02/02	93			100	98	64	35	28	20	15	12	11	10.0	9.3	5.90	2.370				18.7	72.7	5.1	2.497
27/02/02	94			100	98	64	36	27	20	14	13	12	10.0	9.0	5.90								2.526

BORAL RESOURCES (QLD) Pty Ltd

MIX APPROVAL No. INTERIM

SM12 CORE SUMMARY

	RPT No.	CORES	% COMPACTION										PROPERTIES					REFERENCE		
DATE	CNSYY-	Tot.\No.	1	2	3	4	5	6	7	8	9	10	HIGH	LOW	MEAN	STDEV	C.V	RPT NO.	M.D.	NOTES
CARAVONICA OVERLAY 61330A																				
1/06/03	191	13	93.2	93.2	92.9	92.8	91.5	91.4	92.5	90.8	90.8	90.5								
			91.1	91.5	94.1								94.1	90.5	92.0	1.102	91.0	185	2.547	
3/06/03	194	10	90.0	91.9	92.4	91.5	90.9	91.3	92.1	92.0	92.4	91.1	92.4	90.0	91.6	0.718	91.0	192	2.541	
4/06/03	201	10	91.7	91.2	92.4	89.7	89.8	93.3	93.0	92.0	92.8	90.7	93.3	89.7	91.7	1.220	90.7	195	2.525	
5/06/03	202	9	91.0	92.2	93.7	92.5	91.3	94.4	92.3	92.8	91.2		94.4	91.0	92.4	1.081	91.5	197	2.532	
6/06/03	204	7	92.0	89.4	94.0	91.2	93.4	92.5	93.4				94.0	89.4	92.3	1.466	91.2	203	2.549	
GILLIES RANGE 61330A																				
30/07/03	319	7	94.9	94.8	93.8	94.4	91.9	92.8	92.1				94.9	91.9	93.5	1.262	92.5	317	2.529	
31/07/03	322	7	94.2	93.2	94.1	94.8	91.9	91.8	94.7				94.7	91.8	93.5	1.259	92.5	320	2.523	
1/08/03	325	10	91.1	91.2	93.4	92.1	93.9	92.5	91.3	91.4	90.9	92.4	93.9	90.9	92.0	1.027	91.1	323	2.549	
2/08/03	328	7	92.2	94.5	92.2	95.3	91.1	93.0	93.6				95.3	91.1	93.1	1.453	92.0	326	2.541	
4/08/03	331	10	93.6	93.0	92.8	93.5	92.6	93.0	92.0	93.8	93.2	92.0	93.8	92.0	93.0	0.620	92.5	329	2.541	
6/08/03	336	10	92.2	92.3	93.6	91.6	91.0	91.1	91.4	91.7	91.1	91.0	93.6	91.0	91.7	0.818	91.0	334	2.547	
7/08/03	346	7	94.7	91.8	93.0	92.1	92.8	92.8	93.8				94.7	91.8	93.0	0.988	92.3	344	2.540	
BRINSMEAD - KAMERUNGA RD 61480A																				
8/06/04	108	8	93.0	93.0	93.1	92.8	93.1	---	94.5	93.7			94.5	92.8	93.3	0.593	92.9	099	2.553	
9/06/04	114	13	93.7	95.5	94.2	93.1	93.7	93.4	93.1	94.9	94.3	92.9								
			93.6	93.2	93.2								95.5	92.9	93.8	0.755	93.1	101	2.537	
11/06/04	119	13	93.9	93.8	93.5	93.6	93.2	93.9	93.0	94.2	93.4	94.4								
			93.2	95.6	94.5								94.5	93.0	93.9	0.699	93.3	112	2.538	
15/06/04	120	7	92.9	93.5	93.4	93.8	93.4	94.1	94.0				94.1	92.9	93.6	0.414	93.3	115	2.540	
10/06/04	123	13	93.1	93.4	93.4	93.0	93.7	93.4	92.9	93.6	93.0	93.6								
			93.1	93.5	93.0								93.7	92.9	93.3	0.276	93.1	109	2.537	
16/06/04	128	13	94.6	94.2	93.1	94.5	94.7	94.9	93.9	93.3	93.6	94.3								
			95.6	93.4	93.3								95.6	93.1	94.1	0.753	93.4	117	2.539	
17/06/04	129	14	93.9	94.0	93.4	94.4	93.6	94.3	93.9	93.8	93.1	93.9								
			93.3	93.9	93.4	93.7							94.4	93.1	93.8	0.369	93.5	124	2.534	
18/06/04	135	12	93.1	93.2	94.6	94.4	93.8	94.9	93.7	94.6	93.7	93.7								
			95.0	95.1									95.1	93.1	94.2	0.700	93.6	126	2.526	
21/06/04	136	11	93.0	94.2	94.9	94.0	95.1	94.7	93.7	95.2	93.9	95.6								
			94.9										95.6	93.0	94.5	0.775	93.8	130	2.531	

BORAL RESOURCES (QLD) Pty Ltd

MIX APPROVAL No. INTERIM

SM12 CORE SUMMARY

	RPT No.	CORES	% COMPACTION										PROPERTIES					REFERENCE		
DATE	CNSYY-	Tot.\No.	1	2	3	4	5	6	7	8	9	10	HIGH	LOW	MEAN	STDEV	C.V	RPT NO.	M.D.	NOTES
BRINSMEAD - KAMERUNGA RD 61480A (CONT.)																				
22/06/04	137	12	95.0	93.4	95.3	93.7	93.1	93.5	93.5	95.5	93.4	94.3								
			94.0	94.4									95.5	93.1	94.1	0.810	93.4	132	2.537	
24/06/04	149	12	95.3	95.3	93.7	94.2	95.7	94.3	95.5	94.1	93.8	95.6								
			93.5	95.3									95.7	93.5	94.7	0.828	94.0	138	2.527	
25/06/04	150	10	96.1	95.7	94.6	95.1	94.2	93.8	97.2	95.4	97.9	96.4	97.9	93.8	95.6	1.301	94.5	140	2.529	
REX RANGE FQA00056																				
14/07/04	181	8	95.3	95.3	95.4	96.8	95.3	95.7	96.4	97.1			97.1	95.3	95.9	0.743	95.3	166	2.525	
15/07/04	188	10	98.4	95.1	94.0	94.8	95.1	97.3	94.2	98.0	96.1	97.9	98.4	94.0	96.1	1.676	94.7	168	2.527	
16/07/04	189	7	95.6	95.3	94.5	94.7	94.3	93.7	93.6				95.6	93.6	94.5	0.750	93.9	170	2.533	
McCOOMBE ST OVERLAY FQA00478																				
31/05/06	188	8	95.4	94.2	94.5	93.6	93.5	93.4	93.8	94.8			95.4	93.4	94.2	0.709	93.6	185	2.559	
1/06/06	190	7	94.7	94.7	93.1	93.3	93.2	94.1	94.7				94.7	93.1	94.0	0.754	93.4	187	2.540	
2/06/06	193	7	96.3	95.4	94.1	93.9	96.1	96.7	95.2				96.7	93.9	95.4	1.078	94.6	191	2.543	

5/06/06	196	7	94.1	93.1	93.6	96.5	93.2	93.4	95.2				96.5	93.1	94.2	1.258	93.2	194	2.529	
6/06/06	206	7	94.6	96.5	96.7	93.5	93.4	93.6	93.5				96.7	93.4	94.5	1.464	93.4	197	2.528	
7/06/06	209	7	96.0	94.6	94.4	96.0	95.1	96.2	94.9				96.0	94.4	95.3	0.740	94.7	207	2.537	
13/06/06	219	7	95.2	94.4	95.0	94.3	93.2	95.2	95.4				95.4	93.2	94.7	0.772	94.1	217	2.528	
14/06/06	223	7	93.8	95.9	95.7	95.6	95.4	93.4	94.8				95.9	93.4	94.9	0.986	94.2	220	2.528	
15&16/06/06	228	11	94.7	95.7	95.5	96.3	94.2	94.1	94.6	93.7	94.0	93.8								
			95.9										96.3	93.7	94.8	0.922	94.0	224&226	2.531	
17/06/06	231	7	95.1	94.3	93.9	94.7	93.0	95.7	93.1				95.7	93.0	94.3	1.003	93.5	229	2.534	
3&4/07/06	263	8	94.7	95.6	93.6	93.6	94.4	93.6	93.6	94.2			95.6	93.6	94.2	0.725	93.6	261	2.525	
10/07/06	283	7	94.0	95.5	96.3	93.3	93.8	95.3	93.2				96.3	93.2	94.5	1.208	93.6	281	2.526	
11/07/06	286	8	95.0	93.7	95.4	93.6	94.5	93.7	95.8	93.6			95.8	93.6	94.4	0.894	93.7	284	2.538	
18/07/06	295	7	94.1	94.1	95.1	96.1	97.2	95.2	97.0				97.2	94.1	95.5	1.269	94.6	292	2.520	
19/07/06	298	7	93.8	94.2	94.0	94.2	93.7	94.0	94.0				94.2	93.7	94.0	0.186	93.8	296	2.528	

BORAL RESOURCES (QLD) Pty Ltd

MIX APPROVAL No. SM10/98/451

SM10

CORE SUMMARY

	RPT No.	CORES	% COMPACTION										PROPERTIES					REFERENCE		
DATE	CNSYY-	Tot.\No.	1	2	3	4	5	6	7	8	9	10	HIGH	LOW	MEAN	STDEV	C.V	RPT NO.	M.D.	NOTES
COOK HIGHWAY SMITHFIELD 2002 61070A																				
15/01/02	10	21	89.1	89.8	89.0	90.0	92.6	92.4	89	89.2	90.9	87.6								
			90.9	92.2	89.1	88.5	90.6	93.1	88.4	93	90.5	92.1								
			92.1										93.1	87.6	90.4	1.698	88.8	08	2.451	
16/01/2002	19	11	94.7	92.2	92.1	91.5	93.1	94.1	92.9	92.7	92.5	92.3								
			92										94.7	92	92.7	0.945	91.9	11	2.438	
17/01/2002	23	24	91.1	91.2	92.8	91.8	92.3	91.8	91.4	92.1	90.8	92.7								
			91.1	92.8	92.5	91.0	91.3	92.0	90.7	91.1	91.2	91.3								
			92	91.7	91.3	93.1							93.1	90.7			91	20	2.441	
21/01/2002	26	7	91.7	91.4	88.7	91.9	92.7	91.1	92.4				92.7	91.1			90.5	24	2.450	
22/01/2002	29	18	91	91.4	92.1	93.6	92.7	91.6	91.4	92.9	91.2	92.6								
			94.3	92.2	91.3	95.0	95.2	93.0	92.2	93.1			95.2	91			91.4	27	2.443	
23/01/2002	32	8	94.1	91.6	91.1	93.9	92.1	93.0	93.3	94.5			94.5	91.1			92.1	30	2.444	
24/01/2002	36	24	93.1	91.0	91.3	92.2	92.3	91.9	92.3	93.1	91.9	91.8								
			91.9	92.1	91.8	92.9	91.0	93.3	93.3	92.7	92.3	93.3								
			93.9	94.1	92.9	92.4							94.1	91.0			91.7	33	2.447	
06/02/2002	62	18	92.6	93.8	92.8	94.5	93.7	91.8	92.1	93.6	93.4	91.0								
			91.2	91.3	92.3	93.7	93.9	91.1	92.3	94.2			94.5	91.0			91.6	57	2.447	
07/02/2002	67	14	92.9	92.8	93.4	93.5	91.1	91.3	93.1	92.7	91.7	93.0								
			91.1	92.4	92.5	92.3							93.5	91.1			91.7	63	2.446	
28/02/2002	98	2	92.4	94.1									94.1	92.4	93.0	0.85	93.3	96	2.461	

BORAL RESOURCES (QLD) Pty Ltd

MIX APPROVAL No. SM14/02/656

SM14

CORE SUMMARY

	RPT No.	CORES	% COMPACTION										PROPERTIES					REFERENCE		
DATE	CNSYY-	Tot.\No.	1	2	3	4	5	6	7	8	9	10	HIGH	LOW	MEAN	STDEV	C.V	RPT NO.	M.D.	NOTES
WRIGHTS CREEK 61070A 2002																				
13/02/2002	77	7	92.3	92.2	91.9	92.0	92.0	92.2	92.1				92.3	91.9	92.1	0.131	92.0	74	2.545	
14/02/2002	80	9	92.6	92.1	91.7	91.9	92.5	92.3	92.3	92.5	92.7		92.7	91.7	92.3	0.314	92.0	78	2.531	
21/02/2002	84	9	92.5	92.4	92.9	92.3	92.4	92.3	91.8	92.0	91.9		92.9	91.8	92.3	0.319	92.0	82	2.559	
25/02/2002	89	10	91.9	92.1	92.4	92.4	92.6	92.3	92.4	92.5	92.9	92.7	92.9	91.9	92.4	0.271	92.2	87	2.521	
26/02/2002	92	11	92.3	91.9	92.8	92.8	92.5	91.6	91.7	92.4	92.5	91.6								
			91.7										92.8	91.8	92.2	0.452	91.8	90	2.551	

Appendix K Requirements- Checklist

Clause No	Description	Requirement	Comply	Comments
K2	AGGREGATES			
K2.1	Grading	Minimum test frequency monthly or 1 test per 1000 tonnes or 700 m^3 by aggregate supplier together with audit testing by asphalt manufacturer or by asphalt manufacturer.		
K2.2	Flakiness Index	Minimum test frequency 3 monthly arranged by asphalt manufacturer.		
K2.3	Wet Ten Percent Fines	Minimum test frequency yearly but marginal aggregates 6 monthly (ie: <170 kN) arranged by asphalt manufacturer.		
K2.4	Wet/Dry Strength Variation	Minimum test frequency yearly but marginal aggregates 6 monthly (ie: >30%) arranged by asphalt manufacturer.		
K2.5	Degradation Factor	Minimum test frequency yearly but marginal aggregates 6 monthly (ie: <50) arranged by asphalt manufacturer.		
K2.6	Polished Aggregate Friction Value	Minimum test frequency yearly but marginal aggregates 6 monthly (ie: <47) arranged by asphalt manufacturer.		
K2.7	Water Absorption	Minimum test frequency yearly but marginal aggregates 6 monthly (ie: >1.5%) arranged by asphalt manufacturer.		
K2.8	Crushed Faces	Not mandatory if using crushed aggregates only		
K2.9	Weak Particles	Minimum test frequency 3 monthly arranged by asphalt manufacturer.		
K3	FILLER			
K3.1	Imported Filler	Filler supplier certification each delivery, certification includes regular test results for voids in dry compacted filler. Minimum test frequency 3 monthly for voids in dry compacted filler arranged by asphalt manufacturer.		
K3.2	Reclaimed Filler	Minimum test frequency monthly for voids in dry compacted filler arranged by asphalt manufacturer (where reclaimed filler is a specified component of the mix design).		
K3.3	Combined Filler	Minimum test frequency monthly for voids in dry compacted filler arranged by asphalt manufacturer (on combined filler of a mix design).		

K4	BINDER			
K4.1	Delivered Binder	Binder supplier certification each delivery, certification includes test results for • Viscosity at 60°C for conventional and multigrade bitumen's • Softening point and torsional recovery at 25°C for polymer modified binders		
K4.2	Stored Binder	All binders held in storage for more than 2 weeks must be tested for • Viscosity at 60°C for conventional and multigrade bitumen's • Softening point and torsional recovery at 25°C for polymer modified binders		
K5	ASSESSMENT OF ASPHALT QUALITY			
K5.1	Critical sampling and testing points	Conformance tests on • Production batches of asphalt at the point of loading • Compacted asphalt pavement layers		
K5.2	Analysis of Test Data	Parameters assessed for variability as well as compliance with specified requirements and approved mix design details • At the asphalt manufacturing plant - Combined grading results - Binder content - Maximum density • From completed asphalt pavement - Compaction data - Geometric measurements (layer depth alignment, surface levels, roughness) Statistical analysis e.g.: trend diagrams, standard deviations, coefficients of variation		

Appendix G – DMR MRS 11.33b

Main Roads
Supplementary Specification

Peninsula District Interim

SMART Surfacing

(Stone Mastic Asphalt Road Technology)

Queensland Government
Department of Main Roads

MRS11.33B
07_06_04

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Reduced Version – Tables Only

Table 6 – Coarse Aggregate Properties

Property	Unit	Limit	Value
Flakiness Index	Number	Maximum	15*
Ten Percent Fines Value (Wet)	kN	Minimum	150
Wet/Dry Strength Variation	%	Maximum	35
Degradation Factor	Number	Minimum	40
Water Absorption	%	Maximum	2 †
Crushed Particles	%	Minimum	100
Weak Particles	%	Maximum	1
Polished Aggregate Friction Value (PAFV)	Number	Minimum	50

* Maximum of 20 for SM6 and SM8

† For aggregate with water absorption between 2% and 2.5%, project-specific approval may be granted provided that, in the opinion of the Superintendent, a history of satisfactory performance has been demonstrated and suitable adjustments to the mix properties have been made.

Table 7 – Grading Limits for Combined Mineral Aggregates and Added Filler

Sieve Size	Nominal Mix Size % by weight passing sieve		
	SM6	SM8	SM12
13.2			100
9.5		100	45 - 55
6.7	100	45 - 55	29 - 39
4.75	45 – 55	30 - 38	24 - 32
2.36	32 – 36	23 - 27	18 - 22
1.18	22 – 26	18 - 22	14 - 18
0.60	15 – 19	14 – 18	12 - 16
0.30	12 – 16	11 – 15	10 - 14
0.15	10 – 14	9 – 13	8 - 12
0.075	9 – 13	8 – 12	7 - 10

Property	Unit	Limit	Value Stone Mastic Asphalt Nominal Size (mm)		
			SM6	SM8	SM12
Marshall blows	Number	-	50	50	50
Stability	kN	Minimum	6.0	6.0	6.0
Flow	mm	Minimum	2.0	2.0	2.0
Stiffness †[1]	kN/mm	Minimum	2.0	2.0	2.0
Air voids in the compacted mix	%	Minimum Maximum	1.0 tol, 2.5 design 6.0 tol, 3.5 design	1.0 tol, 2.5 design 6.0 tol, 3.5 design	1.0 tol, 2.5 design 6.0 tol, 3.5 design
Voids in mineral aggregate (VMA)	%	Minimum	18.0	17.0	16.0
Binder Volume (unabsorbed) †[2]	%	Minimum	16.0	15.0	14.0
Binder drainage	%	Maximum	0.3	0.3	0.3
Sensitivity to water	%	Minimum	80	80	80
Wheel tracking rate	mm/kCycle	Maximum	TBR	TBR	TBR
Wheel tracking rut depth	mm	Maximum	TBR	TBR	TBR
Mix volume ratio †[2]	-	Maximum	TBR	TBR	1.00

Table 8 – Asphalt Design Requirements

Table 9 – Maximum Permitted Variations from the Approved Production Mix Design

AS Sieve Size (mm)	Maximum Permitted Variation (% by Mass)
≥ 13.2	± 5
9.5	± 5
6.7	± 5
4.75	± 4
2.36	± 3
1.18	± 3
0.60	± 2
0.30	± 2
0.15	± 1
0.075	± 1
Other Properties Binder Content (%)	± 0.2

†[1] Stiffness of the mix = Stability/Flow   †[2] Design mix only   tol = tolerance   TBR = To be recorded

Table 10 – Constituent Material Sample Quantities

Material	Sample Quantity
Binder	8 litres
Fibre	2 kg
Coarse Aggregate - each constituent material of nominal size ≥ 10 mm - each constituent material of nominal size < 10 mm	100 kg 75 kg
Fine Aggregate – each constituent material	50 kg
Added Filler	5 kg
Additive	As requested

Table 11 – Constituent Proportion Limits for Production Mix

Constituent	Limit
Coarse aggregate	For each constituent: ± 20% of the approved proportion up to a maximum of ± 5%† absolute For the total coarse aggregate proportion (>5mm nominal size): ± 5% absolute
Fine aggregate	For each constituent: ± 20% of the approved proportion up to a maximum of ± 5% absolute
Added filler	For each constituent filler proportion and for the total added filler proportion: ± 15% of the approved proportion or ± 0.5% absolute, whichever is the greater

† Where this value is exceeded, the mix design approval may remain current, subject to the approval of the Superintendent, provided that modified constituent proportions are submitted for approval.

Table 12 – Layer Thickness Limits

Asphalt Mix Nominal Size (mm)	Compacted Layer Thickness (mm)	
	Minimum	Maximum
SM6	15	25
SM8	25	30
SM12	35	45

Table 13 – Layer Thickness Tolerances

Asphalt Mix Nominal Size (mm)	Layer Thickness Tolerances (mm)
SM6	± 4
SM8	± 5
SM12	± 7

Appendix H – Northern Development and Comparisons

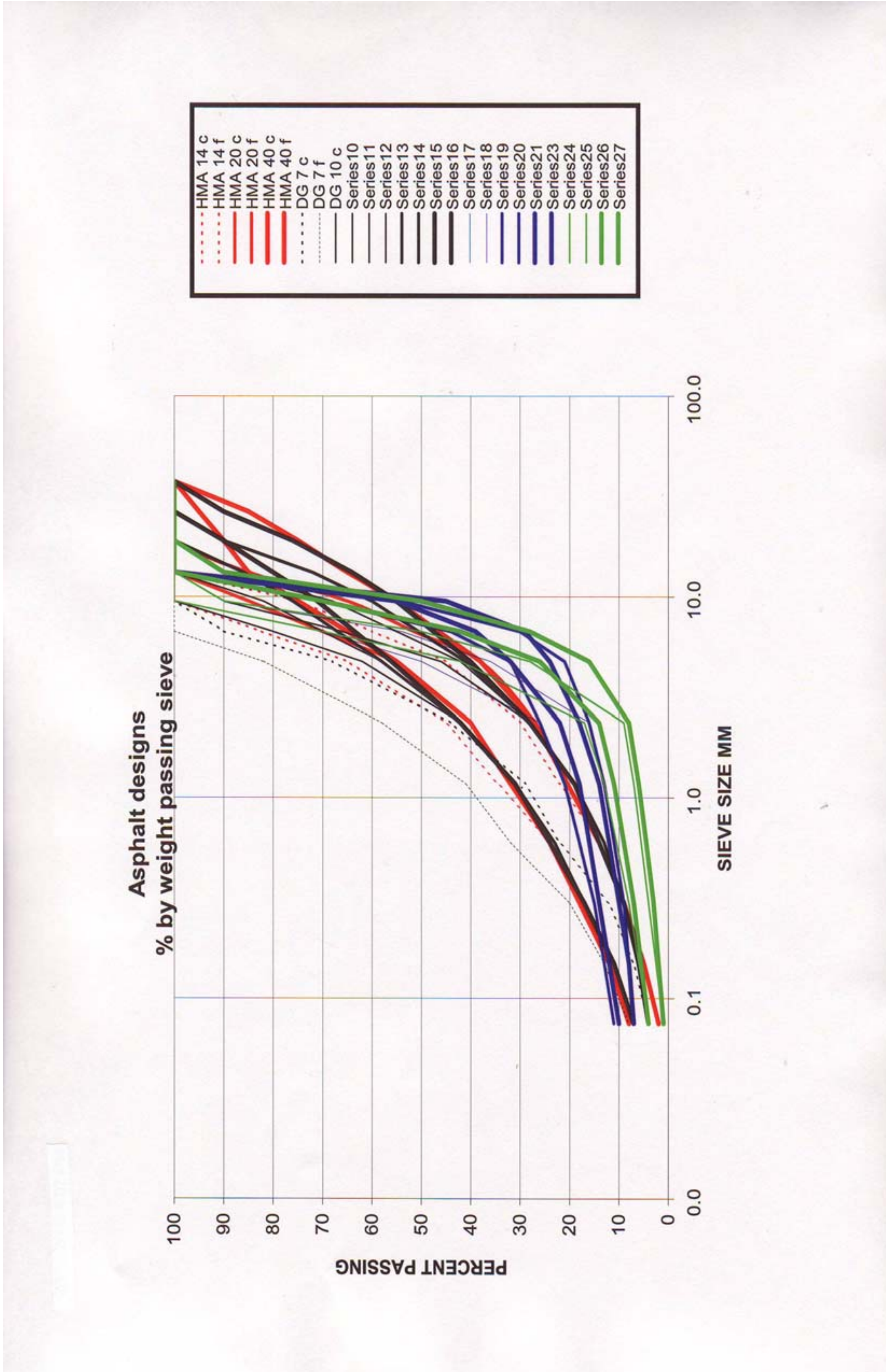
SM12 DEVELOPMENT COMPARISON OF SMA GRADINGS

SIEVE SIZE MM	SM12 TARGET	COARSE LIMIT	FINE LIMIT	BINDER TYPE / MIX TYPE / CODE NO	AB5 / SM10 / SM10/98/451 / BORAL	C320 / SM10 / SM10/98/410 / PNQ	Multigrade / SM10 / SM10/98/472 / PNQ	AB5 / SM14 / SM14/97/374 / BORAL	Multigrade / SM14 / SM14/98/455 / PNQ	C320 / SM14 / SM14/98/411 / PNQ
19	100	100	100		100	100	100	100	100	100
13.2	100	100	100		100	100	100	96	97	97
9.5	50	40	60		96	96	96	60	60	60
6.7	35	27	43		63	63	63	34	38	38
4.75	28	21	35		42	39	39	24	27	27
2.36	22	17	27		21	21	21	17	19	19
1.18	18	14	22		18	16	16	14	15	15
0.600	15	12	18		15	13	13	12	13	13
0.300	13	10	16		12	11	11	11	11	11
0.150	11	8.5	13.5		10.5	10.0	10.0	10.0	9.5	9.5
0.075	9	7.0	11.0		9.0	7.5	7.5	8.5	7.5	7.5
Filler - hydrated lime						7.0	7.0		7.0	7.0
- fly ash										
BITUMEN CONTENT%	6.00				6.10	6.75	6.85	6.10	6.40	6.10
MAXIMUM DENSITY T/M3	2.582				2.449	2.39	2.402	2.436	2.406	2.442
COMPACTED DENSITY T/M3	2.479									
AIR VOIDS%	4	6.5	2.4		5.0	5.6	5.4	5.0		4.5
	11.30									
	14.40									
VMA	17.70									
VF	74.00									
Stab	7.40									
Flow	3.30									
Stif	2.20									

26/10/2006 4:07 PM

Series	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!	#REF!
DESIGN BY R.LOWE& D.HAMILTON													
		MRS 11.30		MRS 11.30		MRS 11.30		MRS 11.33		MRS 11.33		MRS 11.33b	
Mix Type		DG 7		DG 10		DG 14		SM10		SM14		SM12	
SEIVE MM SIZE	Fine Limit	Coarse Limit	Fine Limit	Coarse Limit	Fine Limit	Coarse Limit	Fine Limit	Coarse Limit	Fine Limit	Coarse Limit	Fine Limit	Coarse Limit	Fine Limit
37.5	100	100	100	100	100	100	100	100	100	100	100	100	100
26.5	95	100	100	100	100	100	100	100	100	100	100	100	100
19.0	90	100	100	100	100	100	100	100	100	100	100	100	100
13.2	85	100	100	100	100	90	100	100	100	90	100	100	100
9.5	75	100	100	90	100	68	82	90	100	54	70	45	55
6.7		90	100	66	80			54	70	30	44	29	39
4.75	55	68	82	46	62	42	58	36	50	21	32	24	32
2.36	40	44	58	28	42	28	42	16	26	16	26	18	22
1.18		29	41	19	31	19	31	13	21	13	21	14	18
0.600	24	22	32	13	23	13	23	11	18	11	18	12	16
0.300	18	12	20	9	17	9	17	9	15	9	15	10	14
0.150	12	7	13	6	11	6	11	8	13	8	13	8	12
0.075	8	4	8	4	7	4	7	7	11	7	11	7	10
Maximum Variation to mix design													
37.5	-7	7	-7	7	-7	7	-7	6	-6	6	-6	5	-5
26.5	-7	7	-7	7	-7	7	-7	6	-6	6	-6	5	-5
19.0	-7	7	-7	7	-7	7	-7	6	-6	6	-6	5	-5
13.2	-7	7	-7	7	-7	7	-7	6	-6	6	-6	5	-5
9.5	-7	7	-7	7	-7	7	-7	6	-6	6	-6	5	-5
6.7		6	-6	6	-6	6	-6	5	-5	5	-5	5	-5
4.75	-6	6	-6	6	-6	6	-6	4	-4	4	-4	4	-4
2.36	-5	5	-5	5	-5	5	-5	3	-3	3	-3	3	-3
1.18		4	-4	4	-4	4	-4	3	-3	3	-3	3	-3
0.600	-4	4	-4	4	-4	4	-4	2	-2	2	-2	2	-2
0.300	-3	3	-3	3	-3	3	-3	2	-2	2	-2	2	-2
0.150	-2	2	-2	2	-2	2	-2	1.5	-1.5	1.5	-1.5	1	-1
0.075	-1.5	1	-1	1	-1	1	-1	1	-1	1	-1	1	-1
Binder Content	-0.3	0.3	-0.3	0.3	-0.3	0.3	-0.3	0.2	-0.2	0.2	-0.2	0.2	-0.2
Flakeiness Index M	35.0		30.0		30.0		30.0		30.0		30.0		15.0
Ten % Fines Value Min		150.0		150.0		150.0		150.0		150.0		150.0	
Wet/Dry strength variation Max			35.0		35.0		35.0		35.0		35.0		35.0
Degradation Factor Min		40.0		40.0		40.0		40.0		40.0		40.0	
Water Absorption Max			2.0		2.0		2.0		2.0		2.0		2.0
Weak Particles Max			1.0		1.0		1.0		1.0		1.0		1.0
Crushed faces Min		80.0		80.0		80.0		80.0		80.0		100.0	
PAFV Min		45.0		45.0		45.0		45.0		45.0		50.0	
Number of Marshall blows		50.0		50.0		50.0		50.0		50.0		50.0	
Stability (a) Min		6.0		7.5		7.5		6.0		6.0		6.0	
Flow (b) Min		2.0		2.0		2.0		2.0		2.0		2.0	
Stiffness(a/b) Min		2.0		2.0		2.0		2.0		2.0		2.0	
Air Voids in compacted mix (Design) Min	7.0							2.0	7.5	2.0	7.5	2.5	3.5
Air Voids in compacted mix (Tol.) Min Max												1.0	6.0
Voids in mineral aggregate VMA Min Max		15.0	19.0	14.0	18.0	13.0	17.0	15.0		14.0		16.0	
Voids filled with binder VFB Min Max		58.0	78.0	58.0	78.0	58.0	78.0						
Binder Volume (unabsorbed) Min								13.0		12.0		14.0	
Binder drainage Max									0.3		0.3		0.3
Mix volume ratio Max									record		1.0		1.0
Abrasion Loss Max													
Binder film thickness Min													
Sensitivity to water												80.0	
Wheel tracking rate													
Wheel tracking rut depth													
Compaction < 50mm		90.0		90.0		91.0		91.0		91.0		93.0	

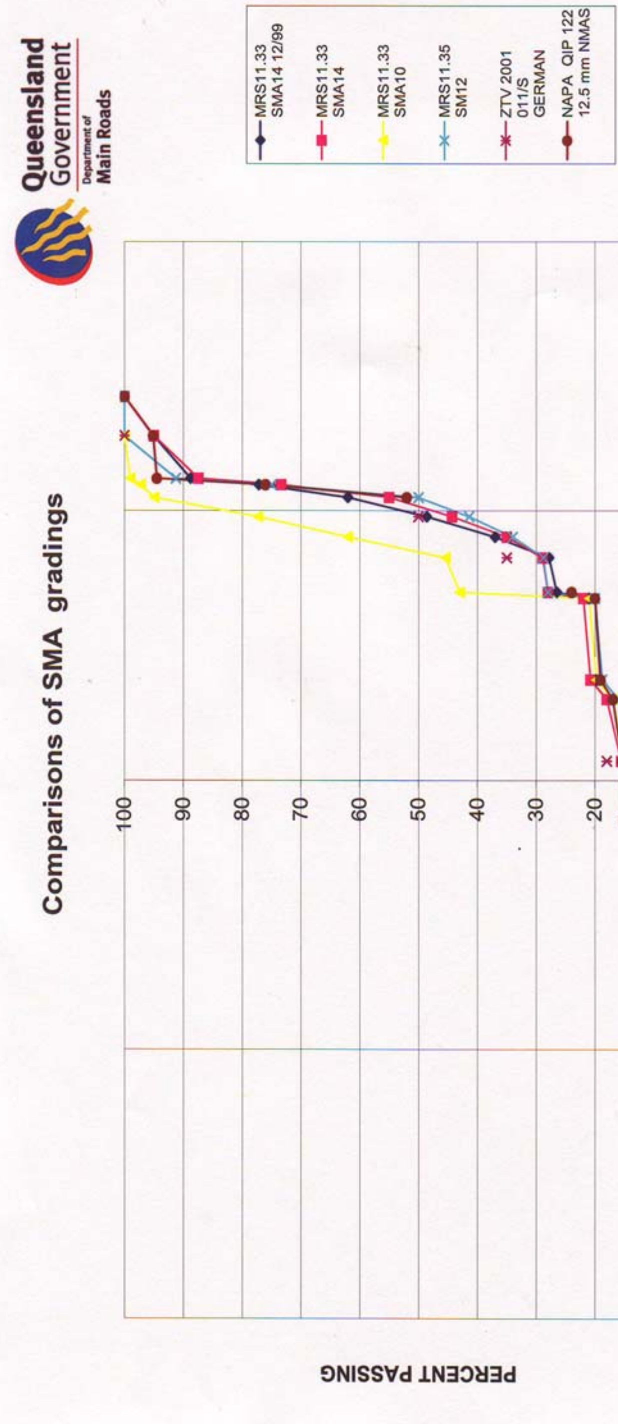
				PIONEER EDMONTON 12MM	PIONEER EDMONTON 10MM	PIONEER EDMONTON 7MM	PIONEER EDMONTON CRUSHER DUST A02-1018	Pozzolanic HYD/LIME A02-1019	Lamura Sand COARSE SAND ED 2976	SAMIcel PUR FIBRES		PIONEER TOWNSVILLE A15E (A02-1108) NOMINAL BITUMEN % 6.00	
	SM10 SM10/98/45 BORAL	SM10 SM10/98/41(PNQ	SM10 SM10/98/47; PNQ	SM14 SM14/97/37 BORAL	SM14 SM14/98/45 PNQ	SM14 SM14/98/41 PNQ	19.00	6.00	0.00	0.3	0.00	COARSE LIMIT	FINE LIMIT
37.5	100	100	100	100	100	100	100.0	100.0	100.0	100.0	100.0	100.0	100.0
26.5	100	100	100	100	100	100	100.0	100.0	100.0	100.0	100.0	100.0	100.0
19.0	100	100	100	100	100	100	100.0	100.0	100.0	100.0	100.0	100.0	100.0
13.2	100	100	100	96	97	97	100.0	100.0	100.0	100.0	100.0	100.0	100.0
9.5	96	96	96	60	60	60	100.0	100.0	100.0	100.0	100.0	45.0	57.0
6.7	63	63	63	34	38	38	100.0	100.0	100.0	100.0	100.0	29.0	39.0
4.75	42	39	39	24	27	27	100.0	100.0	97.4	100.0	100.0	24.0	32.0
2.36	21	21	21	17	19	19	80.7	100.0	75.0	100.0	100.0	19.0	25.0
1.18	18	16	16	14	15	15	56.5	100.0	39.9	100.0	100.0	14.0	20.0
0.600	15	13	13	12	13	13	39.4	100.0	17.1	100.0	100.0	12.0	16.0
0.300	12	11	11	11	11	11	27.5	100.0	7.6	100.0	100.0	10.0	14.0
0.150	10.5	10	10	10	9.5	9.5	18.5	100.0	3.4	100.0	100.0	8.5	11.5
0.075	9	7.5	7.5	8.5	7.5	7.5	12.9	100.0	1.2	100.0	100.0	8.0	10.0


Asphalt designs
% by weight passing sieve
PERCENT PASSING
SIEVE SIZE MM
100
90
80
70
60
50
40
30
20
10
0
0.0
0.1
1.0
10.0
100.0
HMA 14 c
HMA 14 f
HMA 20 c
HMA 20 f
HMA 40 c
HMA 40 f
DG 7 c
DG 7 f
DG 10 c
Series10
Series11
Series12
Series13
Series14
Series15
Series16
Series17
Series18
Series19
Series20
Series21
Series23
Series24
Series25
Series26
Series27

Queensland Government
Department of
Main Roads

COMPARISON OF GRADINGS FOR SMA MIXES

	MRS11.33 SMA14 12/99 QDMR			MRS11.33 SMA14 DRAFT 04/03 QDMR			MRS11.33 SMA10 12/99 QDMR			MRS11.35 SM12 DRAFT '17/10/02 QDMR			ZTV 2001 011/S GERMAN			NAPA QIP 122 12.5 mm NMAS USA		
Sieve mm	COARSE LIMIT	FINE LIMIT	TARGET	COARSE LIMIT	FINE LIMIT	TARGET	COARSE LIMIT	FINE LIMIT	TARGET	COARSE LIMIT	FINE LIMIT	TARGET	COARSE LIMIT	FINE LIMIT	TARGET	COARSE LIMIT	FINE LIMIT	TARGET
26.5																		
19.0	100	100	100	100	100	100						100				100	100	100
13.2	90	100	95	90	100	95	100	100	100	100	100	100	100	100	100	91.076923	99.107692	95.092308
12.5	500	427.33333	0	0	0	633.33333	783.33333	4166.6667	4166.6667	86	96.5	91.25				90	99	94.5
11.2	448	382.89067	0	0	0	567.46667	701.86667	3733.3333	3733.3333	60	90	75				62.266667	89.9	76.083333
9.5	54	70	62	50	60	55	90	100	95	45	55	50				26	78	52
8.0	41.142857	56.071429	48.607143	39.285714	49.285714	44.285714	70.714286	83.928571	77.321429	36.428571	46.428571	41.428571	40	60	50			
6.7	30	44	37	30	40	35	54	70	62	29	39	34						
5.0	22.153846	33.538462	27.846154	24.769231	33.025641	28.897436	38.307692	52.564103	45.435897	24.641026	32.897436	28.769231	30	40	35			
4.75	21	32	26.5	24	32	28	36	50	43	24	32	28				20	28	24
2.36	16	26	21	18	26	22	16	26	21	18	22	20				16	24	20
2.00	15.084746	24.474576	19.779661	17.084746	24.474576	20.779661	15.084746	24.474576	19.779661	16.779661	20.779661	18.779661				15.084746	23.084746	19.084746
1.18	13	21	17	15	21	18	13	21	17	14	18	16				13	21	17
0.71	11.37931	18.568966	14.974138	12.568966	18.568966	15.568966	11.37931	18.568966	14.974138	12.37931	16.37931	14.37931	15	21	18	12.189655	18.568966	15.37931
0.600	11	18	14.5	12	18	15	11	18	14.5	12	16	14				12	18	15
0.300	9	15	12	11	15	13	9	15	12	11	15	13				12	15	13.5
0.250	8.6666667	14.333333	11.5	10.333333	14.333333	12.333333	8.6666667	14.333333	11.5	10	14	12	11	16	13.5	10.333333	15.333333	12.833333
0.150	8	13	10.5	9	13	11	8	13	10.5	8	12	10	9.75	14.125	11.9375	7	16	11.5
0.090	7.2	11.4	9.3	8.2	10.6	9.4	7.2	11.4	9.3	7.2	11.2	9.2	9	13	11	7.8	11.2	9.5
0.075	7	11	9	8	10	9	7	11	9	7	11	9				8	10	9


Comparisons of SMA gradings
Queensland Government
Department of Main Roads
MRS11.33 SMA14 12/99
MRS11.33 SMA14
MRS11.33 SMA10
MRS11.35 SM12
ZTV 2001 011/S GERMAN
NAPA QIP 122 12.5 mm NMAS
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1.0
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80
90
100

References

AAPA, 1998a, "*Surfacing Characteristics of Bituminous – Pavement Work Tips – No. 11",* Australian Asphalt Pavement Association, Melbourne, Australia.

AAPA, 1998b, *"Temperature Characteristics of Binders in Asphalt – Pavement Work Tips – No. 13",* Australian Asphalt Pavement Association, Melbourne, Australia.

AAPA, 2000, Stone Mastic Asphalt "*Design and Application Guide."* Australian Asphalt Pavement Association, Australia. p.14. p.17.

AAPA, 2000b, *"National Asphalt Specification – Edition 1"* Australian Asphalt Pavement Association, Melbourne, Australia.

AAPA, 2005, "*Building on the German Experience"*
Pavements Industry Conference, Australian Asphalt Pavement Association, Australia.

Alderson, A.J., 1998, Development of a Wheel Tracking Test for Australia, *"Focusing on Performance",* "*Proceedings of 1998 AAPA Pavements Industry Conference"*, Surfers Paradise, Australia.

Alford, S., Bullen, F. and Hansen, B., 1997, The Effect of RAP on the Fatigue Characteristics of Residential Asphalt Mixes, *"Our Flexible Future – Pavements for the 21st Century, Proceedings 10th AAPA International Flexible Pavements Conference",* November 16-20, Perth, WA, Paper No. 26.

APRG, 1992, *"Non-structural Asphalt Overlays: A Review of Current Australian Practice, December 1992 Version, APRG Report No. 6",* ARRB Transport Research Ltd, Vermont South, Victoria.

APRG, 1993, *"Stone Mastic Asphalt (SMA)",* August 1993 Version, APRG Technical Note No. 2, ARRB Transport Research Ltd, Vermont South, Victoria.

APRG, 1996, *"Rut-resistant Properties of Asphalt Mixes Under Accelerated Loading: Final Summary Report, APRG Report No. 17/ARR 287",* ARRB Transport Research Ltd, Vermont South, Victoria.

APRG, 1997a, *"AUSTROADS: Selection and Design of Asphalt Mixes; Australian Provisional Guide, APRG Report No. 18",* ARRB Transport Research Ltd, Vermont South, Victoria.

APRG, 2000, *"Evaluation of Field and Laboratory Fatigue Properties of Asphalt Mixes, Report No. AP-T03/00",* AUSTROADS Inc., Sydney, Australia.

Armour, D.H.J., 1998, *"Bituminous Materials",* In Jackson, N and Dhir, R.K. (Eds.), Civil Engineering Materials 4th Edition, Macmillan Education Ltd, London, UK.

ARRB Transport Research, 1998, *"Design of SMA Mixes – Contract Report RC 7005B",* ARRB Transport Research Ltd, Vermont South, Australia.

AS 2150-1995, *"Hot Mix Asphalt"*, Standards Australia, Homebush, Australia.

AS 2891.2.2, 1995, *"Methods of Sampling and Testing Asphalt – Method 2.2: Compaction of Asphalt Test Specimens Using a Gyratory Compactor",* Standards Australia, Homebush, Australia.

AS/NZS 2891.3.1, 1997, *"Methods of Sampling and Testing Asphalt – Method 3.1: Bitumen Content and Aggregate Grading – Reflux Methods,"* Standards Australia, Homebush, Australia.

AS/NZS 2891.1.13.1, 1995, *"Methods of Sampling and Testing Asphalt – Method 13.1: Determination of the Resilient Modulus of Asphalt – Indirect Tensile Method",* Standards Australia, Homebush, Australia.

AS 3882, 1991, *"Rheology – Glossary of Terms and Classifications of Properties",* Standards Australia, Homebush, Australia.

Asphalt Pavement Alliance," *Discover the Mew Asphalt,"- "New Developments for Strategies for Asphalt Pavement."* Lexington.

AUSTROADS, 1992b, Part 4: Guidelines for the Selection of Polymer Modified Binders & Specification for Polymer Modified Binders and Methods for Sampling and Testing, *"Proceedings of the Surfacing Workshop, 16th ARRB Conference, Perth, Western Australia, APRG Report No. 7",* ARRB Transport Research Ltd, Vermont South, Victoria.

AUSTROADS AST 03, 1999, *"Fatigue Life of Compacted Bituminous Mixes Subject to Repeated Flexural Bending',* Austroads Incorporated, Sydney, Australia.

Baburamani, P., 1998, Laboratory Fatigue Performance of Asphalt Mixes – A Preliminary Evaluation, *"Proceeding Conference, 'Focusing on Performance'*", 1998, Australian Asphalt Pavement Association Pavements Industry Conference, Surfers Paradise, Australia, Session 4.

Baburamani, P., 1999, *"Asphalt Fatigue Life Prediction Models – A Literature Review, Research Report ARR 334",* ARRB Transport Research Ltd, Vermont South, Victoria.

Baig, M.G. and Wahhab, H.I.A., 1998, Mechanistic Evaluation of Hedmanite and Lime Modified Asphalt Concrete Mixtures, *"Journal of Materials in Civil Engineering",* Vol. 10, No. 3, ppl153 to 160.

Barksdale, R.D., Alba, J., Khosla, N.P., Kim, R., Lambe, P.C. and Rahman, M.S., 1997, *"Laboratory Determination of Resilient Modulus for Flexible Pavement Design – Final Report Prepared for National Cooperative Highway Research Program",* Transportation Research Board, National Research Council, USA. NCHRP Web Document 14.

Bastow R, Webb M, Roy M, Mitchell J, 2005 – *"An Investigation of the Skid Resistance of Stone Mastic Asphalt layed on a Rural English Country Road Network."* – International Surface Friction Conference – Roads and Runways. Christchurch, NZ.

Bonnaure, F.P., Gravios, A. and Udron, J., 1980, A New Method for Predicting the Fatigue Life of Bituminous Mixes, *"Journal of the Association of Asphalt Paving Technologists",* Vol. 48, pp. 499-529.

Brown, E., Ray, Kandhal, P.S. "Ken", Lee, Dah Yinn & Lee, K., Wayne, 1996, Significance of Tests for Highway Materials, *"Journal of Materials in Civil Engineering",* Volume 8, Number 1, pp 26-40.

BS 3690-1, 1998, *"Bituminous For Building and Civil Engineering, Specification For Bitumen's For Roads and Other Paved Areas",* British Standards Inst., London, UK.

Bullen, F., Jones, A., and Jones, E., 1996, The Use of the Trapezoidal Fatigue Tester in the Evaluation of Asphalt Materials, *"Proceedings Roads 96, Combined 18th ARRB Transport Research Conference / Transit NZ Land Transport Symposium",* Christchurch, NZ, 2-6 September, Vol 2, pp. 17 to 28.

Claxton, M., Lesage, J., Planque, L., and Green, P., 1996, The Use of Binder Rheological Properties to Predict the Performance of Asphalt Mixes, *"Proceedings Roads 96, Combined 18th ARRB Transport Research Conference/Transit NZ Land Transport Symposium",* Christchurch, N.Z., 2-6 September, Vol, pp. 311 to 326.

Cox, J.B., 1994, Refocusing Road Reform, "Proceedings *17th ARRB Conference – part 1",* Gold Coast, Queensland, 14-19 August 1994, pp 75 to 914.

di Benedetto, H. and de La Roche, C., 1998, *"State of the Art on Stiffness Modulus and Fatigue of Bituminous Mixtures",* In: Francken, L. (ed.) Bituminous Binders and Mixes, RILEM Report 17, E & FN Spon, London, UK, pp. 137 to 180.

DMR (Qld), *"Standard Specifications Roads, Polymer Modified Binder, MRS11.18",* 12/99 Edition, Queensland Department of Main Roads.

DMR (Qld), *"Standard Specifications Roads, Dense Graded Asphalt Pavements, MRS11.30",* 12/99 Edition, Queensland Department of Main Roads.

DMR (Qld), *"Standard Specifications Roads, Stone Mastic Asphalt Surfacings, MRS11.33",* Interim 5-97 Edition, Queensland Department of Main Roads.

DMR (Qld), *"Standard Specifications Roads, Stone Mastic Asphalt Surfacings, MRS11.33",* 12/99 Edition, Queensland Department of Main Roads.

DMR (Qld), *"Standard Specifications Roads, Open Graded Asphalt Surfacings, MRS11.34",* 12/99 Edition, Queensland Department of Main Roads.

DMR (Qld), *"Standard Specifications Roads, Fine Gap Graded Asphalt Pavements, MRS11.36",* 12/99 Edition, Queensland Department of Main Roads.

DMR (Qld), 1996, *"Pacific Highway Upgrade – Industry Briefing, 3 April 1996-Pavement Type",* Queensland Department of Main Roads, Nerang, Queensland.

DMR (Qld), 1998a, *"Materials Testing Manual (As amended to December 1998),"* Queensland Department of Main Roads.

- Test Method No. Q306A-1991, *Compacted Density of Dense Graded Asphalt (Wax Sealed).*

- Test Methods No. Q306C-1991, *Compacted Density of Dense Graded Asphalt (Silicone Sealed).*

- Test Methods No. Q306D-1991, *Compacted Density of Open Graded* Asphalt (By Mensuration)

- Test Method No Q307A-1996, *Maximum Density of Asphalt by Water Displacement.*

DMR (Qld), 1999a, *"Heavy Duty Pavement Design: Addendum to the Queensland Pavement Design Manual (Draft)*", September 1999, Queensland Department of Main Roads, Brisbane, Queensland.

DMR (Qld), 2001a, *"Stone Mastic Asphalt – Project R99241-003T",* Draft Report January 2001, Queensland Department of Main Roads, Brisbane, Queensland.

Druschner L, 2005, *"The German origin of SMA"* – Australian Asphalt and Pavements Association, Surfers Paradise, Australia.

EAPA, 1998, *"Heavy Duty Surfaces: The Arguments for SMA",* European Asphalt Pavement Association, Breukelen, The Netherlands.

EAPA, 1999, *"Position Paper on Asphalt Research and Development",* European Asphalt Pavement Association, Breukelen, The Netherlands, http://www.eapa.org/content/research.htm, printed 11 June 1999.

Frederick, R., 1999, *"A Review of the Stone Mastic Asphalt Specification for the Queensland Department of Main Roads MRS 11.33 (5/97)",* Bachelor of Engineering Thesis (Unpublished), School of Civil Engineering, Queensland University of Technology, Brisbane, Australia.

GDOT (nd), *"Stone Matrix Asphalt – Georgia's Experience",* Department of Transportation, State of Georgia, USA.

Harvey, J., Mills, T., Scheffy, C., Sousa, J. and Monismith, C.L., 1994, *"An Evaluation of Several Techniques for Measuring Air-Void Content in Asphalt Concrete Specimens, Jour, of Testing and Evaluation",* ASTM, Vol. 22, pp. 424-430.

Hogan S, Patane J, Lowe R, & Frederick R, 1999, "*Development of Stone Mastic Asphalt for Queensland",* - Technology Transfer Forum, Brisbane. P.3

Ishai, I., and Craus, J., 1996, Effects of Some Aggregate and Filler Characteristics on Behaviour and Durability of Asphalt Paving Mixtures, *"Transportation Research Record 1530",* Trans. Research Board, Nat'l Research Council, No. 1530, pp75-85.

Kadar, p and Donald, G., 1994, Fatigue Testing in Practice – The Equipment and Its Use, *"Proceedings 9th AAPA International Asphalt Conference",* Surfers Paradise, Queensland, November 13-17, 1994, Session 11, Paper 35.

Kandhal, P.S., Lynn, C.A. and Parker, F., 1998, Characterization Tests for Mineral Filler Related to Performance of Asphalt Paving Mixtures, "*Transportation Research Record No. 1638.*"

Kandal, P.S., Parker. F. and Mallick, R.B., 1997, *"Aggregate Tests for Hot Mix Asphalt: State of the Practice",* NCAT Report No. 97-6, National Center for Asphalt Technology, Auburn University, Alabama, USA.

Kim, J.R., Drescher, A. and Newcomb, D.E., 1997, *"Rate Sensitivity of Asphalt Concrete in Triaxial Compression",* Journal of Materials in Civil Engineering, Vol. 9, No. 2, May 1997, pp, 76-84.

Kreide M, Budija M, & Carswell J, 2003, "*The "Original" Stone Mastic Asphalt - The German Experience"* – Conference Proceedings of the 21st ARRB and 11th REAA Conference, Cairns.

Lay, M.G., 1998, *"Handbook of Road Technology Volume 1 – Planning and Pavements, 3rd Edition",* Transportation Studies Volume 8, Gordon and Breach Science Publishers, Amsterdam, The Netherlands.

Lowe, R., 2000, *"Project Progress Report – Stone Mastic Investigation (SM10)",* Internal Report dated 1st August 2000, Queensland Department of Main Roads, Brisbane, Australia.

Lu, Xiaohu, 1997, *"On Polymer Modified Road Bitumen's",* Report TRITA-IP FR 97-30, Division of Highway Engineering, Department of Infrastructure and Planning, Royal Institute of Technology, Stockholm, Sweden.

Lu, X and Isacsson, U, 1997a, Rheological Characterization of Styrene-Butadiene-Styrene Copolymer Modified Bitumen's, "*Construction and Building Materials",* Vol. 11, No. 1, pp. 23-32.

Lu, X and Isacsson, U., 1998, Chemical and Rheological Evaluation of Ageing Properties of SBS Polymer Modified Bitumen's, *Fuel,* Vol. 77 No. 9/10, pp. 961-972.

Luminari, M., and Fidato, A., 1998, *"State of the Art Report on Mix Design and Inventory of Mix Design Methods",* In: Frachen, L., (ed.) Bituminous Binders and Mixes, RILEM Report 17, E and FN Spon, London, U.K. pp 69-101 and 249-314.

Maccarrone, S., Ky, A.V., and Gnanseelan, G.P., 1997a, Permanent Deformation and Fatigue Properties of Polymer Modified Asphalt Mixes, *"Proceedings – Eighth International Conference on Asphalt Pavements",* August 10-14, Seattle, Washington, USA.

Maccarrone, S., Rebbechi, J., Ky, A., 1997b, Evaluation of Stone Mastic Asphalt Performance, *"Our Flexible Future – Pavements for the 21st Century, Proceedings 10th AAPA International Flexible Pavements Conference",* November 16-20, Perth, WA, Paper No. 5.

Matthews, J.M., Monismith, C.L. and Craus, J., 1993, Investigation of Laboratory Fatigue Testing Procedures for Asphalt Aggregate Mixtures, *"Journal of Transportation Engineering",* Vol. 119, No. 4, July/August 1993, pp. 634 to 654.

McGhee, KK & Clark TM. & Reid RA, August 2005 *Final Report "A Performance Baseline for Stone Matrix Asphalt."* VDT, Charlottesville, Virginia.

Mogawer, W.S. and Stuart, K.D., 1995, Effect of Coarse Aggregate Content on Stone Matrix Asphalt Rutting and Draindown, *"Transportation Research Record 1492",* Transportation Research Council, No. 1492, pp 1-11.

Munuandy, R and Umar, Radin R.S., 1997, Modified Rubberized Stone Mastic Asphalt for Malaysian Roads, *"Roads Engineering Association of Asia and Australasia Journal",* No. 10, pp. 8-13.

NAASRA, 1984a, *"Bituminous Surfacing, Volume 2, Asphalt Work, CMPC-8",* national Association of State Roads Authorities, Sydney, Australia.

NAASRA, 1984b, *“Funding the Future: Australian Roads – The Major Findings of the NAASRA Roads Study, 1984”,* National Association of State Road Authorities, Sydney, Australia.

NAS, 1999, *“National Co-operative Highway Research Program – Active Project. Designing Stone Matrix Asphalt Mixtures”,* National Academy of Sciences, USA, http://www2.nas.edu/trbcrp/6972.html, printed 24 March, 1999.

NCAT, 1998a, *“Designing Stone Matrix Asphalt Mixtures, Volume 1- Literature Review, Final Report”,* National Center for Asphalt Technology, Auburn University, Alabama, USA.

NCAT, 1998b, *“Designing Stone Matrix Asphalt Mixtures, Volume III – Summary of Research Results, Final Report”,* National Center for Asphalt Technology, Auburn University, Alabama, USA.

NCAT, 1998c, *“Designing Stone Matrix Asphalt Mixtures, Volume IV – Mixture design Method, Construction Guidelines, And Quality Control Procedures, Final Report”,* national Center for Asphalt Technology, Auburn University, Alabama, USA.

NCAT, 2001, Notes from *“Course in Asphalt Technology – February 9-16 & March 5-9, 2001”,* National Center for Asphalt Technology, Auburn University, Alabama, USA.

NCHRP, 1999, *“Designing Stone Matrix Asphalt Mixtures for Rut-Resistant Pavements, Report 425”,* National Academy Press, Washington, D.C. USA.

Nicholls J.C., 1998b, *“Roads Trials of Stone Mastic Asphalt and Other Thin Surfacings, TRL Report 314”,* Transport Research Laboratory, Crowthorne, UK.

Nicholls JC, Carswell IG, “*The Behaviour of Asphalt in Adverse Hot Weather Conditions”*, - TRL Report 494, U.K.

Nikolaides, A., 2000, Rutting and Volumetric Properties of SMA Mixtures, *“Proc. Instn Civ. Engrs – Transp”,* Vol. 141, Aug 2000, pp 135-141.

Nunn, M.E., 1994, *"Evaluation of Stone Mastic Asphalt (SMA): A High Stability Wearing Course Material, TRL Report No. PR 65",* Transportation Research Laboratory, Crowthorne, UK.

Nunn, M.E., Brown, A. and Lawrence, D., 1999, Assessment of Practical Tests to Measure Deformations Resistance of Asphalt, Pre-print of *"3rd European Conference on Performance and Durability of Bituminous Materials and Hydraulically Stabilized Composites",* University of Leeds, Leeds, UK.

Oliver, J.W.H., 1997, Development of PMB Specifications for Asphalt Rut Resistance, *"Our Flexible Future – Pavements for the 21st Century, Proceedings 10th AAPA International Flexible Pavements Conf."* Nov. 16-20, Perth, WA, Paper No 20.

Oliver, J.W.H., 1999, *"Summary Report on Project NT&E 9804: New Australia/NZ Asphalt Mix Design Procedure, Contract Report RC7092D",* ARRB Transport Research Ltd, Vermont South, Victoria.

Oliver, J.W.H., 2000, *"Implementation of the New Australian Asphalt Mix Design Procedure: Summary Report 1999/2000, Research Report No. ARR 338",* ARRB Transport Research Ltd, Vermont South, Victoria.

Oliver, J.W.H, 2001, *"Modification of APRG 18 Procedures to Improve Deformation Resistance: Analysis of Victorian Dada, T&EPN 018, Contract Report RC2010-B Draft",* ARRB Transport Research Ltd, Vermont South, Victoria.

Oliver, J.W.H. and Alderson, A., 1999, *"The Influence of Added Fillers on Asphalt Properties",* Contract Report No. RC7092B for AUSTROADS, ARRB Transport Research Ltd, Vermont South, Victoria.

Parry, M., Starr, J., Zia, Y. and Copley, S., 1998, Performance Advantages of Polymer Modified Bitumen's (PMBs) with Particular Reference to Sealing Binders, *"Proceedings of the 9th Road Engineering Association of Asia and Australasia Conference (REAAA)",* Wellington, New Zealand, May 3-8, pp 377-383.

Part l, M.N., Vionson, T.S., Hicks, R.G. and Younger, K, (1995), Performance-Related Testing of Stone Mastic Asphalt, *"Asphalt Paving Technology 1995, Journal of the Association of Asphalt Paving Technologists – Proceedings of the Technical Sessions",* Portland, Oregon, USA, Vol. 64, March 27-29, pp 96-127.

Patane J, Bryant P, & Vos R, 2005, "*Development and Performance of New Stone Mastic Asphalt Specification*" – Road System and Engineering Technology Forum, Brisbane.

PrEN 12697-26, 2001-08, *"Bituminous Mixtures – Test Methods for Hot Mix Asphalt – Part 26: Stiffness",* Draft European Standard (August 2001 version).

PrEN 13108-5, 2000, *"Bituminous Mixtures – Materials Specification – Part 5: Stone Mastic Asphalt",* Provisional European Standard.

Pronl, A.C. and Erkens, S.M.J.G., 2001, A Note on Fatigue Bending Tests Using a Haversine Loading, "*Road Material and pavement Design",* Volume 2, Issue 4.

Prowell, B.D. and Schreck, R.J., 2000, Virginia's Use of Laboratory Wheel-Tracking as a Mix Performance Predictor, *"First International Conference "World of Asphalt Pavements'",* Sydney, Australia, Session 3, Paper 2. Published on CD-Rom.

QDOT, 1990, *"Pavement Design Manual – 2nd Edition",* Queensland Department of Transport, Brisbane, Australia.

Queensland Department of Main Roads. - *"Report on Investigation of Bruce Highway at Federal between Middle Creek/ Skyring Creek Roads and Christies Road"* 2005. Technical Expert Review by the Road System and Engineering Group. QDMR.

Read, J.M. and Brown, S.F., 1996, Practical Evaluation of Fatigue Strength for Bituminous Paving Mixtures, *"Eurasphalt & Eurobitume Congress",* Strasbourg, France.

Rebbechi, John, Mangan David, Nicholls Marissa, Bethune John, 2003, ARRB and REAA Conference *"Stone Mastic Asphalt – A Decade of Australia Experience"* ARRB and REAA, Australia.

Remtulla, A., Chik, B. and Man, L., 2001, Effect of Storage and Transport on Polymer Modified Binders, "*Seeking Improvements"* Proceedings 2001 AAPA Pavements Industry Conference, 9th -11th September 2001, Surfers Paradise, Qld.

Richardson JTG, 1999, *"Stone Mastic Asphalt in the UK"* – SCI Lecture Paper. UK.

Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, Dah-Yinn, Kennedy, T.W., 1996, "*Hot Mix Asphalt Materials, Mixture Design and Construction, 2nd Edition",* NAPA Research an Education Foundation, Lanham, Maryland, USA.

RTANSW, 1998, *"Stone Mastic Asphalt Guide, Document No TP-GDL-008",* Roads and Traffic Authority of New South Wales, Sydney, Australia.

Said, S.F., 1988, "*Characteristics of Asphalt Concrete Mixtures, Report No. 583A",* Swedish Road and Traffic Research Institute, Sweden.

Said, S.F., 1996, Fatigue and Stiffness Properties of Roadbase Layer Using Indirect Tensile Test, "*Eurasphalt & Eurobitume Congress",* Strasbourg, France.

Schrimpf, C., 1998, Stone Mastic Asphalt Technology, *"Proceeding Conference 'Focusing on Performance'",* 1998, Australian Asphalt Pavement Association Pavements Industry Conference, Surfers Paradise, Australian, Session 2 Part 2.

Shardlow G., 1997, *"Design and Performance of Stone Mastic Asphalt, Open Graded Asphalt Courses, and Intersection Mixes"* – Conference Proceedings, Volume 1, Session 6, Item 16, APPA, Australia.

Shell, 1978, Standard Method of Test for Determining the Fatigue Life of Compacted Bituminous Mixtures Subjected to Repeated Flexural Bending:M-009, in Harrington, E.T., Leahy, R.B., and Youtcheff, J.S. (Eds.), *"The SUPERPAVE Mix Design System*

Manual of Specifications, Test Methods. and Practices, SHRP A 379", Strategic Highway Research Program, Washington, D.C.

Siniadinos, C., 1994, Development of Equipment for Fatigue Life Testing of Asphalt Beams, *"Proceedings 9th AAPA International Asphalt Conference",* Surface Paradise, Queensland, November 13-17, 1994, Session 11, Paper 35.

Srivastava, A., Hopman, P.C. and Molenaar, A.A.A., 1992, SBS Polymer Modified Asphalt Binder and its Implications on Overlay Design, In: Wardlaw, K.R. and Shuler, S. (Eds), *"Polymer Modified Asphalt Binders, ASTM STP 1108",* American Society for Testing and Materials, Philadelphia, U.S.A., pp 309-329.

Stephenson G, Bullen F, 2002, *"The design, creep and fatigue performance of Stone Mastic Asphalt"* – International Society for Asphalt Pavements, Copenhagen, Denmark.
Troutbeck R & Kennedy C, 2005 "*Review of the use of Stone Mastic Asphalt (SMA) Surfacings by the Queensland Department of Roads".* QDMR, Australia.

Stephenson, G.J., 2002, Use of Stone Mastic Asphalt Mixtures in Road Pavement Maintenance on Construction, *"Doctor of Philosophy Thesis, School of Civil Engineering",* Queensland University of Technology, Brisbane, Australia.

Stroup-Gardiner, M., Newcomb, D.E., Anderson, H. and Epps, J.A., 1988, Influence of Lime on the Fines Content of Asphalt Concrete Mixtures, In: Brown, E.R. (ed), *"New Pavement Materials",* American Society of Civil Engineers, New York, USA.

Van Loon, H. and Butcher, M., 2001, Asphalt Stiffness Master Curves, *"Seeking Improvements, Proceedings 2001 AAPA Pavement Industry Conference",* September 9-11, Surfers Paradise, Australia.

Vic Roads, 1998, *"Economic Evaluation of Maintenance Treatments – Guidelines and User Manual for EVALM Version 2",* Vic Roads, Kew, Victoria.

Vonk, W.C. and Valkering, C.P., 1996, Extension of the Service Temperature Range of Road Binders with SBS Thermoplastic Elastomers, "*"Proceeding Roads 96,* *Combined18th ARRB Transport Research Conference/Transit NZ Land Transport Symposium",* Christchurch, New Zealand, 2-6 September, Vol. 2, pp. 267 to 277.

Wohlk, C.J. and Nielsen, C.B., 1996, Heavy Duty Stone Mastic Asphalt, *"Proceeding Eurasphalt & Eurobitume Congress",* Strasbourg, France.

Wonson, K and Bethune, J., 2000, A Decade of Advances in Australian Asphalt Technology, *"First International Conference 'World of Asphalt Pavements'",* Sydney, Australia, Session 12, Paper 1. Published on CD-Rom.

Woodside, A.R., Woodward, W.D.H., and Akbulut, H., 1998, Stone Mastic Asphalt – Assessing the Effect of Cellulose Fibre Additives, *"Proceedings of The Institution of Civil Engineers, Municipal Engineering",* Vol. 127 No. 3. pp103-108.

Woodward WDH, Woodside AR, & Jellie JH, 2005, "*Early and mid life SMA skid resistance"* - International Surface Friction Conference: Roads and Runways, Christchurch, NZ.

Yeo, R. and Foley, G., 1997, Bituminous and Concrete Surfacings Trial – Report on Performance Monitoring, *"Our Flexible Future – Pavements for the 21st Century, Proceedings 10th AAPA International Flexible Pavements Conference",* November 16-20, Perth, WA, Paper No. 25.

Adelaide International Public Works Conference – 2005
"Stone Mastic Asphalt (SMA) on Australian Local Roads -
The Randwick City Council Experience"

International Surface Friction Conference
"*Roads and Runways"*
Christchurch N.Z. – 2005

North Carolina Department of Transportation
"Stone Mastic Asphalt Mixture Study for Bituminous Concrete Courses – A Design and Performance Evaluation"

AAPA (Australian Asphalt Pavement Association)
International Flexible Pavements Conference – Melbourne 2003

Better Roads
Innovations on Hot-Mix Asphalt

NAPA (National Asphalt Pavement Association)
History of Asphalt

Coastal Engineering Technical Note
"Use of Bitumen in Coastal Structures"

Dr Max G Lay, AM
"Asphalt Mix Designs"

Department of Main Roads - Australia
"Stone Mastic Asphalt: The Facts"