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ANNUAL SUMMARY REPORT

Project Title: P39 Long-life Pavement Alternatives for Queensland Year 1 – 2014/15

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P39 Long-life Pavement Alternatives for Queensland

SUMMARY

This report presents the work that has been conducted in Year 1 of the National Asset Centre of Excellence (NACOE) P39 project. The purpose of the project is to review available literature on long-life pavements and conduct research work to identify opportunities for improvements to the current pavement design methodology in Queensland. Key tasks completed are listed as follows:

- Task 1 – refine project scope
- Task 2 – review literature
- Task 3 – identify existing long-life pavements in Queensland
- Task 4 (a) – review historic performance
- Task 4 (b) – preliminary analysis of the method outlined in the draft Australian Asphalt Pavement Association (AAPA) *Supplement to Austroads Pavement Design Guideline* (2015)
- Task 5 – draft interim report
- Task 6 – scoping for Year 2.

A literature review was conducted for three types of long-life pavements, namely: (i) fully flexible (ii) semi-rigid and (iii) rigid pavements. Despite the different definitions adopted by road agencies on long-life pavements, this generally refers to pavement where there is no cumulative damage over the pavement life and any maintenance is limited to the surfacing layer.

In February 2015, the project team received a copy of the AAPA draft *Supplement to the Austroads Pavement Design Guide* (2015). A number of design thickness calculations have been conducted specifically for the Queensland environment. The proposed AAPA method results in significant reduction in total structural asphalt thickness when compared to typical international pavements and also to the current Austroads/Department of Transport and Main Roads (TMR) design methodology (2013).

One of the limitations of the AAPA method is that it is based on a limited number of Australian calibration sites. In Queensland, AAPA indicated that only two sites were used. To address this, the second part of this study searched for existing long-life pavement candidate sites in Queensland. Unfortunately, no new sites were found along the major arterial roads where a search was conducted using the ARMIS database. The pavement sections often contained either the cement treated subbase or the asphalt base was not old enough (or did not have sufficient cumulative traffic) to meet the AAPA criteria of long-life pavements. Information from the two Queensland AAPA calibration sites is briefly summarised in this report.

The reliability of the AAPA methodology has also been raised by Austroads as requiring further consideration. AAPA is currently revising its method to address the comments raised by Austroads.

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1 INTRODUCTION

Over the past two decades, many research studies have been conducted in the area of long-life pavements. The concept is that various pavement types can last for an indefinite amount of time. Overseas research on asphalt flexible pavements often associates long-life pavement status with when the critical strain is limited to below a fatigue endurance limit (FEL). It was postulated that under this condition, the damage caused by the passage of a vehicle load was offset by the healing of the asphalt material. The maintenance of long-life pavements is limited to the surfacing layer caused by oxidation and other environmental effects. Overseas studies in Britain and the United States have presented evidence that well-constructed pavements often perform well past the intended design life, with no bottom-up fatigue cracking observed as would have been predicted by conventional asphalt pavement thickness design models.

In Australia, the Australian Asphalt Pavement Association (AAPA) began the development of the Asphalt Pavement Solutions for Life (APSfL) in 2011 to address the concerns that the current Australian pavement design procedures might be producing overly conservative asphalt thicknesses.

1.1 Tasks Undertaken in Year 1

The key tasks that have been undertaken in Year 1 of this project are listed as follows:

- Task 1 – refine project scope
- Task 2 – review literature
- Task 3 – identify existing long-life pavement sections in Queensland
- Task 4 (a) – review historic performance
- Task 4 (b) – preliminary analysis of the method outlined in the draft *Supplement to Austroads Pavement Design Guideline* (AAPA 2015)
- Task 5 – draft interim report
- Task 6 – scoping for Year 2.

This project first presents the literature findings of the long-life pavement concept. It is noted that many different pavement types can be classified as long-life pavements if the maintenance works are limited only to the surfacing. Pavement types that can be associated with long-life status include fully flexible pavement, semi-rigid pavement and rigid pavements. After the completion of Tasks 1 and 2, a hold-point was placed before the commencement of Task 3. This hold-point was necessary because the draft AAPA Asphalt Pavement Solutions for Life (APSfL) supplement was not available to the project team until February 2015.

A meeting was held between ARRB/TMR and AAPA on 16 February 2015 to clarify the scope for Year 1 of the P39 NACOE project. It was decided that the project should focus only on the long-life fully flexible pavement options (i.e. full depth asphalt pavements). The outcome of the meeting was to replace Tasks 4 and 5 in the original proposal with Task 4 (a) and Task 4 (b) as detailed above.

1.2 Report Structure

Section 1 outlines the tasks undertaken in Year 1 and presents the structure of the report. Section 2 presents the findings from the literature review on different types of long-life pavements. The review includes the current available research studies undertaken both nationally and internationally. Section 3 presents a brief outline of the AAPA long-life pavement design methodology. A comparison of the pavement design thickness between the proposed AAPA draft design supplement and the current TMR design methodology is presented in Section 4.

Section 5 then discusses the use of the ARMIS database to identify pavements within the Queensland state-controlled road network that can potentially be classified as long-life pavements.

During the meeting with AAPA representative in February 2015, it was confirmed that the majority of the Australian calibration sites are located in New South Wales (NSW) and only two sites from Queensland were used. Lastly, Section 6 presents the summary of findings and the proposed scope of works for future years of this project.

2 LITERATURE REVIEW

One of the tasks in this project is to undertake a comprehensive literature review of different types of long-life pavements that are available in Australia and overseas. In European countries, the term ‘long-life’ pavements has often been used. In the USA, ‘long-life’ pavements are often referred to as ‘perpetual’ pavements. In this report the term ‘long-life’ pavement is generally used.

Various research studies have been carried out in Europe and the USA where many in-service pavements were found to outlast their original nominated design lives. For asphalt flexible pavements, in traditional pavement deterioration models, it is expected that deflection increases with time as the number of cumulative traffic cycles increases.

For long-life pavements that are fully-flexible (i.e. full depth asphalt pavements), evidence indicated that deflection often does not follow this trend and tends to stiffen up with time as oxidation and densification of asphalt layers takes place over the years. In recent research studies, this phenomenon is often explained using the FEL concept, which is the threshold when the damage caused by traffic load repetition is offset by healing of the material. The development of the FEL will be presented in Section 2.2.

In this section, different types of ‘long-life’ pavements will be presented. Typical design thicknesses and design features will also be discussed.

2.1 Definition of Long-life Pavements

The definition of long-life pavement is different around the world and is also dependent on the types of long-life pavement. Definitions adopted for different types of long-life pavements are summarised in Table 2.1.

Table 2.1: Definitions of long-life pavements

Region/organisation	Pavement type	Long-life pavement definitions
Strategic Highway Research Program (SHRP) 2, USA	Asphalt flexible	SHRP 2 has defined long-life pavements as those lasting in-service for 50 years or longer without needing major rehabilitation.
Asphalt Pavement Alliance, USA (Asphalt Pavement Alliance 2015)	Asphalt flexible	Perpetual pavement (a long-life asphalt pavement) was defined as ‘an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement’.
UK, Transport Research Laboratory (TRL) (Nunn et al. 1997)	Asphalt flexible	‘Well-constructed roads that are designed above a threshold strength will have a life in excess of 40 years. These roads are referred to as long-life roads.’
AAPA, Australia (Sullivan et al. 2014)	Asphalt flexible	A long-life asphalt pavement (LLAP) was defined as a site which was greater than 30 years old, had experienced in excess of 8 x 10^7 ESA loadings and had no structural damage.
European Long-Life Pavement Group (ELLPAG) – FEHRL phase 1 report (FEHRL 2004)	Asphalt flexible	‘A long-life pavement is a type of pavement where no significant deterioration will develop in the foundations or the road base layers provided that correct surface maintenance is carried out.’ ‘Deterioration: This includes whatever the network manager considers important e.g. significant cracking or (progressive) deformation in the structural layers of a fully-flexible pavement; for other types of pavement ‘deterioration’ could be quite different.’
FHWA US (Lubinda, Liu, Scullion 2010)	Asphalt flexible	Over 3 x 10^7 18-kips (80 kN) ESALs over a 20-year period.

Region/organisation	Pavement type	Long-life pavement definitions
Transportation Research Board (TRB), US (TRB 2001)	Asphalt flexible	'These pavements have thick asphalt layers overlying sound foundations comprised of granular base and sub base materials or directly on a prepared sub grade. A conscientious process of design and construction formulated to preclude structural rutting and bottom-up fatigue cracking will result in a pavement system where the distresses are confined to the surface and easily maintained or corrected.'
ELLPAG – FEHRL Phase 2 (FEHRL 2009)	Semi-rigid	'A well designed and well-constructed pavement where the structural elements last indefinitely provided that the designed maximum individual load and environmental conditions are not exceeded and that appropriate and timely surface maintenance is carried out.'
Europe and Canada (FHWA 2007a)	Concrete	'Concrete pavement is considered synonymous with 'long-life'. It is expected that the concrete pavements be strong and durable, provide service lives of 25, 30 or more years before rehabilitation or replacement, and require little if any maintenance intervention over the service life.'
Tayabji and Lim (FHWA 2007b)	Concrete	A useful definition of long-life concrete pavement in the USA is summarised as follows: ▪ Original concrete service life is 40+ years. ▪ Pavement will not exhibit premature construction and materials-related distress. ▪ Pavement will have reduced potential for cracking, faulting and spalling. Pavement will maintain desirable ride and surface texture characteristics with minimal intervention activities, if warranted, for ride and texture, joint resealing and minor repairs.
US (Rohne 2009)	Concrete	'Definition of long-life concrete pavements in other states vary but typically it refers to pavements with service lives of 30 to 40 years or more.' 'The Minnesota Department of Transportation began building high-performance concrete pavements (HPCP) in 2000 under the FHWA TE-30 program. Since that time, the HPCP design has becomes the standard for most urban high-volume highways. The current design service life for Minnesota's HPCP is 60 years.'

Despite the specific long-life pavement definitions that have been adopted by different road agencies, the essential expectations are that the long-life pavement will need to be designed and constructed to the best-practice and only minimal repair treatments are required over the long design period adopted. Despite the differences in definitions adopted by different road agencies, features that are expected from a long-life pavement are listed as follows:

- well-designed and well-constructed
- require regular surface maintenance intervention over the service life
- typical service life expected to exceed a minimum of 40 years or longer.

For the purpose of this project, the following definition is used for long-life pavements:

> A long-life pavement is a type of pavement in which damage to the foundations or the road base layers will not accumulate under its predicted annual traffic loading provided that correct surface maintenance is carried out.

2.2 FEL, Rest Period and Healing

2.2.1 Concept of Endurance Limit

The concept of FEL assumes that there is a threshold strain value below which fatigue damage will not occur or can be healed during unloading. A FEL is believed to exist in asphalt by some authors, the FEL value to be adopted for design is still a subject of constant debate and on-going study. In 1972 the concept of FEL was initially proposed with a single endurance limit of 70 micro-strain for LLAP design (Monismith & McLean 1972). This single value approach forms the basis of the early long-life pavement design guidelines and was used by several US states.

Past literature reviews indicated that the value of the endurance limit has not been agreed universally, with value typically between 70 and 250 micro-strain. Over the years, researchers have come up with different FELs. Some of the limits are summarised below:

- 70 micro-strain ($\mu\varepsilon$) based on research work by Monismith and McLean (1972)
- 200 $\mu\varepsilon$ based on research conducted on in-service pavements in Japan (Nishizawa, Shimeno & Sekiguchi 1997)
- 96–158 $\mu\varepsilon$ for a long-life pavement in Kansas by back-calculated Falling Weight Deflectometer (FWD) data (Wu, Siddique & Gisi 2004)
- 115–250 $\mu\varepsilon$ determined through uniaxial laboratory testing (Bhattacharjee, Swamy & Daniel 2009)
- 100–400 $\mu\varepsilon$ for 11 Superpave mixtures and presented as a function of flexural modulus (Thompson & Carpenter 2006).

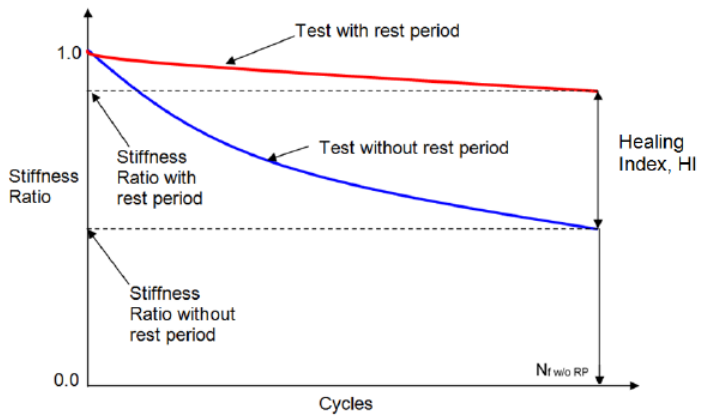
A recent National Cooperative Highway Research Program (NCHRP) study on endurance limits suggested that there is no single value that applies to all conditions. The endurance limit varies depending on binder grade, binder content, air voids, temperature and the rest period between loading cycles (Souliman & Mamlouk 2014). The study also concluded that the hot mix asphalt (HMA) stiffness (modulus) can be a surrogate property that takes into account all primary mix variables. However, this concept needs to be used carefully since increasing or decreasing both air void and binder content can offset the effect of each other and result in similar stiffness values with different endurance limits.

2.2.2 Rest Period and Healing

The existence of FEL is associated with the healing phenomenon in the asphalt material. The FEL of asphalt is a function of the rest period between load pulses and the testing temperature. The longer the rest period between pulses, the more likely an asphalt will heal from damage caused by the cumulated load repetitions.

A typical stiffness ratio versus number of load cycle curves is shown in Figure 2.1. This is the ratio between the stiffness at a particular loading cycle and the initial stiffness. It has been shown that by incorporating rest period in the fatigue testing, more cycles are required for the same reduction in stiffness ratio. The healing index (HI) is then defined as the difference between the stiffness ratios for tests with and without a rest period.

Figure 2.1: Illustration of the relationship between stiffness ratio, number of cycles and healing



Source: Souliman and Mamlouk (2014).

Laboratory fatigue experiments, particularly experiments incorporating rest periods between loading cycles, can be time consuming. There are a number of methods available, such as the plateau value approach, the pseudo strain analysis approach, the viscoelastic continuum damage model and the NCHRP 9-44A method. Overviews of the different methods were discussed by Underwood, Zeiada & Kaloush (2014). The researchers also undertook a comparison study of the endurance limit derived using the different methods. Different methods provide different estimates for the material endurance limit value with generally better agreement at the lower temperature range.

2.3 Long-life Pavement Types

Depending on the definition adopted for the long-life pavements, various pavement types can be potentially defined as long-life pavements. In this literature review, the three types of long-life pavements identified by ELLPAG (FEHRL 2004, FEHRL 2009) were listed as follows:

- fully flexible (full-depth asphalt)
- semi-rigid
- rigid (concrete).

2.3.1 Fully Flexible (Full-depth Asphalt)

A typical long-life pavement configuration in the USA is shown in Figure 2.2. These pavements often consist of the following key layers:

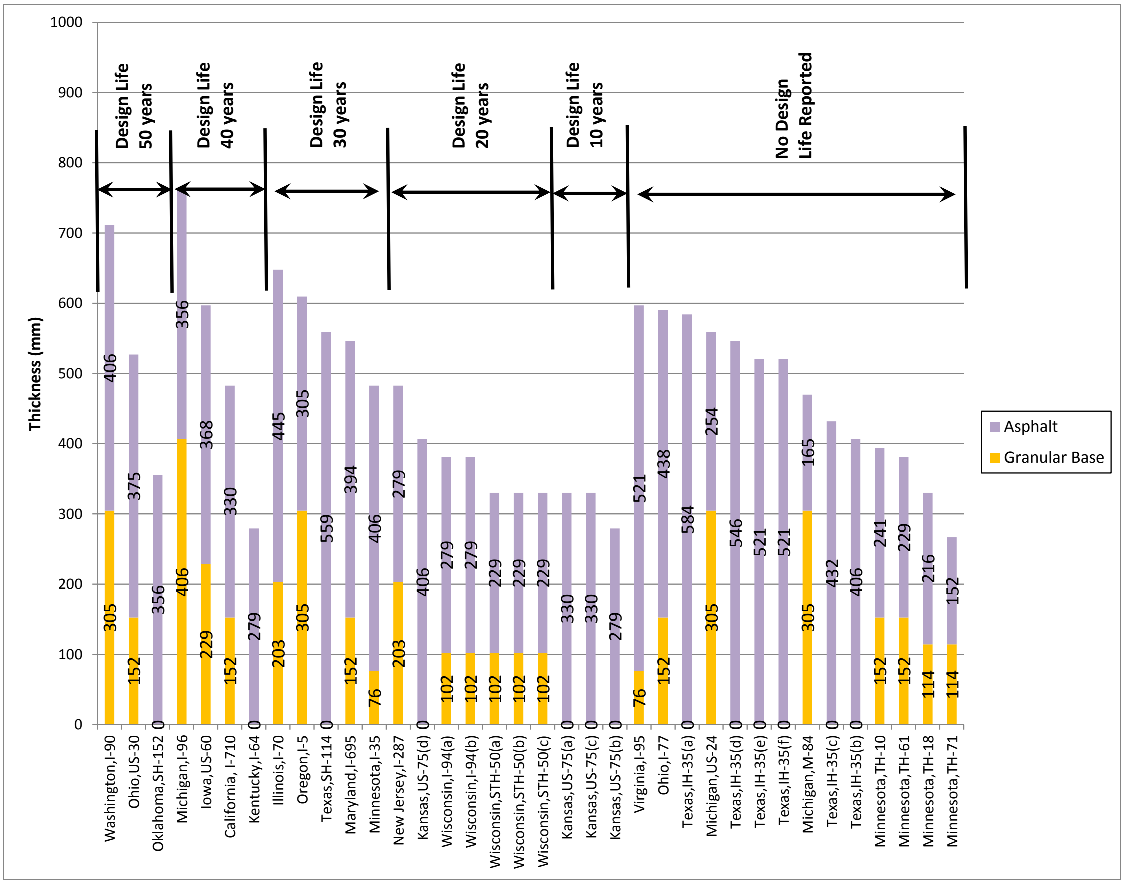
- rut resistant wearing course
- high modulus rut resistant base course
- fatigue resistant subbase asphalt course
- pavement foundation (treated soil or granular subbase).

Figure 2.2: Schematic of "perpetual" pavement

Source: Tarefder and Bateman (2012).

A study by Tarefder and Bateman (2012) conducted a survey in the USA to determine the typical "perpetual" pavement configurations in different states. Pavement thicknesses reported in the study are summarised in Figure 2.3.

Figure 2.3: Asphalt and granular base layer thicknesses for typical US heavy duty pavements



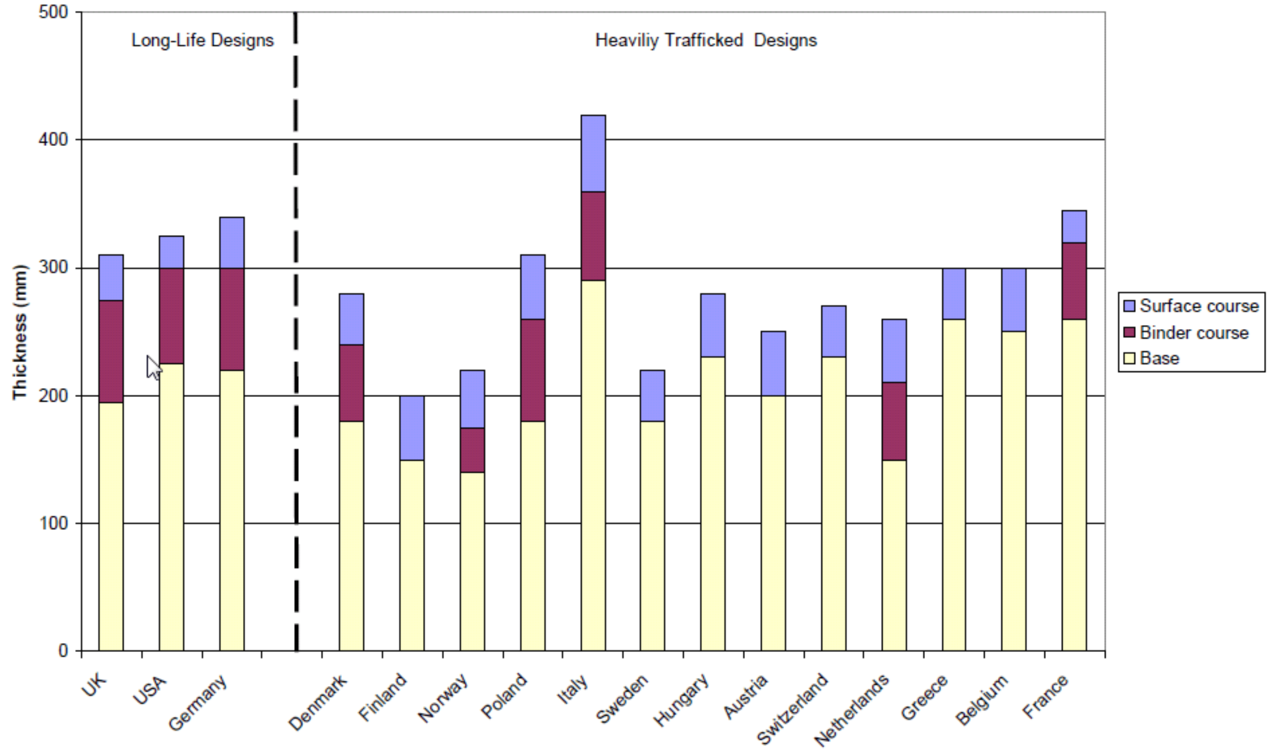
Source: Tarefder and Bateman (2012).

In Britain, fully flexible long-life pavement consists of an asphalt layer of adequate thickness to reach an indefinite fatigue life. In 2006, the UK Highway Agency required a minimum 150 mm thick layer of cement treated subbase layer to be provided for long-life pavements with traffic over 80 million standard axles (MSA).

Ferne and Nunn (2006) undertook a study on long-life pavements across different European countries. Figure 2.4 summarises the total pavement thickness for each of the heavily trafficked pavements. Three of the designs produced by the UK, USA and Germany are defined as long-life designs, which have total asphalt thicknesses between 310 mm and 340 mm. For other countries, the pavements for the heavy trafficked roads have total pavement thicknesses between 200 mm and 420 mm.

The FEHRL study in 2004 also reported that the participants of the European study indicated that the preferred approach in developing a long-life pavement design method involves a strong preference among members to use improved materials and/or design to prevent the expected modes of deterioration from occurring (accounts for 84% of all respondents). Only 14% of the respondents indicated that the long-life pavement should be designed based on a threshold strength and where there will be no accumulated wear beyond the threshold. Only 3% of the respondents preferred the extrapolation of the current design curves.

Figure 2.4: European pavement designs for national maximum design traffic levels



Source: Ferne and Nunn (2006).

2.3.2 Semi-rigid Long-life Pavements

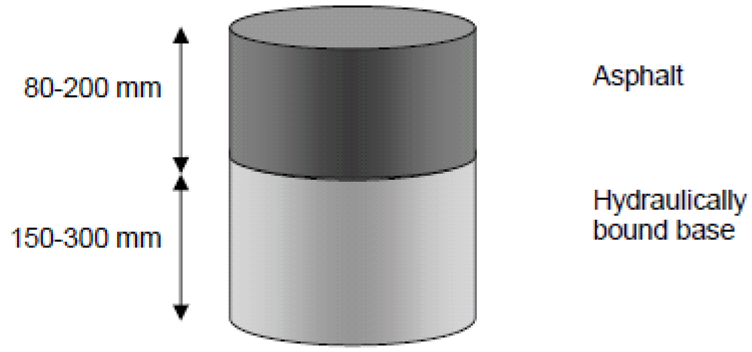
Semi-rigid long-life pavements are often referred to as flexible composite pavements. This type of pavement have been used around the world for heavy duty pavements. In Australia, it was the most common type of pavement and it was previously believed to get the best of both worlds; the stiffness of cement-bound materials and the flexibility of asphalt (Rickards & Armstrong 2010).

A literature review of semi-rigid long-life pavements in Europe (FEHRL 2009) showed that the technology for long-life semi-rigid pavements is not as developed as that for fully-flexible pavement types. It was also noted that while many European countries had established assessment and design methods for long-life fully-flexible pavement types, the equivalent approach for semi-rigid pavements did not exist, except in the UK.

The main mode of deterioration in hydraulically-bound base is a tensile failure when the stress in the layer exceeds the tensile strength of the material. Naturally occurring transverse thermal shrinkage cracks can form soon after construction. Hydraulically-bound base material that has high stiffness and a high coefficient of thermal expansion is more susceptible to formation of transverse cracking.

Over time, transverse cracks in the hydraulically-bound base are believed to propagate through the asphalt layers. Narrow width, closely-spaced cracks perform better than wide and less frequently spaced cracks. The above process of crack initiation and crack propagation is a complex process and continuing research in these areas is being conducted in Europe. A typical semi-rigid pavement structure in Europe is shown in Figure 2.5.

Figure 2.5: Typical structure of a European semi-rigid pavement for heavy traffic



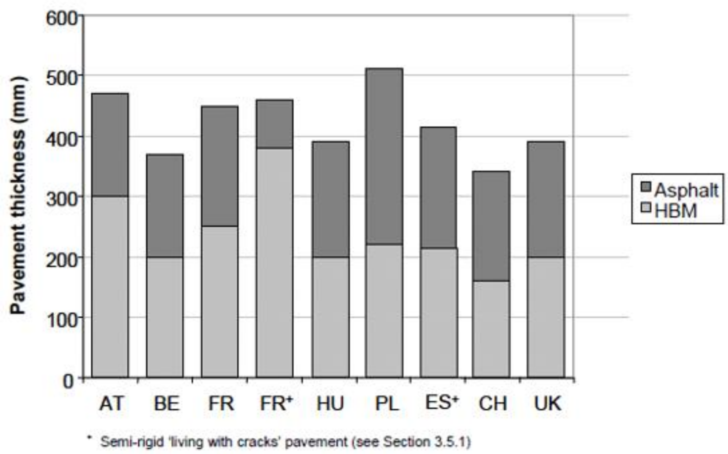
Source: FEHRL (2009).

Over the last two decades, much research has been carried out to improve the understanding of the behaviour of cemented material (Austroads 2010, Austroads 2014). It was recommended that the fatigue relationship for cemented material should be characterised using the concept of breaking strain.

Thickness Design for Semi-rigid Pavement

Research has highlighted that both strategies (i.e. prevention of cracks and propagation of crack can be accommodated in the long-life semi-rigid design concept provided that the appropriate provision is made in the future assessment and maintenance procedures (FEHRL 2009). Some typical thickness of semi-rigid pavements for heavy traffic for European roads are presented in Figure 2.6. The thickness of the hydraulically-bound base material (referred to as HBM) varies from 150 mm to 380 mm, and the thickness of the asphalt layer overlying the HBM layer varies from 80 mm to 300 mm. The wide range of asphalt thicknesses reflects the design strategy adopted, as highlighted in the discussion above. For heavily trafficked UK roads, approximately 200 mm of asphalt is required to delay the propagation of transverse cracks formed in the HBM layer.

Figure 2.6: Suggested semi-rigid designs for heavy traffic conditions (soil-cement subbase layer required under the hydraulically-bound base)



Source: FEHRL (2009).

2.3.3 Rigid (Concrete)

In Europe and the USA, concrete pavements are often used for heavy-duty highway applications. Studies conducted by ELLPAG and FHWA both indicated that concrete pavements can be a viable long-life pavement alternative (FEHRL 2009, FHWA 2007a, FHWA 2007b).

In Australia, NSW has the most experience with building concrete pavements. Vorobieff and Moss (2006) estimated that 95% of all concrete pavements constructed in Australia were located in NSW. Approximately 80% of all the highway concrete pavements constructed in NSW are jointed plain concrete pavement without dowels, and the remainder are either continuously reinforced, joined reinforced, or steel-fibre reinforced concrete. All heavy-duty concrete pavements are constructed over a lean mix concrete subbase. It has been highlighted that both aspects in design and construction are equally important to achieve long-life outcomes.

There are two SHRP long-term pavement performance (LTPP) heavy-duty plain concrete pavement sites in Australia that were tested in 2000 using the Accelerated Load Facility device (Vuong et al. 2000). However, the pavements were not loaded to failure to draw conclusive statement that they could be classified as long-life pavements.

Researchers in the USA (FHWA 2007b, Rohne 2009) reported typical design features used in long-life concrete pavements from different states. The key features are summarised Table 2.2.

Table 2.2: Typical design features for long-life concrete pavements in some states in the USA

State	Typical type of long-life concrete pavement type	Service design life	Pavement structure	Joint and reinforcement details
Minnesota	HPCP	60 years	Slab: 12–13 in. (305–330 mm) Base: 4 in. (102 mm) (dense graded granular) Subbase: min. 36 in. (914 mm) (select granular)	Transverse spacing: 15 ft (4.6 m) spacing Dowel bars: 1.5 in. (38 mm) dia., 15 in. (381 mm) long, spaced 12 in. (305 mm) apart
Illinois	CRCP	30–40 years	Slab: up to 14 in. (350 mm) Base: 4–6 in. (102–152 mm) (hot-mix asphalt stabilised) Subbase: 12 in. (300 mm) well graded aggregate Compacted subgrade	Reinforcement ratio: 0.8% tie-bar: 25 m dia., 750 mm long, spaced at 600 mm. (centre-line and lane-to-shoulder longitudinal joint)
Washington	PCC	50 years	Slab: 12 in. (305 mm) Base: 2.4–4.0 in. (60–100 mm) (HMA base)	Transverse spacing: 15 ft (4.6 m) Tie-bar: No.5 bars, 30 in. (750 mm) long, 36 in. (900 mm) spacing

Source: FHWA (2007b).

2.4 Field Evaluation of Long-life Pavement Performance

Pavement material characteristics are often determined using laboratory testing. In Australia, asphalt material characteristics are typically tested to determine its stiffness, rutting and fatigue performance. It has long been recognised that the fatigue life determined in the laboratory usually under-estimates the fatigue life observed in the field.

There are on-going research studies to explain the differences between the test results measured in the laboratory and in the field. This typically involves a long term pavement monitoring program or accelerated pavement testing (APT). The AAPA APSfL project used the field data from the National Centre for Asphalt Technology (NCAT) test track facility at Auburn University and selected LTPP sections in Australia to calibrate against the stiffness vs FEL relationship.

2.4.1 Australian LTPP Sections

Before the commencement of the APSfL project, AAPA has conducted a study to review the LTPP studies available in Australia (Rickards & Armstrong 2010). The work reviewed all 34 sites across Australia and summarised the findings from three phases of the AAPA LTPP study conducted between the period of 1998 and 2008 (ARRB Transport Research 2001, Foley 2008, Youdale 2004).

Two of the AAPA LTPP sites (Site Q5 and the site near Shailer Park) in Queensland were included as part of the AAPA APSfL calibration sites. Extensive information on pavement configuration, cumulative traffic, material testing and historical pavement condition data have been documented in three studies (ARRB Transport Research 2001, Foley 2008, Youdale 2004).

2.4.2 Accelerated Pavement Testing (APT) at National Centre for Asphalt Technology (NCAT)

As an alternative to the use of LTPP sites, APTs are often used to determine the field pavement performance within a reasonable timeframe. APTs include the one owned by ARRB, APT owned by the Federal Highway Administration (FHWA), as well as test tracks built in the USA at NCAT, Minnesota Department of Transportation (MnRoads) and WesTrack. These facilities often provide valuable field data to evaluate performance of long-life pavements.

A cumulative strain distribution study was conducted at the NCAT test track facility to develop a field-based “perpetual” pavement strain threshold (Willis 2009). Instead of a single FEL value, the study found that the fatigue life of the pavement is determined by the percentage of a range of asphalt strain. This leads to the concept of a cumulative strain distribution. The development of the cumulative strain distributions for cracked and non-cracked test track sections allows researchers to establish the strain distribution threshold for “perpetual” pavement behaviour. This also provides a field measurement comparison with the laboratory-established threshold strain limit. Generally, the laboratory threshold strain is less than the maximum field-based strain. The study also found that the laboratory threshold strain is less than the observed maximum in situ strain.

3 DESIGN METHODS – AAPA APSFL

For fully flexible long-life pavements, there are a number of design methods that are available. In this section, the AAPA APSfL design methodology will be presented and discussed.

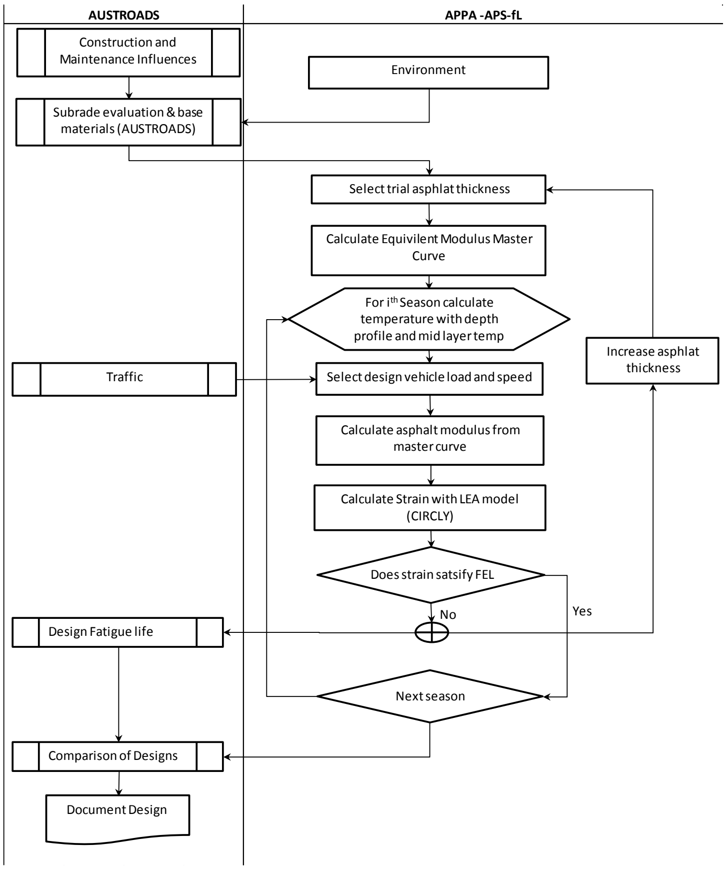
3.1 Introduction

In 2011, AAPA initiated a multi-year study to investigate and address concerns that the current Australian pavement design procedures may be producing overly conservative asphalt thickness. The findings to date have been documented in presentations and published at a number of conferences. The philosophy of the APSfL is to eliminate bottom-up fatigue cracking and periodic resurfacing work is only needed to repair cracking from oxidation and other surface initiated cracking within the surfacing layer. This section summarises the findings from the APSfL based on the following AAPA documents:

- AAPA presentation to steering group of the Austroads Pavement Task Force (AAPA 2014)
- AAPA draft design supplement to Austroads Guide to Pavement Technology Part 2 (AAPA 2015)
- Technical paper presented at an international conference (Sullivan & Nikraz 2014).

Based on the presentation given by AAPA to the Steering Group of the Pavement Task Force in February 2014 (AAPA 2014), a flow chart showing the proposed APSfL design procedure is shown in Figure 3.1.

Figure 3.1: Proposed AAPA design procedure



Source: AAPA (2014).

3.2 AAPA Draft Supplement to the Austroads Pavement Design Guide, for Long-life Asphalt Pavement Design

In March 2015, AAPA provided the project team with a copy of the draft supplement outlining the method to design highway and freeway pavements using the proposed LLAP design methodology (AAPA 2015). A copy of the draft design supplement (revision March 2015) is included in Appendix A of this report. Some important features of the draft supplement are presented in Table 3.1. At the time the supplement was provided, AAPA advised that it should be considered to be an unfinished draft.

Table 3.1: Design features in the AAPA draft supplement

Supplement section	Topic	Design features
1.0	Introduction	Supplement is for the design of pavement with an indeterminate structural life, for highway and freeway pavements.
3.2.5	Working platform	It is recommended for a subgrade with a stiffness of less than a 100 MPa, a working platform be established to achieve a stiffness of 100 MPa.
3.7	Pavement layering considerations	The LLAP utilises a three layer asphalt system: ▪ Upper layer is designed as a durable and rut resistant wearing course. ▪ Intermediate layer is the main structural layer in the pavement. ▪ Lowest asphalt layer (bottom 60 mm) is designed to be a low void (in place voids < 4%) mix fatigue resistant mix.
3.12	Maintenance strategy	LLAP are designed to eliminate fatigue cracking therefore no full/partial depth repairs are expected in the pavement life. All maintenance is expected to be top down resulting from oxidation and/or surface initiated cracking.
4.3	Temperature	Significant effect on the performance of LLAP should be taken into account in the design of LLAP. The temperature of the asphalt should be characterised by use of the Weighted Mean Annual Pavement Temperature (WMAPT) and the effective asphalt layer temperature at two extremes of temperature. The temperature at the midpoint of the combined asphalt layers can be calculated using the Australian modified Bells equation.
5.9	Subgrade failure criteria	To ensure adequate cover over the subgrade, a minimum number of allowable repetitions of 1×10^8 shall be applied, or 650 $\mu\varepsilon$, for analysis conducted at WMAPT.
6.4	Asphalt	Asphalt stiffness can be determined using the following test methods: ▪ Indirect tension on cylindrical samples (AS2891.13.1). ▪ Four-point bending on prismatic beams (AG:PT/T233). ▪ Direct compression on cylindrical samples (AASHTO TP62 or AASHTO TP79).
6.4.2	Poisson's ratio	The design procedure used a constant Poisson's ratio for asphalt of 0.35. This value should be used for all design calculations.
6.4.5	FEL	Proposed fatigue endurance limit: $FEL = 2800 Smix^{-0.34} - 100$ Where Smix = mix stiffness (MPa)
7	Design traffic	The cumulative number of axle loadings is immaterial, as the healing potential of the asphalt exceeds the damage done by vehicle loading. Furthermore, a critical vehicle used for LLAP should be taken as the upper 97.5th percentile axle load, this equates to a standard axle load of 9 tonne in Australia.

Source: AAPA (2015).

The design concept is that the healing potential of the asphalt exceeds the damage done by vehicle loading. Similar to other long-life pavement design methodology, it requires the construction of high quality asphalt layers over a strong foundation support. The goal is to eliminate fatigue cracking and the only maintenance work required is limited to the surfacing asphalt layer.

In the Australian context where the asphalt mix falls within a small mix volumetric window, the authors concluded that the FEL can be determined using a single mix stiffness parameter. Asphalt stiffness measured is dependent on the asphalt test method used, and the supplement provides a formula to convert between the three types of asphalt stiffness testing. Since the stiffness of asphalt is highly dependent on the seasonal temperature, the AAPA draft supplement used the WMAPT, summer and winter seasons to characterise the stiffness within a single equivalent asphalt layer. The design example given in the supplement used the modified JULEA, an isotropic multilayer linear elastic model, to compute the critical tensile strain at the bottom of the asphalt layer. The corresponding tensile strain at the bottom of the equivalent asphalt layer is then limited to the FEL proposed under each of the three different temperature seasons. The mechanistic design procedure provided by AAPA is summarised in Table 3.2.

Table 3.2: AAPA LLAP mechanistic design procedure

Step	Activity
1	Select a trial pavement
2	Determine subgrade stiffness
3	Determine working platform stiffness (if relevant)
4a	Determine WMAPT
5	Obtain MMAT for January and July maximum daily temperature for January and minimum daily temperature for July
6a	Determine surface temperature (To) for summer maximum and winter minimum
7	Determine effective asphalt layer temperature (Teff)
8	Determine elastic parameters of asphalt layers for summer, WMAPT and winter
9	Determine effective modulus of combined asphalt layers
10	Determine FEL for summer, WMAPT and winter
11a	Approximate the axle load as two circular vertical loads with a total load of 45 kN and centre-to-centre spacing of 330 mm and uniform vertical stress of 750 kPa
11b	Determine the critical locations as: 1. Bottom of the asphalt layers (summer, WMAPT and winter) 2. Top of the subgrade (WMAPT only)
11c	Input the values into the layered elastic analysis and determine the maximum tensile strain at the base of the asphalt layers and top of the subgrade
12	Compare calculated horizontal strain at bottom of the FEL for summer, WMAPT and winter to the FEL
13	Compare calculated compressive strain to the allowable strain at the WMAPT
14	If the calculated strain is less than the FEL than design is acceptable If not: 1. Select a new pavement configuration and return to step 1, or 2. Determine the life of the pavement as per AGPT-Part 2

Source: AAPA (2015).

At the time of this report, Austroads had engaged ARRB Group to evaluate possible issues if the APSfL method is implemented in the Austroads *Guide to Pavement Technology: Part 2* (2012). In June 2015, a presentation by the ARRB Group to the Austroads Pavement Task Force identified a number of issues, which are listed below:

- Validation was based on visual assessment of surface cracking, and may not have adequately determined whether cracking was surface only cracking or deeper structural cracking, nor did it conclusively identify if structural maintenance had occurred. Additionally, some sites had been resurfaced recently which raises some doubt over visual-only condition assessments.
- Some of the sites used for validation did not meet the minimum traffic criteria as per the AAPA definition in Table 2.1.

- Reliability of the AAPA method needs further consideration, and it was thought that the reliability levels that could be assigned based on the AAPA validation are much less than what is typically adopted for road pavements. Due to the very limited number of sites (six) used for the validation, at the reliability levels typically used for pavement design it would be expected that most sites would not be structurally cracked.
- A number of potentially unnecessary differences between the AAPA methodology and current Austroads AGPT02, such as:
    - the different asphalt characterisation method
    - the different subgrade characterisation (back calculation of in situ value vs. soaked CBR).
    - the different characterisation of unbound materials (isotropic model used in the development of the FEL relationship.
    - the design spreadsheet uses a different relationship for sub–layering.
    - the procedure used half of a single axle dual tyre (SADT) to determine FEL strain.

During the presentation, ARRB Group highlighted that there are limited Australian calibration sites (six) and only two sites were located in Queensland.

While a number of issues were raised regarding the draft AAPA methodology, it was reaffirmed that Austroads recognises the synergies between the AAPA project and Austroads' desired to incorporate fatigue endurance limits into AGPT02. An outcome of the meeting was a commitment from AAPA to address the concerns raised and provided an updated design supplement, which would subsequently be reviewed by ARRB Group on behalf of Austroads.

Further details of the Queensland sites can be found in a number of Australian AAPA LTTP studies (ARRB Transport Research 2001, Foley 2008, Youdale 2004), which are summarised in Appendix B of this report.

4 EVALUATION OF THE IMPLEMENTATION OF THE APSFL METHODOLOGY FOR QLD

4.1 Factors Influencing the Pavement Design Thicknesses

In order to assess and compare the difference in design thickness reported using the APSfL design spreadsheet and compare with the solution derived using the current Austroads/TMR methodology, as detailed in AGPT02 (Austroads 2012) and TMR supplement (Department of Transport and Main Roads 2013), the following key design parameters have been investigated across Queensland:

- heavy vehicle (HV) design speeds
- pavement temperature
- design subgrade strength.

4.1.1 HV Design Speeds

The material behaviour of asphalt is dependent upon the speed of the HV load. However, it is noted that the AAPA APSfL design supplement (AAPA 2015) is only applicable for freeway and highway pavements. In this study, only the asphalt modulus corresponding to a HV speed of 80 km/h is considered.

It is noted that the AAPA APSfL design spreadsheet allows speeds lower than 80 km/h to be computed (but it is not recommended to have a speed lower than 40 km/h which approaches the creep speed of a HV).

4.1.2 Climatic Conditions from the Bureau of Meteorology (BOM)

The current Austroads and TMR pavement design methodology relies upon the use of the WMAPT when characterising the material behaviour of asphalt pavements. While this approach has its limitations, this represents the current state of design practice in Queensland. Other NACOE research projects are underway to evaluate and improve on this issue.

On the other hand, the APSfL design method requires other climatic data beyond the WMAPT values to be used as inputs to the design spreadsheet. This information can be obtained from the Bureau of Meteorology (BOM) website and the relevant climatic data for selected towns across Queensland is summarised in Table 4.1. The WMAPT across major towns in Queensland ranges between 27 °C and 37 °C. The table also contains other weather station data required for the APSfL design method, such as temperature data collected during summer and winter months.

Table 4.1: Weather station information for selected towns across Queensland

Site	Location	WMAPT (°C)	Station No.	Latitude (°)	Summer Max. (°C)	Summer Ave Min. (°C)	Winter Max. (°C)	Winter Ave Min. (°C)
QL01	Beaudesert	31	40014	28.02	31.0	19.2	21.2	5.1
QL02	Brisbane	32	40223	27.42	29.1	20.9	20.6	9.5
QL03	Bundaberg	33	39128	24.91	30.2	21.5	22.1	10.2
QL04	Cairns	37	31011	16.87	31.5	23.8	25.7	17.1
QL05	Charters Towers	36	34084	20.05	34.5	22.4	24.8	11.4
QL06	Emerald	35	35264	23.57	34.4	22.2	23.2	8.9
QL07	Gladstone	34	39326	23.87	30.7	23.0	22.9	11.8
QL08	Goodiwindi	32	41521	28.52	34.0	20.3	19.2	4.6
QL09	Ipswich	32	40101	27.61	32.0	20.0	21.1	7.0
QL10	Mackay	34	33045	21.17	30.7	23.2	22.5	11.2
QL11	Rockhampton	35	39083	23.38	32.1	22.1	23.1	9.6
QL12	Roma	33	43091	26.55	34.2	20.9	20.2	3.7
QL13	Toowoomba	27	41103	27.58	27.6	16.7	16.3	5.3
QL14	Townsville	37	32040	19.25	31.5	24.3	25.1	13.7

Source: Bureau of Meteorology (2015).

4.1.3 Design Subgrade

The strength of the design subgrade has a significant influence on the full-depth asphalt (FDA) design thickness of any design solutions. One of the premises of long-life pavement is that it is well-supported by an underlying subgrade foundation. In fact, the APSfL design supplement indicated that a minimum of 100 MPa is required at the top of the layer immediately below the asphalt layers. For subgrade with lower than 100 MPa (or CBR 10%), some granular subbase or working platform (improved layer) layer is required. For this reason, the design examples used only adopt design subgrade cases of CBR 5% and CBR 10%, with an improved layer above the subgrade.

4.2 Characterisation of Asphalt Elastic Modulus

4.2.1 APSfL Design Solutions

The AAPA design method is based on a large number of dynamic modulus compression tests performed on cylindrical samples and the laboratory results form the basis of the asphalt master curves recommendations. For the purpose of this comparison study, typical asphalt mixes used in Queensland have been included, namely:

- wearing course – 14 mm sized dense graded asphalt with Class 320 binder, DG14(C320)
- intermediate course – 14 mm sized dense graded asphalt with Class C320 binder, DG14(C320)
- base course – 20 mm sized dense graded asphalt with Class C600 binder, DG20(C600).

It should be noted that it is common practice to adopt polymer modified binder in the wearing and intermediate courses for very heavily trafficked roads in Queensland. However, to simplify the comparison study in this report it was assumed that these layers contain C320 bitumen for all traffic levels.

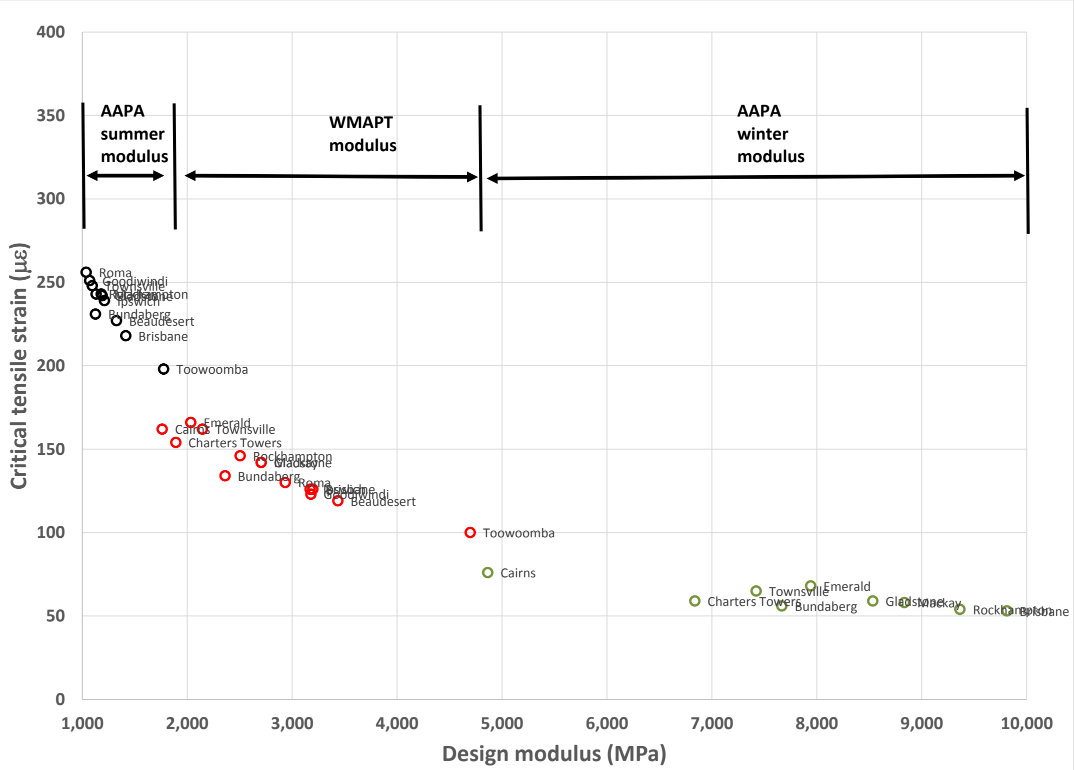
For heavy duty pavements in Queensland, it is common practice to place an improved layer (minimum thickness of 150 mm) underneath the asphalt layers. This layer has been incorporated into the CIRCLY model used.

The thickness of the wearing and intermediate courses are both held constant at 50 mm, and the design thickness for the asphalt base course varies until the design solution meets the FEL value calculated from the design spreadsheet.

Presumptive master curve fitting parameters were provided in the design supplement for typical Australian asphalt mixes and they are used in the design spreadsheet to compute the design modulus at three different temperature environments (summer, WMAPT and winter), which will be used to compute critical tensile strain at the bottom of asphalt layers.

Using the AAPA design spreadsheet, critical tensile strains for various Queensland towns are presented in Figure 4.1. As discussed above, the design modulus changes across the three temperature environments. As expected, the hotter summer month will have the lowest asphalt stiffness (or modulus) and highest critical tensile strain computed at the bottom of the asphalt layer, vice versa, the asphalt has the highest modulus during the winter months.

Figure 4.1: Critical tensile strains vs design modulus determined using the AAPA APSfL method for different temperature environments



The APSfL design supplement advocates a low-void and high bitumen content layer as the lowest asphalt layer. The intention of this layer (typical thickness is 60 mm) is to provide a high fatigue resistant layer to reduce the risk of crack formation from the bottom. However, this design feature has not been used in this study because this practice does not align with current practice in Queensland.

4.2.2 Current Austroads/TMR Pavement Design Methodology

Design thicknesses developed using the current Austroads/TMR design method apply temperature correction factors to presumptive asphalt mixture moduli. These temperature adjusted values are then used as input in CIRCLY to compute the pavement design thickness. Presumptive moduli and temperature adjusted elastic moduli for different towns in Queensland are presented in Table 4.2.

Table 4.2: Presumptive and temperature adjusted elastic moduli for DG14 (C320) and DG20HM (C600) asphalt mixes

Location	WMAPT (°C)	Temp. correction factor	DG14 (C320) asphalt mixture volume of binder = 11% (HV speed of 80 km/h)		DG20HM (C600) asphalt mixture volume of binder = 10.5% (HV speed of 80 km/h)	
			Presumptive values at WMAPT of 32 °C (MPa)	Temperature adjusted moduli values for WMAPT of selected town (MPa)	Presumptive values at WMAPT of 32 °C (MPa)	Temperature adjusted moduli values for WMAPT of selected town (MPa)
Beaudesert	31	1.083	2400	2600	3100	3350
Brisbane	32	1.000		2400		3100
Bundaberg	33	0.923		2200		2850
Cairns	37	0.670		1600		2100
Charters Towers	36	0.726		1750		2250
Emerald	35	0.787		1900		2450
Gladstone	34	0.852		2050		2650
Goodiwindi	32	1.000		2400		3100
Ipswich	32	1.000		2400		3100
Mackay	34	0.852		2050		2650
Rockhampton	35	0.787		1900		2450
Roma	33	0.923		2200		2850
Toowoomba	27	1.492		3600		4650
Townsville	37	0.670		1600		2100

4.3 Comparison of APSfL and Current Austroads/TMR Pavement Design Methodology

4.3.1 Design Thickness Comparison

A range of total structural asphalt thicknesses (i.e. total thickness of the surfacing, binder and base asphalt layers) were computed are presented in Table 4.3. The design thicknesses for the design subgrade of CBR 5% and CBR 10% are plotted in Figure 4.2 and Figure 4.3, respectively. The two design methods used are listed as follows:

- AAPA APSfL design method (AAPA 2015) (implemented using a design spreadsheet)
- Austroads/TMR method (computed using the Mincad CIRCLY computer program).

Table 4.3: Total structural asphalt thickness (mm) using the APSfL design method and the current TMR design method for subgrade CBR 5% and 10%

Location	WMAPT (°C)	Design subgrade CBR 5%				Design subgrade CBR 10%			
		AAPA (2015)	TMR (2013) 1.00E+ 07 ESA	TMR (2013) 8.00E+ 07 ESA	TMR (2013) 1.00E+ 08 ESA	AAPA (2015)	TMR (2013) 1.00E+ 07 ESA	TMR (2013) 8.00E+ 07 ESA	TMR (2013) 1.00E+ 08 ESA
Toowoomba	27	260	225	300	310	215	95	165	175
Beaudesert	31	270	245	325	335	210	115	190	200
Brisbane	32	270	250	330	340	210	115	195	205
Goodiwindi		275				215			
Ipswich		270				210			
Bundaberg	33	270	255	340	N/A	210	120	200	N/A
Roma		275				215			
Gladstone	34	270	260	345	N/A	215	120	200	N/A
Mackay		270				210			
Emerald	35	275	265	350	N/A	210	130	210	N/A
Rockhampton		275				210			
Charters Towers	36	275	270	355	N/A	215	135	215	N/A
Cairns	37	275	275	365	N/A	215	140	220	N/A
Townsville		275				205			

Note: For locations where design traffic of 1×10^8 ESA is unlikely, the design thicknesses in accordance to TMR (2013) were not computed.

The AAPA method is for the design of long-life pavement which is intended to only have maintenance works carried out in the surfacing layer. Therefore, the design thickness reported is not associated with a particular allowable loading (or design ESAs). For the subgrade CBR 5% case, total asphalt thicknesses were reported to be between 260 and 275 mm across Queensland. For the subgrade CBR 10% case, total asphalt thicknesses range between 205 and 215 mm. When compared with the current Austroads/TMR pavement design method, the design thickness proposed by the AAPA method is generally smaller. In particular, as the design subgrade decreases and traffic loading increases beyond 1E+08 ESA, the differences in thickness between the two methods became more substantial.

The current draft supplement does not include a process to directly consider design reliability. It is presumed that the procedure reflects a reliability level of 50 %. The draft method needs to include a process allowing designs to be conducted to different desired project reliability levels.

When comparing the design thicknesses from the AAPA method to the international long-life pavements discussed in Section 2, it appears that the AAPA thicknesses are less than the thicknesses as reported in Figure 2.3 and Figure 2.4 for pavements with design life of 30 years or more.

Figure 4.2: AAPA APSfL and Austroads/TMR design thickness comparison with a design subgrade CBR 5%

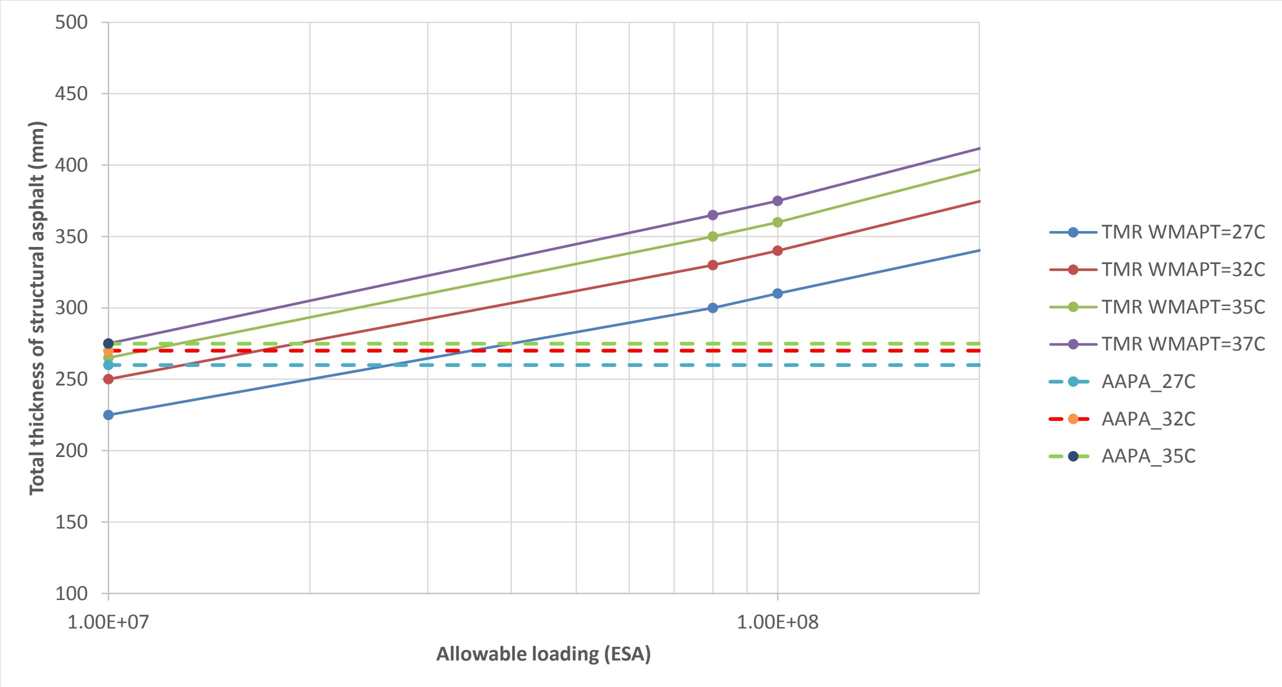
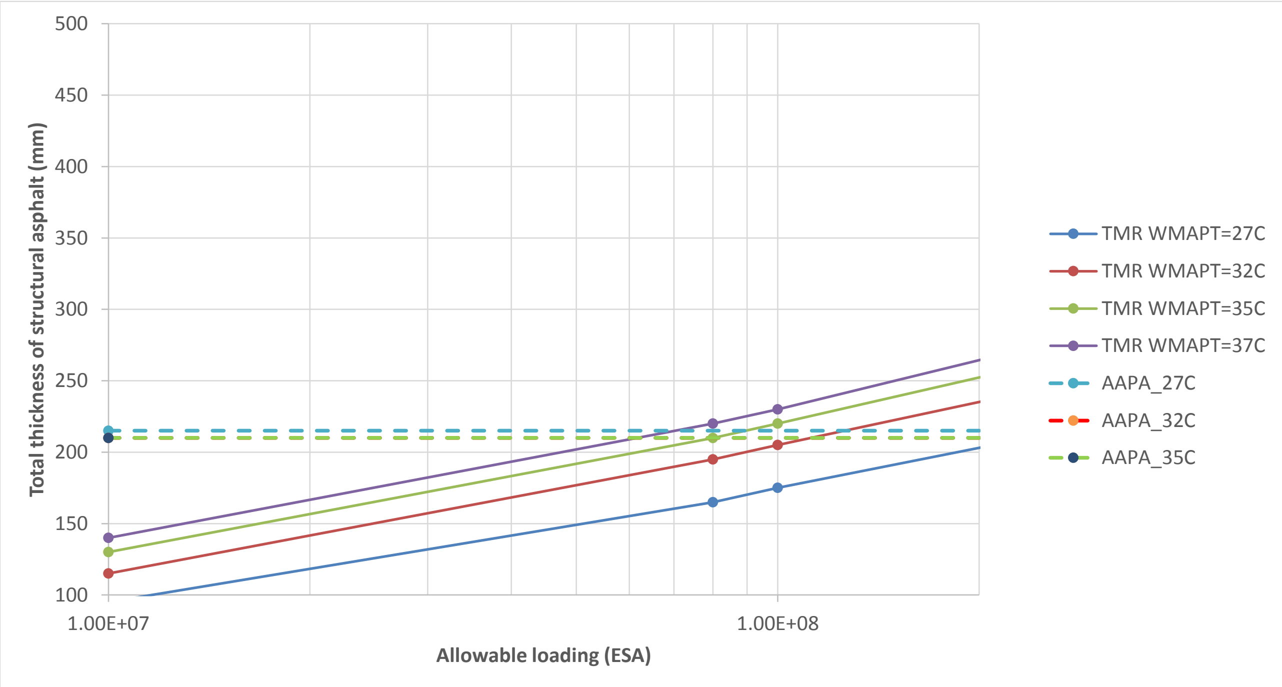


Figure 4.3: AAPA APSfL and Austroads/TMR design thickness comparison with a design subgrade CBR 10%



4.3.2 Comparison of the Effect of Different Standard Applied Loads

The differences in design thicknesses were presented in the previous section using the current Austroads/TMR pavement design method (Department of Transport and Main Roads 2013) and AAPA (2015) design methods. Both methods design the thickness of the asphalt structure so that the critical tensile strain at the lowest asphalt layer is within the acceptable level corresponding to the design traffic loading. For the AAPA (2015) method, this is to limit the equivalent asphalt layer below the proposed FEL and by definition, the FEL is independent of the design traffic. For the current Austroads/TMR pavement design method, the critical tensile strain was determined using the AGPT02-12 (Austroads 2012) which was based on the Shell asphalt fatigue equation.

Both design methods differed in terms of the stress-strain model and the configuration of the standard load. The AAPA model was based on an isotropic model for the improved layer and subgrade, while the Austroads/TMR method uses the anisotropic model when characterising the improved layer and the subgrade. Furthermore, the AAPA model used a 44 kN (4.5 tonne) half-axle load and the TMR model typically used a full-axle load of 80 kN. An investigation of the critical tensile strain for a 200 mm thick asphalt pavement is presented in Table 4.4.

Table 4.4: Critical tensile strains (microns) at the bottom of a 200 mm thick asphalt pavement (E = 2000 MPa)

CIRCLY model	Maximum tensile strain near applied wheel load (XX and YY planes)
Case A AAPA model (isotropic) 44 kN half axle load (AAPA)	ε_{YY}(max) = 260 $\mu\varepsilon$
Case B Austroads/TMR model (anisotropic) 44 kN half axle load (AAPA)	ε_{YY}(max) = 280 $\mu\varepsilon$
Case C Austroads/TMR model (anisotropic) 80 kN full axle load (Austroads/TMR)	ε_{YY}(max) = 257 $\mu\varepsilon$

The critical tensile strains in different pavement model and design axle loading configurations were determined using CIRCLY. The tensile strain profiles in the XX and YY planes for the three different models are presented above for comparison. The critical value is the maximum of the values reported in both planes.

As shown in Table 4.4 the critical tensile strain for the AAPA model (i.e. Case A) was reported as 260 $\mu\varepsilon$, and is similar in value with the TMR 80 kN full-axle load of 257 $\mu\varepsilon$ (i.e. Case C). It is also worth noting that when applying a 44 kN half axle load on a Austroads/TMR anisotropic model, as illustrated in Case B, the critical tensile strain is 280 $\mu\varepsilon$. For the above pavement structure, the analysis indicated that the difference in tensile strain calculated using the two methods is minimal. Therefore, it is believed that the reduction in pavement thickness is primarily contributed from the selection of failure strain criteria, and this will be discussed further in Section 4.3.3.

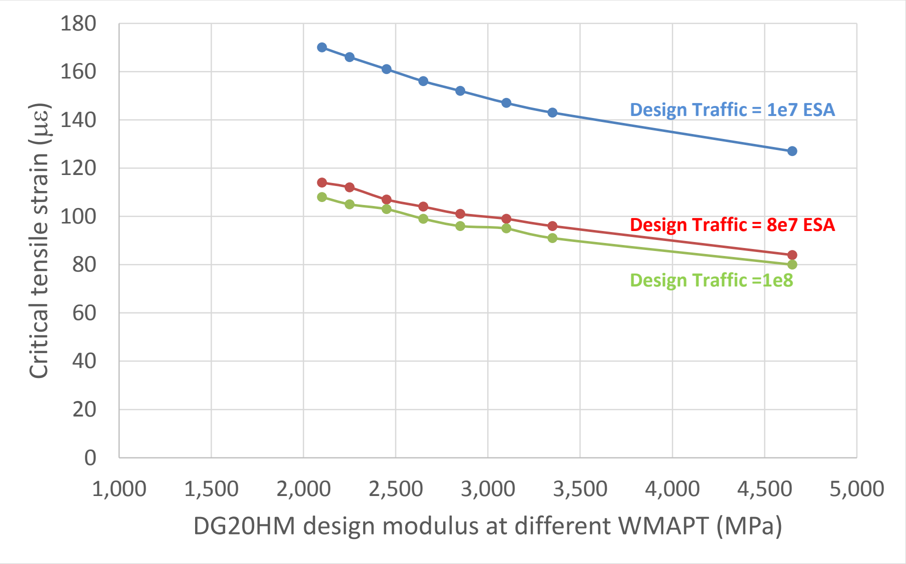
4.3.3 Comparison of the Critical Strains at the Bottom of the Asphalt Layer

In this section, the critical strain computed using the current Austroads/TMR pavement design method and AAPA (2015) methods are compared. Using the current TMR pavement design method, for a range of design traffic between 1.0E+07 and 5.0E+08 ESA, the critical tensile strain at the bottom of the DG20HM asphalt base layer is presented in Table 4.5. The value for the asphalt design modulus is influenced by the heavy vehicle speed and the WMAPT. The critical strain is plotted against the design modulus in Figure 4.4. As expected, the critical strain decreases as the design modulus and the design traffic increase.

Table 4.5: Critical tensile strain at the bottom of the lowest asphalt layer for a range of design traffic

		Critical tensile strain based on the TMR (2013) method (micro-strains)		
WMAPT (°C)	DG20HM design modulus (MPa)	Design traffic 1.00E+07 ESA	Design traffic 8.00E+07 ESA	Design traffic 1.00E+08 ESA
27	4650	127	84	80
31	3350	143	96	91
32	3100	147	99	95
33	2850	152	101	96
34	2650	156	104	99
35	2450	161	107	103
36	2250	166	112	105
37	2100	170	114	108

Figure 4.4: Asphalt critical tensile strain computed using the current Austroads/TMR pavement design method at different design traffic levels

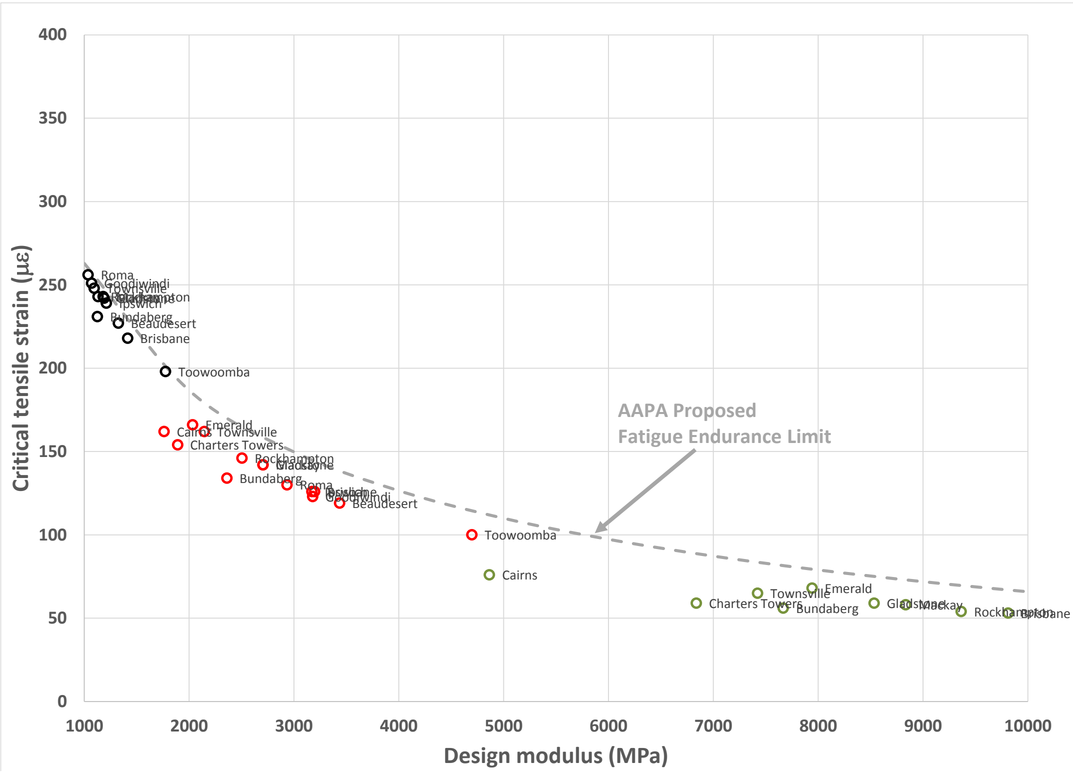


Based on the AAPA (2015) method, critical strains were computed for the AAPA long-life pavements and are presented in Table 4.6. The design method considers three temperature environments (i.e. summer, WMAPT and winter) and the objective is to have a long-life pavement with adequate asphalt thickness where all critical strains fall below the corresponding FEL proposed. The same dataset is plotted in Figure 4.5 together with the AAPA proposed FEL. As expected, all reported critical strains fall below the FEL and even for Toowoomba (WMAPT = 27 °C) the FEL is around 120 $\mu\varepsilon$.

Table 4.6: AAPA asphalt modulus and critical strains computed using the AAPA APSfL design spreadsheet

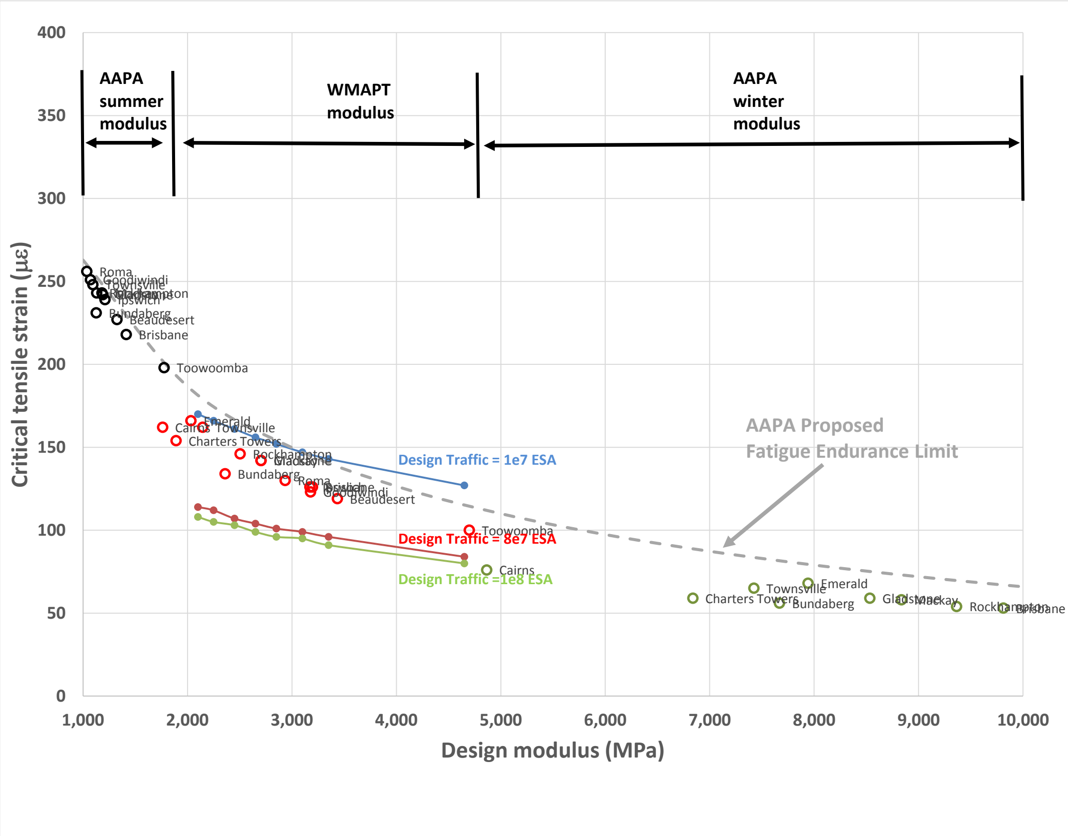
		AAPA modulus (MPa)			Critical strain of asphalt (με)			Fatigue endurance limit (με)		
Location	WMAPT	Summer	WMAPT	Winter	Summer	WMAPT	Winter	Summer	WMAPT	Winter
Beaudesert	31	1325	3436	11 365	227	119	47	230	139	59
Brisbane	32	1414	3197	9812	218	126	53	223	144	67
Bundaberg	33	1126	2361	7668	231	134	56	233	152	70
Cairns	37	997	1762	4865	245	162	76	250	180	97
Charters Towers	36	892	1891	6839	265	154	59	266	173	76
Emerald	35	899	2034	7942	265	166	68	263	146	53
Gladstone	34	1189	2705	8534	242	142	59	242	159	75
Goodiwindi	32	1071	3179	11 846	251	123	44	254	145	57
Ipswich	32	1211	3172	10 681	239	126	50	240	145	62
Mackay	34	1181	2705	8836	243	142	58	243	159	73
Rockhampton	35	1131	2505	9365	243	146	54	248	166	70
Roma	33	1037	2935	12 015	256	130	44	258	152	56
Toowoomba	27	1776	4698	11 965	198	100	48	199	114	56
Townsville	37	1095	2146	7423	248	162	65	252	180	84

Figure 4.5: Critical strains computed using the AAPA (2015) method for different temperature regimes



A better comparison of the critical strains from the two design methods is presented in Figure 4.6 (which is a combination of the information presented in Figure 4.4 and Figure 4.5). It is noted that the FEL proposed by AAPA over the modulus range between 2000 and 5000 MPa, is similar to the critical strain line for a design traffic of 1.0E+07 ESA, and this is considerably lower than the 8.0E+07 ESA nominated by the AAPA (2015) criteria for long-life pavements. It is also noted that the use of WMAPT alone appears to dictate the behaviour of the asphalt pavement over the modulus range of 2000 and 5000 MPa.

Figure 4.6: Comparison of critical strains from Austroads/TMR (2013), AAPA (2015) and the AAPA FEL



5 REVIEW OF EXISTING PAVEMENT SECTIONS IN QUEENSLAND

One of the concerns identified by the Austroads Pavements Task Force of the proposed AAPA APSfL design method is that the FEL was established based on a limited number of Australian calibration sites. The design method relies on the information from the Roads and Maritime Services (RMS) Structural Testing Evaluation of Pavement (STEP) database and only two calibration sites were located in Queensland, namely:

- Pacific Highway near Shailer Park (southbound)
- Bruce Highway near Kallangur, located 2.2 to 2.4 km south of Boundary Road (Q6 site).

While the AAPA project has gone to some effort to identify appropriate sites, the practical reality is that there appears to be very few suitable sites in Australia that can be used for calibration.

One objective of the project is to identify existing pavement sections in Queensland which are potential candidates for Queensland calibration sites. Pavement sites which meet the following list of attributes can qualify as LLAP sites:

- full and partial depth asphalt
- no cemented material as base or subbase
- at least 30 years old
- cumulative traffic of at least 8 x 10^7 ESA
- no cracking
- assessed (by visual inspection) as having over 20 years remaining structural life.

5.1 ARMIS Database

The ARMIS database was used as a screening tool to identify potential sites in Queensland. AAPA APSfL design method only covers full depth asphalt pavements which do not contain a cement-bound layer. These types of pavements are typically found along highways in South East Queensland. In order to limit the number of searches, the following major highways were included in the analysis in Year 1:

- Bruce Highway (10A)
- Pacific Motorway (12A)
- Cunningham Highway (17B)
- Warrego Highway (18A)
- Mount Lindesay Highway (25A)
- Pacific Motorway (U12A)
- Gateway Motorway (U13C)
- Western Arterial Road (U18A).

Based on the ARMIS database, road sections that may be potential LLAP candidate sites are presented in Table 5.1. The information is then compared with available traffic information and construction drawings to further narrow down the search.

Sites 3 and 4 are eliminated because the asphalt base is only 13 years old. Sites 6, 7 and 9 are eliminated because the Average Annual Daily Traffic (AADT) was less than 5000 vehicles per day in year 2014–15, and therefore the cumulative traffic is not likely to meet the LLAP requirement. Site 8 is located at the top of the Toowoomba Range and because of the terrain and proximity to a major intersection, this site is unlikely to meet the freeway environment condition expected for the AAPA APSfL design methodology.

After the elimination, only Sites 1, 2 and 5 remain as candidates. It is noted that Sites 1, 2 and 5 were previously identified by AAPA as the LLAP calibration sites in Queensland. Sites 1 and 2 are near the location of the Q5 and Q6 site, and Site 5 has also been studied.

Therefore, so far, no additional sites have been identified.

Table 5.1: Potential LLAP candidate sites in Queensland based on ARMIS data

Site no.	1	2	3	4	5	6	7	8	9
Road	10A	10A	10A	10A	12A	17B		18A	U13C
Lane Direction	Gazattel	Anti-Gazattel	Anti-Gazattel	Gazattel	Gazattel	Gazattel	Anti-Gazattel	Gazattel	Gazattel
Approx. location	Bruce Hwy – North of Anzac Ave		North of Pine River		Shailer Park	South of Ipswich – Rosewood Rd intersection		Top of Toowoomba Range	South of Bicentennial Rd., Boondall
Lane no.	1 and 3	2 and 4	2	3	1	1	2	3	3
Chainage	Ch. 5.3–6.3 km	Ch. 5.2–6.3 km	Ch. 0.1–0.28 km	Ch. 0.1–1.21 km	Ch. 8.5–8.66 km	Ch.19.9–20.1 km	Ch.19.9–20.1 km	Ch. 91.9–92.2 km	Ch. 10.4–10.6 km
% HV	11.4(1)	11.4(1)	11.4(1)	11.4(1)	8.3	24.7	25.8	17.5	12.0
Traffic year	2014	2014	2014	2014	2014	2015	2015	2014	2014
1-Way AADT	53 046 vpd	52 124 vpd	70 815 vpd	70 625 vpd	53 550 vpd	2851 vpd	2824 vpd	10 902 vpd	4797 vpd
Year of asphalt base construction	1989	1989	2002	2002	1997	2001	2001	1993	1989
Asphalt base thickness (mm)	325	325	310	340	285	295	295	270	225
Year of recent resurfacing	2004	2004	–	–	–	–	–	2008	2002
AC resurfacing thickness (mm)	85	85	–	–	–	–	–	30	65
No. of years since base construction	26 years	26 years	13 years	13 years	18 years	18 years	18 years	22 years	26 years

1 This percentage of HV data was not available within the section, the value was obtained from Ch. 28.94–34.86 km (site 20221).

5.2 Available Performance Data

In this section, only the condition data from Sites 1, 2 and 5 are presented. At this time, only network-level performance condition data is available to the project team, and this information is presented in Table 5.2, Table 5.3 and Table 5.4. Project-level condition data and detailed visual cracking survey are not available. If required, field investigation or a detailed visual condition survey may need to be conducted in future years. This information should complement the information available from the three sites.

Table 5.2: Current pavement performance data for Bruce Highway, 10A, Lane 1, Ch. 5.1–6.3 km (site 1)

<table>
<tr><th>Chainage (km)</th><th>Lane code</th><th>Austroads roughness count</th><th>International roughness index</th><th>Rutting maximum (mm)</th><th>Rutting 1.2m IWP average</th><th>Rutting 1.2m IWP standard deviation</th><th>Cracking extent crocodile</th><th>Cracking extent longitudinal</th><th>Cracking count transverse</th><th>Pothole %</th><th>Patching severity index</th><th>Patching %</th></tr>
<tr><td>5.1–5.2</td><td>1</td><td>49</td><td>1.74</td><td>5</td><td>2.9</td><td>0.7</td><td rowspan="6">0</td><td rowspan="6">0</td><td rowspan="6">0</td><td rowspan="12">0</td><td rowspan="12">0</td><td rowspan="12">0</td></tr>
<tr><td>5.2–5.3</td><td>1</td><td>35</td><td>1.13</td><td>4</td><td>2.8</td><td>0.4</td></tr>
<tr><td>5.3–5.4</td><td>1</td><td>38</td><td>1.36</td><td>6</td><td>3.7</td><td>0.7</td></tr>
<tr><td>5.4–5.5</td><td>1</td><td>36</td><td>1.16</td><td>7</td><td>3.5</td><td>1.3</td></tr>
<tr><td>5.5–5.6</td><td>1</td><td>32</td><td>1.08</td><td>7</td><td>4.2</td><td>0.9</td></tr>
<tr><td>5.6–5.7</td><td>1</td><td>32</td><td>1.11</td><td>6</td><td>3.4</td><td>0.7</td></tr>
<tr><td>5.7–5.8</td><td>1</td><td>32</td><td>1.13</td><td>6</td><td>3.9</td><td>0.8</td><td>0</td><td>2</td><td>8</td></tr>
<tr><td>5.8–5.9</td><td>1</td><td>26</td><td>0.86</td><td>6</td><td>3.5</td><td>0.7</td><td>0</td><td>0</td><td>2</td></tr>
<tr><td>5.9–6.0</td><td>1</td><td>32</td><td>1.1</td><td>5</td><td>3.6</td><td>0.6</td><td rowspan="4">0</td><td rowspan="4">0</td><td rowspan="4">0</td></tr>
<tr><td>6.0–6.1</td><td>1</td><td>36</td><td>1.21</td><td>8</td><td>3.7</td><td>1.4</td></tr>
<tr><td>6.1–6.2</td><td>1</td><td>32</td><td>1.06</td><td>4</td><td>2.6</td><td>0.4</td></tr>
<tr><td>6.2–6.3</td><td>1</td><td>29</td><td>0.97</td><td>5</td><td>2.9</td><td>0.7</td></tr>
</table>

Table 5.3: Current pavement performance data for Bruce Highway, 10A, Lane 2, Ch. 5.1–6.3 km (site 2)

<table>
<tr><th>Chainage (km)</th><th>Lane code</th><th>Austroads roughness count</th><th>International roughness index</th><th>Rutting maximum (mm)</th><th>Rutting 1.2m IWP average</th><th>Rutting 1.2m IWP standard deviation</th><th>Cracking extent crocodile</th><th>Cracking extent longitudinal</th><th>Cracking count transverse</th><th>Pothole %</th><th>Patching severity index</th><th>Patching %</th></tr>
<tr><td>5.1–5.2</td><td>2</td><td>40</td><td>1.33</td><td>6</td><td>3.1</td><td>1.1</td><td rowspan="12">0</td><td rowspan="12">0</td><td rowspan="12">0</td><td rowspan="12">0</td><td rowspan="12">0</td><td rowspan="12">0</td></tr>
<tr><td>5.2–5.3</td><td>2</td><td>31</td><td>1.05</td><td>7</td><td>3.8</td><td>0.9</td></tr>
<tr><td>5.3–5.4</td><td>2</td><td>28</td><td>0.82</td><td>7</td><td>3.6</td><td>1.0</td></tr>
<tr><td>5.4–5.5</td><td>2</td><td>30</td><td>1.0</td><td>6</td><td>3.5</td><td>0.7</td></tr>
<tr><td>5.5–5.6</td><td>2</td><td>33</td><td>1.01</td><td>6</td><td>2.7</td><td>1.2</td></tr>
<tr><td>5.6–5.7</td><td>2</td><td>41</td><td>1.37</td><td>9</td><td>4.9</td><td>1.2</td></tr>
<tr><td>5.7–5.8</td><td>2</td><td>40</td><td>1.3</td><td>8</td><td>4.8</td><td>1.2</td></tr>
<tr><td>5.8–5.9</td><td>2</td><td>32</td><td>1.05</td><td>7</td><td>4.2</td><td>0.9</td></tr>
<tr><td>5.9–6.0</td><td>2</td><td>31</td><td>0.99</td><td>7</td><td>4.3</td><td>0.8</td></tr>
<tr><td>6.0–6.1</td><td>2</td><td>33</td><td>1.03</td><td>10</td><td>5.4</td><td>1.4</td></tr>
<tr><td>6.1–6.2</td><td>2</td><td>40</td><td>1.38</td><td>7</td><td>4.4</td><td>0.7</td></tr>
<tr><td>6.2–6.3</td><td>2</td><td>27</td><td>0.9</td><td>7</td><td>4.1</td><td>0.9</td></tr>
</table>

Table 5.4: Current pavement performance data for Pacific Motorway, 12A, Lane 1, Ch. 8.5–8.7 km (site 5)

Chainage (km)	Lane code	Austroads roughness count	International roughness index	Rutting maximum (mm)	Rutting 1.2m IWP average	Rutting 1.2m IWP standard deviation	Cracking extent crocodile	Cracking extent longitudinal	Cracking count transverse	Pothole %	Patching severity index	Patching %
8.5–8.6	1	34	1.09	12	4.1	2.5	0	61	5	0	0	0
8.6–8.7	1	49	1.57	10	3.2	2.1	0	40	48			

Based on the above network-level data, Sites 1 and 2 located along Bruce Highway appear to be in good condition in terms of rutting and roughness. For the majority of the site, no cracking was reported, except between Ch. 5.7–5.9 km, where transverse and longitudinal cracking was noted. As no visual surface imagery is available to the project team at this time, nor any details on depth of cracking (that is, whether it is surface only cracking or deeper cracking), it is difficult to verify and identify the nature of the cracking.

Site 5 near Shailer Park along the Pacific Highway has moderate rutting (maximum 10–12 mm) and significant transverse and longitudinal cracking. The level of cracking reported is consistent with the recent surface image scan (Radar Portal Systems 2015). Again, the depth of cracking is unknown.

While two out of the three potential Queensland calibration sites are showing evidence of cracking, it is not possible to conclude from the available information whether these are surface cracks or deeper structural cracks. However, it is noted that the asphalt thicknesses from the proposed AAPA methodology would be less than the actual thickness of the two cracked pavements. Without further knowledge on the nature of the cracking, this raises uncertainty on the outcomes of the current AAPA methodology. As has been previously noted, this may be a result of the AAPA methodology requiring further adjustment to provide outcomes that more closely align with the reliability levels typically adopted for road pavement design.

Further investigation of the potential Queensland calibration sites needs careful consideration. Even if the nature of the cracking can be identified, due to the very limited number of sites it would likely be very difficult to draw meaningful conclusions from the results to the extent necessary to properly calibrate a long life pavement design methodology.

6 SUMMARY AND PROPOSED WORK FOR YEAR 2

6.1 Summary

This report presents the work that has been conducted in Year 1 of the NACOE P39 project. Key tasks completed are listed as follows:

- Task 1 – refine project scope
- Task 2 – review literature
- Task 3 – identify existing sections in Queensland
- Task 4 (a) – review historic performance
- Task 4 (b) – preliminary analysis of the method outlined in the draft AAPA supplement to Austroads pavement design guideline
- Task 5 – draft interim report
- Task 6 – scoping for Year 2.

A literature review was conducted for three types of long-life pavements, namely: (i) fully flexible (ii) semi-rigid and (iii) rigid pavements. The literature review summarises current research studies in Australia and overseas. Despite the different definitions adopted by road agencies on long-life pavements, this generally refers to pavements where there is no cumulative damage over the pavement service life and any maintenance work is limited to the surfacing layer. For the purpose of this project, the definition of a long-life pavement is:

> A long-life pavement is a type of pavement in which damage to the foundations or the road base layers will not accumulate under its predicted annual traffic loading provided that correct surface maintenance is carried out.

During the course of the project, the project team has shifted the focus to the fully flexible asphalt pavements. Other pavement types may be considered in the future. In February 2015, the project team received a copy of the AAPA draft supplement to the Austroads Pavement Design Guide. Limited design thickness calculations have been conducted specifically for the Queensland environment. As expected, the proposed AAPA method results in a significant reduction in total structural asphalt thickness when compared to typical international designs and also the current Austroads/TMR design methodology. Based on the calculations conducted, it appears that the critical tensile strain between the JULEA and CIRCLY models is insignificant. It is believed that the reduction in pavement thickness is primarily contributed from the selection of failure strain criteria in the TMR and AAPA methods.

It is noted that a separate Austroads project in FY2014–15 is underway to review the proposed AAPA draft supplement and its potential implications if adopted in the current Austroads pavement design guideline AGPT02-12 (2012). Austroads has raised a number of concerns with AAPA regarding their proposed method. AAPA has subsequently committed to provide a revised methodology that addresses those concerns.

The primary concerns with the AAPA method include the limited availability of Australian calibration sites, and the need to address design reliability. In Queensland, AAPA indicated that only two sites were used. To address this, the second part of this study conducted a search for potential long-life pavement candidate sites in Queensland. Unfortunately, no new sites were found along the major arterial roads included in this study. Information on the two Queensland AAPA calibration sites were collated and briefly summarised in this report.

In the future, subject to funding availability, detailed field investigations could be conducted at the Queensland calibration sites to confirm the long-life status of these pavement sections. However, due to the limited numner of sites it would likely be very difficult to draw meaningful conclusions from the results to the extent necessary to properly calibrate a long life pavement design methodology. At best, the knowledge could provide some additional information on which a policy judgement could be made based on an engineering risk assessment and government investment decisions.

6.2 Proposed Work for Year 2

Austroads is currently awaiting the revised procedure from AAPA that addresses the comments raised at the Austroads Pavements Task Force meeting in June 2015. By the end of FY2015–16, recommendations on the revised APSfL *Supplement to the Austroads Pavement Design Guide* (AAPA 2015) are expected to become available as part of the outcome from Austroads Project TT1826 (*Improved Design Procedures for Asphalt Pavements*).

The tasks proposed for Year 2 of this project are listed as follows:

- Task 1 – refine project scope
- Task 2 – review Austroads recommendation
- Task 3 – identify issues and changes specific to Queensland
- Task 4 – draft long-life asphalt pavements technical note
- Task 5 – interim report.

In this year's study, only a subset of roads were retrieved from the ARMIS database. No new pavement site was identified that met the criteria listed in the AAPA APSfL design methodology. Other sites may be identified in heavily trafficked roads owned by a city council in the future.

Future works beyond Year 2 can focus on carrying out detailed field investigations to confirm the long-life pavement status of selected pavement sections. Visual surface distress (e.g. rutting, cracking) can be helpful as a preliminary screening tool, however, the ultimate goal is to take cores from in-service pavements and confirm if fatigue cracking (also known as bottom-up cracking) can be found. This can also be supplemented by conducting additional deflection testing of in-service pavements.

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APPENDIX A AAPA SUPPLEMENT TO THE AUSTROADS PAVEMENT DESIGN GUIDE, FOR LONG-LIFE ASPHALT PAVEMENT DESIGN (DRAFT)

Australian Asphalt Pavement Association Supplement to the Austroads Pavement Design Guide, for Long Life Asphalt Pavement Design

1 Introduction

This Supplement is for the design of Long Life Asphalt Pavement (LLAP) with an indeterminate structural life, for highway and freeway pavements. It is not intended to be used for the design of pavements with a fixed life or for pavements constructed in slow moving traffic situations such as urban arterial roads. This supplement should be read in conjunction with *AGPT02*. (Austroads, 2012).

3 Construction and Maintenance Considerations

3.2.5 Working Platforms

As compaction of asphalt layers on subgrades with stiffness less than 100MPa, is difficult, it is recommended for a subgrade with a stiffness of less than a 100MPa, that a working platform be established to achieve a stiffness of 100MPa below the asphalt layer.

The working platform should be a minimum sub base quality gravel (CBR > 30%). In many cases to lower moisture susceptibility on greenfield construction the working platform may be cement modified with up to 1.5% cement.

3.7 Pavement Layering Considerations

The LLAP concept utilises a three layer asphalt system. The lowest asphalt layer in this system is designed to be a low void mix fatigue resistant mix, the upper layers are designed as a durable and rut resistant while the intermediate layer is the main structural layer in the pavement.

The high rut resistant durable surface layer may consist of dense grade asphalt or a SMA, typically 14mm. Were an OGA is provided for noise and/or drainage considerations, a 14mm DG surface layer shall be provide immediately below the OGA surface. For dense graded asphalt wearing courses, durability and rut resistance can be improved by incorporating a modified binder into the surfacing layer. In such cases binders such as M1000 or A35P are preferred in order to retain the structural contribution of the wearing course to the overall pavement structure.

The intermediate asphalt layers(s) should consist of conventional (C320, 450 or 600) asphalt DG 20 mix which is durable and rut resistant suitable, for the climatic region.

It is recommended that the LLAP incorporate a low void (in place voids<4%) asphalt fatigue layer, in the bottom 60mm of the pavement. The low void layer is usually achieved by the use of higher binder content in the bottom layer of the DG 20 asphalt mix.

3.12 Maintenance Strategy

LLAP are designed to eliminate fatigue cracking therefore no full/partial depth repairs are expected in the pavement life.

In LLAP all maintenance is expected to be top down resulting form oxidation and/or surface initiated cracking therefor maintenance will be limited to periodic overlays and/or thin mill and re-sheets treatment.

4 Environment

4.3 Temperature

The distribution of yearly and daily temperatures can have a significant effect on the performance of LLAP and should be taken into account in the design of LLAP. For example, traffic loading which occurs at night in the middle of winter, will result in relatively brittle asphalt with a lower (Fatigue Endurance Limit) FEL. At the height of day in the middle of summer, the asphalt has a lower stiffness resulting in higher critical strains, but has a higher FEL.

For LLAP design, the temperature of the asphalt should be characterised by use of the both Weighted Mean Annual Pavement Temperature (WMAPT) and the effective asphalt layer temperature at two extremes of temperature; the hottest summer month midday temperature and the coolest winter month early morning temperature.

The WMAPT is given in Appendix C of the AGTPT-Part 2 and the procedure for calculating the effective layer temperature for the combined asphalt layers is given in Section 4.3.1.

4.3.1 Calculation of Effective Layer Temperature

The temperature at the midpoint of the combined asphalt layers can be calculated Australian modified Bells equation (Roberts et al. (2010)).

$$T_2 = AAF \times \left[8.77 + 0.649 \times T_0 + (2.2 + 0.044 \times T_0) \times Sin\left(2\pi \times \frac{hr-14}{24}\right) + log_{10}\left(\frac{h_i}{100}\right) \times \left(-0.503 \times T_0 + 0.786 \times MMAT + 4.79 \times Sin\left(2\pi\frac{hr-18}{24}\right)\right)\right] \quad (4.1)$$

MMAT	=	Mean Month Air Temperature, for the month of testing
T_o	=	surface temperature at time of day
hr	=	time of test on 24 hour (decimal) clock. (eg: 14.33 = 2.20 pm)
h_i	=	Combined asphalt layer thicknesses (mm)
T_2	=	temperature at mid-point in pavement layer
AAF		*Adjustment Factor,* = 0.0175 *x MMAT*+ 0.6773.

The advantage of this equation is that it uses readily available monthly statistical data from the Bureau of Meteorology, which is available at:
http://www.bom.gov.au/climate/data/index.shtml?bookmark=200
VBA function which can be used in MS Excel can be found in Appendix A8.1

4.3.1.1. Surface Temperature (Summer)

The upper 95th percentile surface temperature (T_o) of the pavement can be estimated by using the hottest summer month average maximum temperature and the simplification of the radiation balance approach, shown following.

$$T_0 = T_a + 25.5\left(Cos(Z)\right) - 2.5 \quad (4.2)$$

Where

T_a	=	Air Temperature (ºC)
T_0	=	Surface Temperature (ºC)
z	=	Zenith angle

The Zenith angle is given by:

Latitude-23.3, for sub-tropical locations and, 0 for locations in the tropics.

For calculation of upper summer temperature time should be taken as 1pm i.e. 13.0 in the modified Bells equation.

4.3.1.1. Lower Surface Temperature (Winter)

For the lower minimum temperature, which occurs in winter early morning, the radiation balance equation collapses to:

To = Ta

For calculation of the lower winter temperature time should be taken as 6am i.e. 6.0 in the modified Bells equation.

4.3.1.1. Effective Pavement Temperature

The effective pavement temperature (T_{eff}) is then taken as the midpoint temperature (T_2) + 2ºC.

5 Subgrade Evaluation

The support provided to the asphalt layers in the LLAP is one of the most important factors in determining the required asphalt thickness.

5.1 Measures of Subgrade Support

For LLAP design the subgrade support shall be characterised in terms of the stiffness (resilient modulus (Mr)). It is recommended that the stiffness be measured form tri-axial testing or determined from back calculation of FWD data.
For stiffness determined from FWD testing, it essential to use the same analysis method in back and forward calculation, therefore if anisotropy is not considered is the back calculation it should not be used in the development of the perpetual pavement design.
It is recommended to validate the back calculated subgrade modulus by comparing the back calculated results to insitu CBR determined indirectly from DCP testing in accordance with Section 5.5 of the *AGPT02*.

5.9 Subgrade Failure Criteria

Research by Nunn et al. (2001) has shown that for asphalt pavement with asphalt thickness greater than 180mm rutting is confined to the asphalt layer(s). However to ensure adequate cover over the subgarde a minimum number of allowable repetitions of $1x10^8$ shall be applied, or 650με, for analysis conducted at WMAPT.

6.4 Asphalt

6.4.1.3 Characteristics for Design

Asphalt stiffness can be determined utilising any of the following test methods:

- Indirect Tension on Cylindrical (IT-CY) samples (AS2891.13.1)
- 4 Point Bending on Prismatic Beams (4PB-PB) samples (AG:PT/T233)
- Direct Compression on Cylindrical (DC-CY) samples (ASHTO TP62 or AASHTO TP79)

All of these test methods have a different definition of time and different stress states.
For the purposes of design, the modulus results need to be converted to equivalent mix stiffness under a haversine load pulse resulting from a moving truck. Comparable stiffness is obtained from the three test methods by using a constant definition of time as shown the following:

- The (IT-CY) should be considered a haversine pulse load with time equivalent to double the rise time.
- The (4PB-PR) Flexural modulus test should be considered a cyclic frequency load with a haversine load pulse of ½ the full load pulse width.
- The (DC-CY) dynamic compressive modulus test should be considered as cyclic harmonic frequency with load pulse equal to the radial pulse time.

The mathematical conversion of these three time definitions is given in section 6.4.3.2.

At extreme temperatures and slow vehicle speeds there can be a difference between the modulus of asphalt in tension and compression due to the stress susceptibility of asphalt. Under normal Australian operating conditions this difference is negligible. However, under extreme conditions (>40ºC) stress susceptibility can exhibits an influence on the stiffness of asphalt mixes. In LLAP design there is a net confining stress on the pavement.

Due to this confining effect it is not recommended that asphalt stiffness determined from tension or pure flexural tests be used for modelling at higher temperatures (>40ºC). For these extreme conditions, it is recommended that modulus be determined based on the results of the dynamic compressive modulus test with utilising a 200kPa confinement.

6.4.2 Factors Effecting Modulus and Poisson's Ratio

The main factors effecting modulus are discussed following. While, it is known that Poisson's Ratio varies as a function of asphalt mix type and temperature, the development and calibration of the design procedure used a constant Poisson's ratio for asphalt (0.35). Therefore this value should be used for all design calculations.

6.4.2.1 Bitumen Class and Content

Australia has four major paving grades of binder C320, AR450, C600 and Multigrade binders. These grades of binders affect the stiffness and temperature susceptibility of the asphalt. At the time of design it is unlikely that the designer will know the exact properties of the binder which will be used in the design. It is therefore more relevant in Australia to use typical properties of binder classes in Australia than measurement of bitumen properties. The use of typical properties has been found to be no less accurate than the use of binder properties in complicated modulus equations (i.e. Bari et al. 2006).

The range of bitumen contents used in Australia for typical mix design does not vary to a great extent. Again at the time of structural design it is unlikely that the final binder content of the asphalt mix design will be known. Therefore it is more relevant, and as accurate, to use typical modulus values than binder content to predict asphalt modulus.

6.2.2.2 Air Voids

Design air void contents do not vary to a significant extent across Australia and have been found to have very low impact on modulus values. It is therefore relevant for Australian conditions that the effect of voids be ignored, provided the design air void range remains in the typical range of Australian mixtures (3.5-5.5%).

6.2.2.3 Aggregates

As all specifications in Australia control the shape and angularity of the aggregate, the effect of aggregate type is not measurable in Australian mixtures. Provided the aggregate comply with the relevant specifications, aggregates type and grading need not be considered in design.

6.2.2.4 Temperature

Temperature has a significant effect on the stiffness of Australian asphalt mixes. Under typical operating temperatures experienced in Australia modulus of Australian mixtures can vary between 25,000MPa to 1,000MPa.

Therefor the effective temperature of the asphalt for the design season must be taken into account during pavement design.

Temperature shall be taken into account by using a polynomial temperature shift factor (a_T) at a 25ºC reference temperature where the shift factor is given by:

$$a_T = 10^{a(T-25)^2+b(T-25)} \quad (6.1)$$

Where:

T	=	pavement design or testing temperature
a and b	=	Fitting coefficients of the polynomial equation

6.4.2.2 Rate of Loading (Time)

The effect of traffic speed is significant, especially between urban and freeway traffic speeds. To determine the modulus at a given loading speed the loading speed needs to be converted to a time of loading.

The time of loading has been found to be a directly related to the strain load pulse time resulting from vehicle loading. For design it should be assumed that the vehicle acts as a haversine pulse with a wave length of 1.8m, the 1.8m wave length should be used regardless of asphalt thickness. The equivalent haversine loading time may be estimated using the following equation.

(6.2)

Where;

t_{hp} = load duration sections

v = speed of traffic (km/hr)

The equivalent loading time (reduced time) at the design temperature shall be determined using the time temperature superposition principal.

The reduced pulse time at the design temperature is then determined using the temperature shift factor (a_T) as shown following

$$t_{hp(r)} = a_T t_{hp} \quad (6.3)$$

Where;

a_T = temperature shift factor

$t_{hp(r)}$ = the reduced load pulse time at the design temperature

6.4.3 Determination of Modulus

6.4.3.2 Laboratory Measurement

The design procedure has been developed from the results of extensive dynamic modulus test of typical Australian production mixes. The procedure makes use of dynamic modulus master curves which can be produced from temperature frequency sweep testing.

For consistency and aid in interpretation it is recommended that master curves only be presented in the equivalent haversine pulse time (t_{hp}) space and represented by a sigmoidal function as shown following.

$$\log(|S_{mix}|) = \alpha + \frac{\beta}{1+e^{(\gamma+\delta(\log(t_{hp(r)})))}} \quad (6.4)$$

Where:

$t_{hp(r)}$ = reduced haversine pulse load at the reference temperature

α = the minimum value of the mix stiffness

$\alpha+\beta$ = The maximum mix stiffness

γ, δ = shape fitting parameters, determined through numerical optimisation of experimental data

It was found that there is no difference between the dynamic modulus determined from AASHTO TP62 or AASHTO TP79 and either method can be used to determine the dynamic modulus of the mix

As the definition of time in other modulus is different to the dynamic modulus tests, if test other than the dynamic modulus are proposed to be used to determine modulus, a frequency conversion will need to be undertaken. It has been found that the following frequency conversions are required to convert modulus from other Australian test methods to the results of dynamic modulus testing.

Resilient Modulus, IT-CY

$$t_{hp} = 2R_t \quad (6.5)$$

Flexural Modulus, 4PB-PR (AG:PT/T233)

$$t_{hp} = \frac{1}{2f_{fm}} \quad (6.6)$$

Dynamic Compressive Modulus, DC-CY (AASHTO TP62 and TP 79)

$$t_{hp} = \frac{1}{2\pi f_{dm}} \quad (6.7)$$

Where;

f_{dm} is the equivalent dynamic modulus frequency

R_t is the rise time (typically 0.04sec)

f_{fm} is the flexural modulus frequency (typically 10Hz)

6.4.3.3 Typical Charts

If the exact mix to be used in the pavement is not known at the time of design, which is typically the case, the typical master curves for Australian mixes (Figures 6A.1 to 6A.6) should be used to estimate the dynamic modulus of the mix as a function of binder class and nominal aggregate size. The information required as an input for the master curves is:

- Vehicle speed, where speed is in km/hr
- Effective temperature (^{o}C) of the asphalt

Alternatively, the stiffness can be determined using the standard temperature shift factors and master curve fitting parameters for the time temperature shift factor, Table 6.1 and the sigmoidal master curve fitting parameters Table 6.2 following. The rate of loading is the equivalent haversine pulse loading time.

Table 6.1 Temperature Shift Factors

	a	b
Conventional Binders	-0.001	0.116

Table 6.2 Master Curve Fitting Parameters

Mix	α	β	γ	δ
DG14-C320	2.379	1.878	0.043	0.706
DG14-C450	2.357	1.860	-0.023	0.735
DG20-C320	2.569	1.715	-0.157	0.818
DG20-C450	2.005	2.328	-0.454	0.647
DG20-C600	1.985	2.363	-0.465	0.658

6.4.5 Suggested Fatigue Endurance Limit

The FEL developed form calibration on full scale test tracks and validated against actual Long life pavements in Australia is shown by the general relationship shown in equation 6.8. This relationship determines the maximum tensile strain where damage is balanced by the healing potential of an asphalt mix and is given by:

$$FEL = 3800 S_{mix}^{-0.34} - 100 \quad (6.8)$$

Where;

FEL = Fatigue Endurance Limit (in micro strain)

S_{mix} = mix stiffness in MPa

7 Design Traffic

The design procedure intent is to design a pavement where the rate of damage is equal to healing potential of the asphalt. Under this scenario the cumulative number of axle loading is immaterial, as the healing potential of the asphalt exceeds the damage done by vehicle loading.

Research by Thompson has shown that asphalt can withstand sporadic overloads and return to endurance limit performance. Therefore the critical vehicle used for perpetual pavement design should be taken as the upper 97.5th percentile axle load.

For the majority of Australian pavements this will equate to a standard axel loaded to 9 tonnes.

8 Design of LLAP

8.1 Mechanistic Procedure

In Summary the procedure consists of:

1. Pavement materials are considered isotropic
2. Visco Elastic properties of asphalt are considered by using vehicle speed and affective layer temperature
3. Response to loading is calculated by linear elastic theory
4. Critical responses are assessed as
    a. Tensile strain at the bottom of the asphalt layers
    b. Vertical compressive strain on subgrade
5. Axle loading consisting of a dual wheeled single axle with a load of 9.0t.
6. Tyre contact stress is assumed to be 750kPa
7. 3 season shall be modelled
    a. Morning loading in winter
    b. WMAPT
    c. Day time loading in summer

8.2.3 Combined Asphalt Layer Modulus

It has been found that the use of a single equivalent layer of asphalt provides results as accurate as multiple asphalt layers. The equivalent modulus of multiple asphalt layers can be determined from use of conservation of the moment of inertia approach as shown following:

$$\left(\sum_{i=1}^{n} h_i\right) E_{eff}^{\frac{1}{3}} = \sum_{i=1}^{n} h_i E_i^{\frac{1}{3}} \qquad (8.1)$$

Where:

E_{eff}	=	is the effective stiffness of the combined layers
h_i	=	The thickness of the ith layer
E_i	=	The stiffness of the ith layer
n	=	Number of asphalt layers

Where the asphalt wearing uses a different binder from that of the base and/or fatigue course, it is not recommended that the asphalt layers be combined into a single layer.

Table 8.1 Mechanistic Design Procedure

Step	Activity	Reference
1	Select a trail pavement	Sec 3.7
2	Determine subgrade stiffness	Sec 5.1
3	Determine working platform stiffness (if relevant)	AGPT 6.2 & T6.4 & 8.2.3
4a	Determine WMAPT	AGPT AC
5	Obtain MMAT for January and July maximum daily temperature for January and minimum daily temperature for July.	
6a	Determine surface temperature (To) for summer maximum and winter minimum	Sec 4.3.1.1
7	Determine effective asphalt layer temperature (Teff)	Sec 4.3.1
8	Determine elastic parameters of asphalt layers for summer, WMAPT and winter	Sec 6.4.3
9	Determine effective modulus of combined asphalt layers	Sec 8.2.3
10	Determine FEL for summer, WMAPT, winter	6.4.5
11a	Approximate the axle load as two circular vertical loads with a total load of 45kN and centre to centre spacing of 330mm and uniform vertical stress of 750kPa	
11b	Determine the critical locations as: 1. Bottom of the asphalt layers (summer, WMAPT, winter) 2. Top of the Subgrade (WMAPT only)	
11c	Input the values into the layered elastic analysis and determine the maximum tensile strain at the base of the asphalt layers and top of the subgrade	
12	Compare calculated horizontal strain at bottom of the to the FEL for summer, WMAPT and winter to the FEL	
13	Compare calculated compressive strain to the allowable strain at the WMAPT	
14	If the calculated strain is less than the FEL than design is acceptable. If not; 1. Select a new pavement configuration and return to step 1, or, 2. Determine the life of the pavement as per AGPT-Part 2	

```
Function Mod_BELLS3(sT0 As Single, sTime As Single, sDepth As Single, sMMAT As Single) As Variant
'*****************************************************************
'   (C) Australian Asphalt Pavements Association
'   VBA for use in MS Excel Function written to solve
'   the modified Bells equation, Roberts (2010)
'*****************************************************************

' Dimension Variables

Dim sTd As Single, sLogDepth As Single,  sFirstBracket As Single,  sLastTerm As Single
Dim sSin14 As Single, sSin18 As Single, cPI As Single, sDecHrs As Single, AAF As Single


 cPI = 3.14159265358979

 AAF = 0.0175 * sMMAT + 0.6773
 sDecHrs   = sTime

 If sDecHrs > 11 Or sDecHrs < 5 Then
     If sDecHrs < 5 Then sDecHrs = sDecHrs + 24
     sSin18 = Sin(2 * cPI * (sDecHrs - 18) / 24)
 Else
   sSin155 = -1
 End If

If sDecHrs > 9 Or sDecHrs < 3 Then
   If sDecHrs < 3 Then sDecHrs = sDecHrs + 24
   sSin14 = Sin(2 * cPI * (sDecHrs - 14) / 24)
 Else
   sSin135 = -1
End If

If sDepth > 0 Then
   sTd = 8.77 + 0.649 * sT0
   sLogDepth = Log(sDepth / 100) / Log(10)
   sFirstBracket = -0.503 * sT0 + 0.786 * sMMAT + 4.79 * sSin18
   sLastTerm = (2.2 + 0.044 * sT0) * sSin14
   sTd = sTd + sLogDepth * sFirstBracket + sLastTerm

   Mod_BELLS3 = AAF * sTd


Else
   Mod_BELLS3 = "err"
End If
End Function
```

Figure A 1: DG14 C320 Mixes

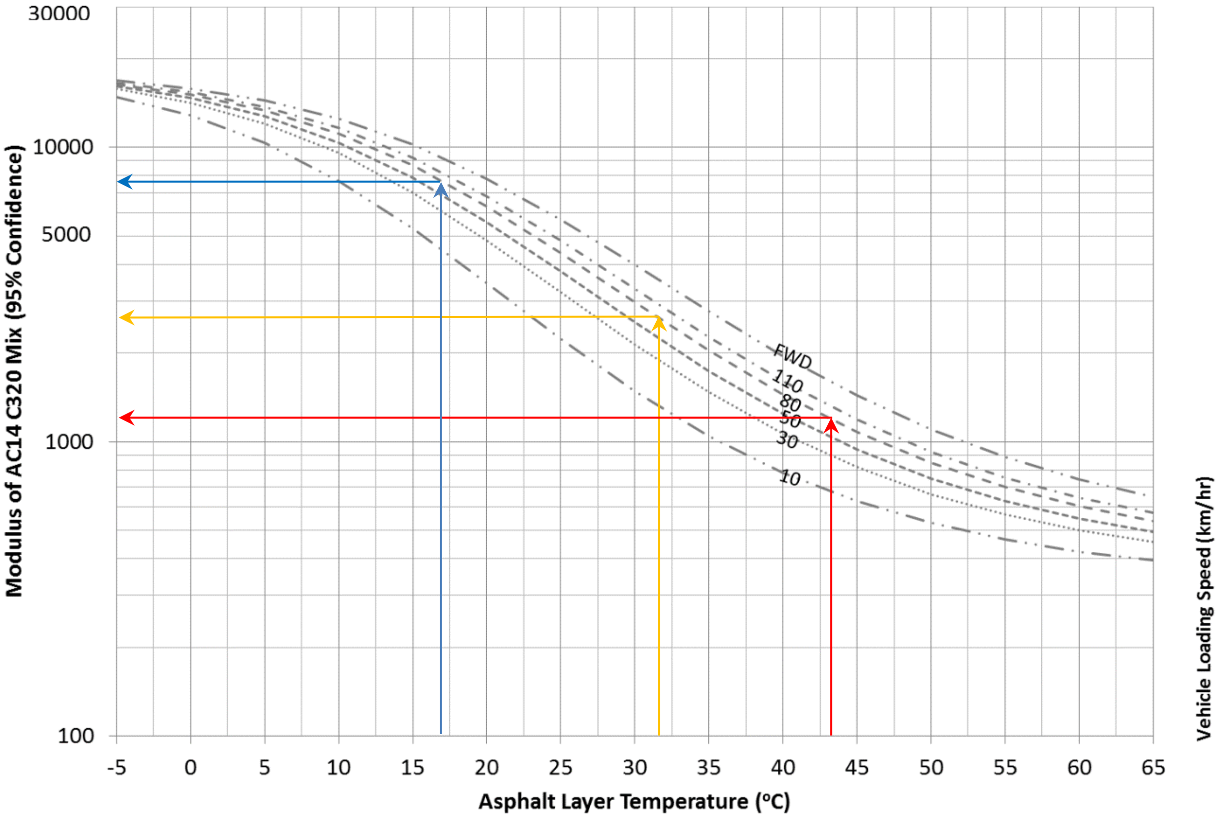


Figure A 2: DG14 C450 Mixes

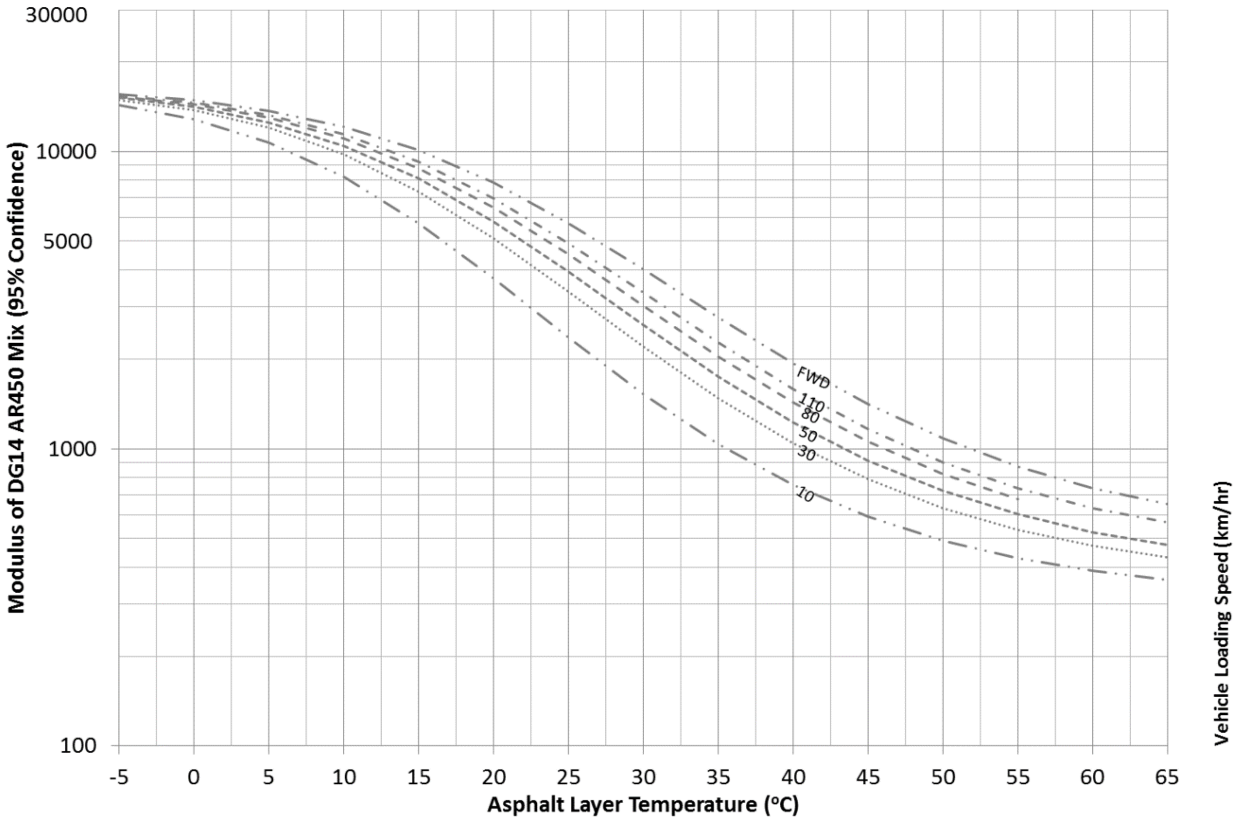


Figure A 3: DG20 C320 Mixes

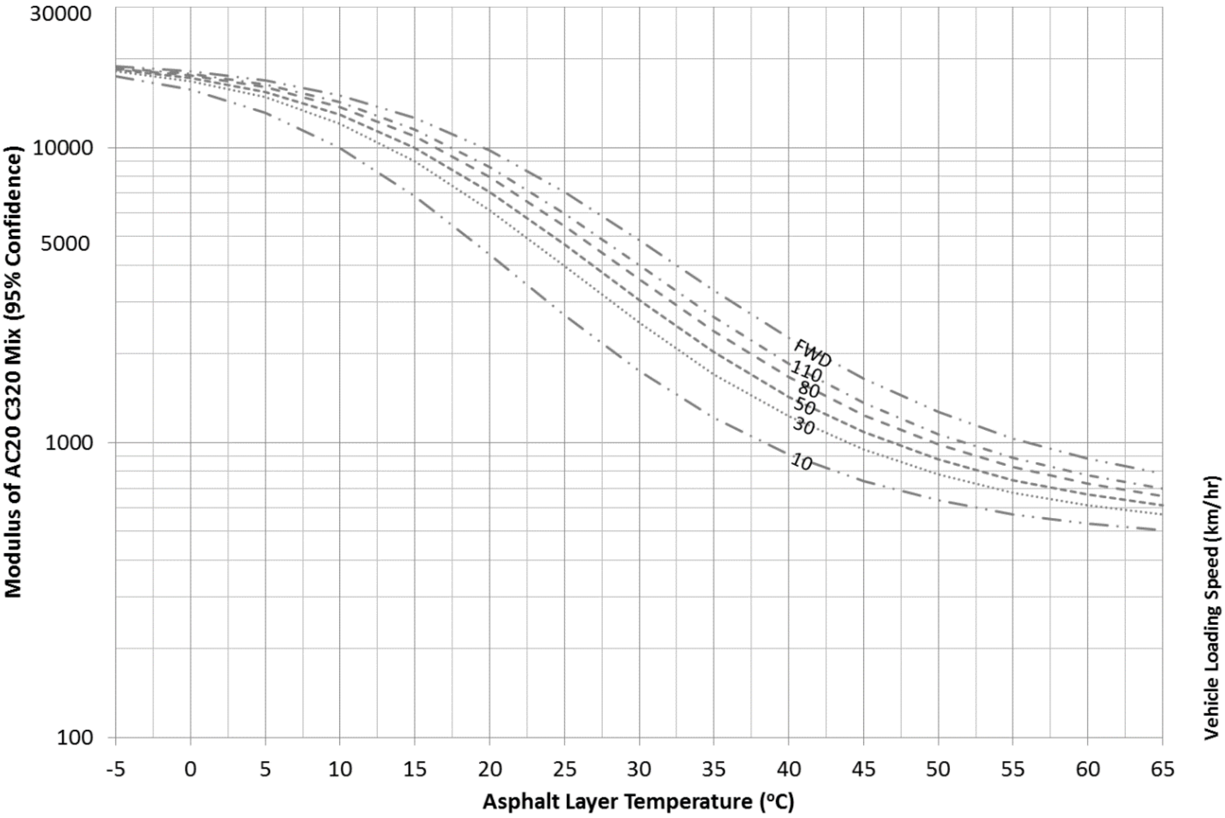


Figure A 4: DG20 C450 Mixes

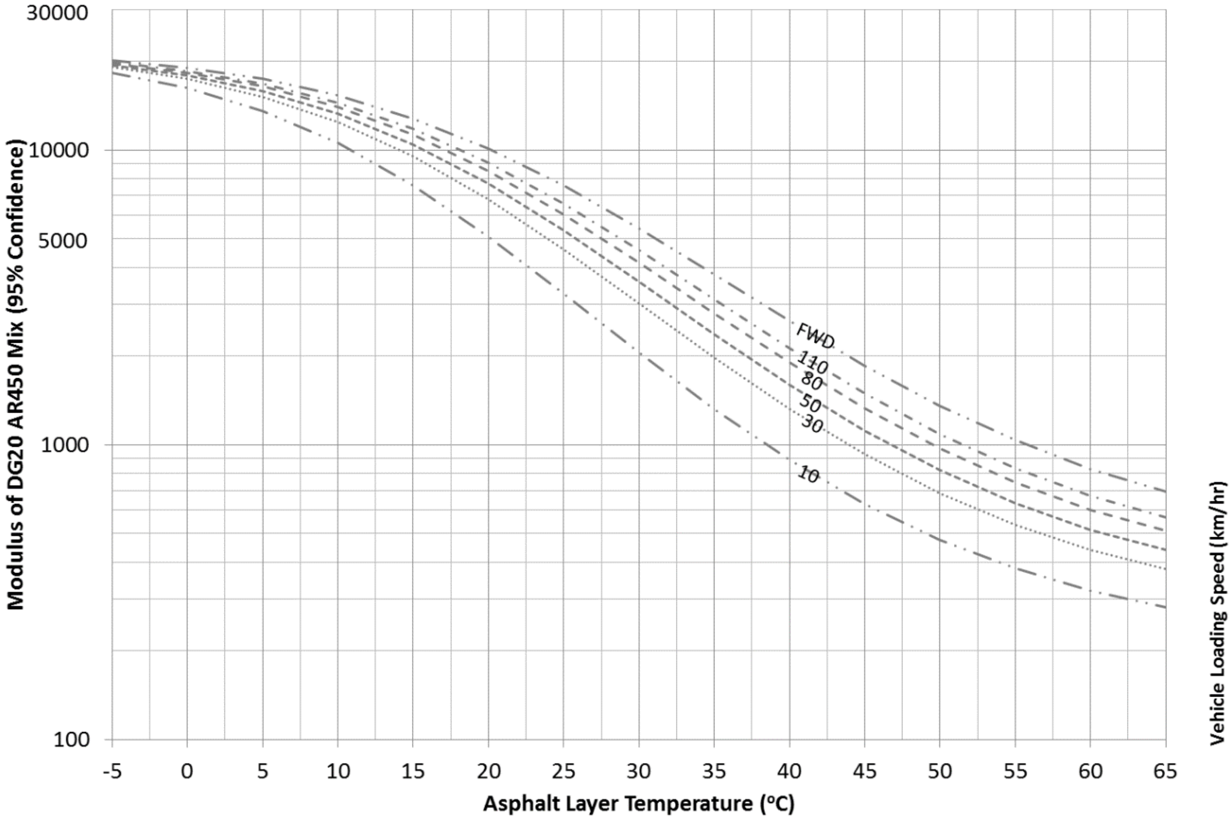
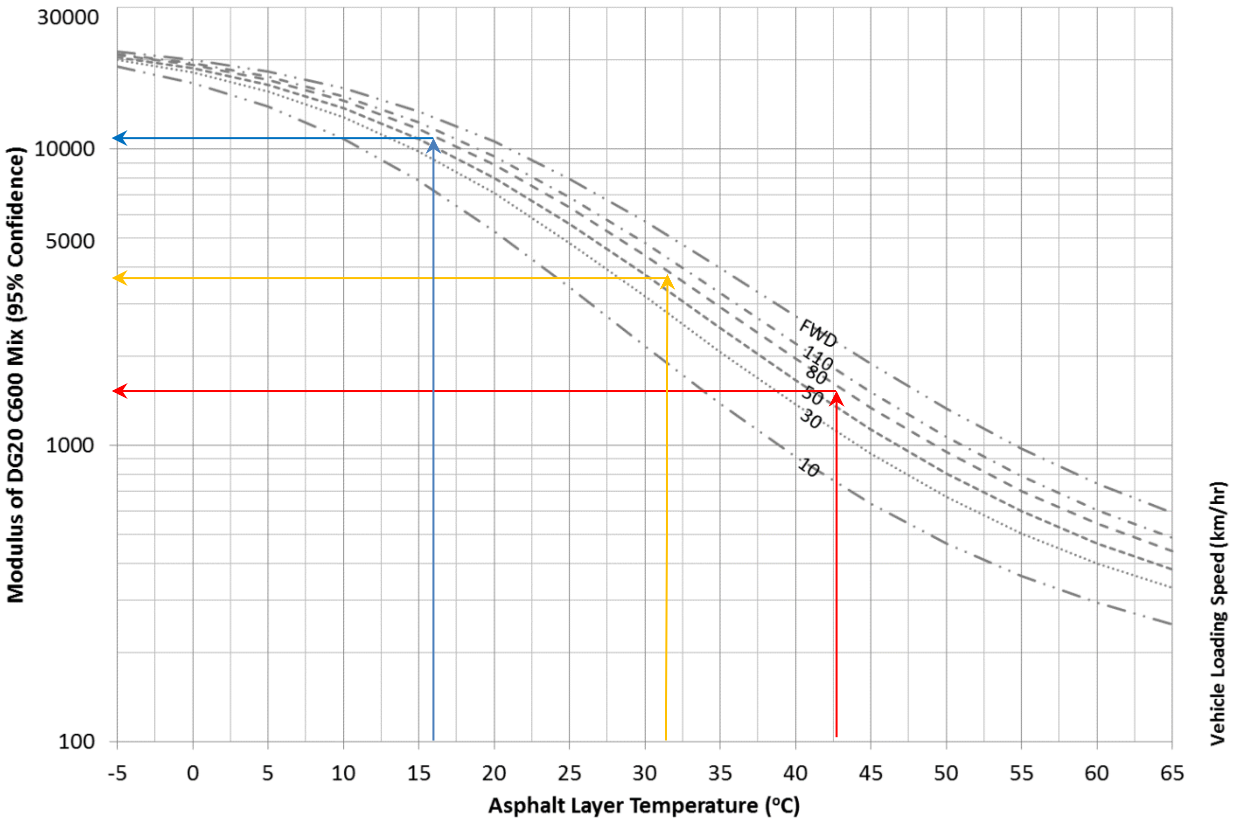


Figure A 5: DG20 C600 Mixes



Design Example

Design Parameters:

a) Location Brisbane Metro

1) Trial Pavement

50mm DG 14 C320
xmm DG 20 C600, trail 170mm
60mm DG 20 C600 (low void)
150mm cement treated type 2.2
Subgrade 50MPa

a) Assume 170mm DG 20 C600 Layer

2) 95th lower Percentile Subgrade stiffness 50MPa.
   a. (DCP max 30mm/blow) (CHECK)
3) Working platform, 150mm cement treated type 2.3 material. Upper modulus 750MPa.
   a. Design modulus 120MPa
4) Determine the WMAPT for Brisbane = 31.9 AGPT-Part 2 Appendix C
5) From all available climate statistics for BOM site 040214:
   - Latitude 27.48
   - MMAT January =(29.4+20.7)/2 = 25^{o}C
   - MMAT July = (20.4+9.5)/2 = 15^{o}C
   - Average maximum January = 29.4
   - Average minimum July = 9.5^{o}C
6) Calculate Surface temperature (T_0)
   a. 95th percentile upper temperature 52^{o}C
   b. Average minimum July 9.5^{o}C
7) Calculate the effective pavement temperature using the VBA function in Appendix A8.1
   a. Effective maximum temperature January 43^{o}C
   b. Effective minimum temperature July 16^{o}C
8) From Figure 6.1-1 DG14 C320 modulus is:
   a. Summer 1225MPa
   b. WMAPT 2570MPa
   c. Winter 8150MPa
9) From Figure 6.1-4 DG20 C600 modulus is:
   a. Summer 1540MPa
   b. WMAPT 3600MPa
   c. Winter 11000MPa
10) Calculate the effective layer modulus from equation 8.1
   a. Summer 1480MPa
   b. WMAPT 3410MPa
   c. Winter 10400MPa
11) Calculate the FEL from equation 6.8
   a. Summer 1480MPa 218$\mu\varepsilon$
   b. WMAPT 3410MPa 139$\mu\varepsilon$
   c. Winter 10400MPa 64$\mu\varepsilon$
12) Using Linear Elastic Analysis (i.e. CIRCLY) calculate critical strain using a 9tonne axel
   a. Summer 202$\mu\varepsilon$
   b. WMAPT 114$\mu\varepsilon$
   c. Winter 48$\mu\varepsilon$

   d. Subgrade 318$\mu\varepsilon$

13) Compare Calculated strain to FEL
   a. Summer 202<218$\mu\varepsilon$ OK
   b. WMAPT 114<138$\mu\varepsilon$ OK
   c. Winter 48<64$\mu\varepsilon$ OK
   d. Subgrade 318 $<$ 650$\mu\varepsilon$ OK

APPENDIX B INFORMATION COLLECTED FROM TWO AAPA CALIBRATION SITES IN QUEENSLAND

B.1 AAPA Long-life Pavement Criteria

Correspondence with AAPA confirmed that two calibration sites from Queensland were used when developing the proposed methodology. LLAP criteria set out by AAPA are summarised in Table B 1. In this section, information that the project team was able to obtain from the two Queensland calibration sites are presented herein, namely:

- Bruce Highway near Kallangur, located 2.2–2.5 km south of Boundary Road (also referred to as Site Q6)
- Pacific Motorway near Loganholme/Shailer Park.

Table B 1: AAPA LLAP criteria

1. Full and partial depth asphalt
2. No cemented material as base or subbase
3. At least 30 years old
4. Exceeded cumulative traffic of 8 x 10^7 ESA
5. No cracking
6. Assessed (by visual inspection) as having over 20 years remaining structural life

B.2 Bruce Highway near Kallangur (Site Q5 and Q6)

B.2.1 Introduction

A number of heavy duty asphalt pavement sites in South East Queensland were studied by different groups (ARRB Transport Research 2001, Youdale 2004). The information for the Bruce Highway site (i.e. Q5 and Q6) is summarised herein using the data from the above sources. It is noted that AAPA only included the Q6 site located near Kallangur in the northern part of Brisbane along the Bruce Highway, in their calibration study. However, because of the proximity between the Q5 and Q6 sites, the information is presented herein for completeness.

During the ARRB Group study in 2001, a number of tasks were completed including site selection, visual assessment survey, deflection testing, visual assessment, sample collection, laboratory testing program and data analysis. Based on the data collected in 2001, additional pavement analysis was conducted by Youdale (2004). A summary of the pavement composition for Site Q5 and Q6 is shown in Table B 2, and a list of testing conducted during the study is presented in Table B 3.

Table B 2: Description and location of Q5 and Q6 sites

Site ID	Description	Section length (m)	Pavement composition
Q5	Bruce Highway (1.3 to 1.5 km south of Boundary Road) Southbound slow lane	200	20 mm AC14 75 mm HIPAR AC14(C320) 50 mm AC14(C320) 200 mm AC20 (C320) **Total structural asphalt thickness = 345 mm**
Q6	Bruce Highway (2.2 to 2.4 km south of Boundary Road) Southbound slow lane	200	20 mm AC14 75 mm AC14(C320) 50 mm AC14(C320) 200 mm AC20 (C320) **Total structural asphalt thickness = 345 mm**

Source: Youdale (2004).

Table B 3: Data sets available

	Condition				Design			Material properties							
Site ID	Roughness	Rutting	Deflection/curvature	Visual rating	Design profile	Design traffic	Subgrade DCP	Binder viscosity	Mix volumetrics	Asphalt fatigue	Refusal density	Wheel tracking	Asphalt creep	Asphalt modulus	Moisture sensitivity
Q5	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Q6	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√

Source: Youdale (2004).

B.2.2 Design and Accumulated Traffic

Youdale (2004) provided details of the cumulative traffic, which is shown in Table B 4. In this Year 1 study, traffic count data were obtained for these sections and the estimated cumulative traffic was around 4.87E+07 ESA at the end of 2014. Details of the calculation and the assumed design traffic parameters are shown in Table B 5.

Table B 4: Q5 and Q6 traffic data to 2000

Site ID	Description	Design life (years)	Opened to traffic	Design traffic (ESAs)	Traffic to date	% Design traffic to date
Q5	Bruce Highway (1.3 to 1.5 km South of Boundary Road)	20	1979	1.4 x 10 ^7	9.2 x 10 ^6	66
Q6	Bruce Highway (2.2 to 2.4 km South of Boundary Road)	20	1979	1.4 x 10 ^7	9.2 x 10 ^6	66

Source: Youdale (2004), originally from ARRB (2001).

Table B 5: Cumulative traffic estimation for Q5 and Q6 sites

Year	AADT (1-Way)	Percentage of HVs (%)	Lane distribution factor (assumed)	ESA per HV (assumed)	ESA per annum (estimated)
1979–2000	Cumulative traffic during this period (Youdale 2004)				9.20E+06
2001	29 249	11.6	0.67	2.5	2.08E+06
2002	33 277	9.2	0.67	2.5	1.86E+06
2003	35 261	10.1	0.67	2.5	2.17E+06
2004	35 261	10.0	0.67	2.5	2.15E+06
2005	35 261	9.9	0.67	2.5	2.13E+06
2006	41 181	10.1	0.67	2.5	2.55E+06
2007	44 518	9.7	0.67	2.5	2.63E+06
2008	45 941	11.3	0.67	2.5	3.17E+06
2009	47 687	11.3	0.67	2.5	3.29E+06
2010	48 961	11.3	0.67	2.5	3.38E+06
2011	49 873	11.4	0.67	2.5	3.46E+06
2012	49 563	10.4	0.67	2.5	3.15E+06
2013	51 706	12.1	0.67	2.5	3.81E+06
2014	52 124	11.4	0.67	2.5	3.62E+06
				Cumulative traffic	**4.87E+07**

B.2.3 Deflection and Curvature

The project team is not aware of any additional project-level FWD deflection data being collected near the site. However, FWD deflection data as shown in Table B 6 were previously reported by Youdale (2004). The deflections exhibit typical heavy duty pavements with over 300 mm of structural asphalt layers.

Table B 6: Q5 and Q6 FWD deflection data

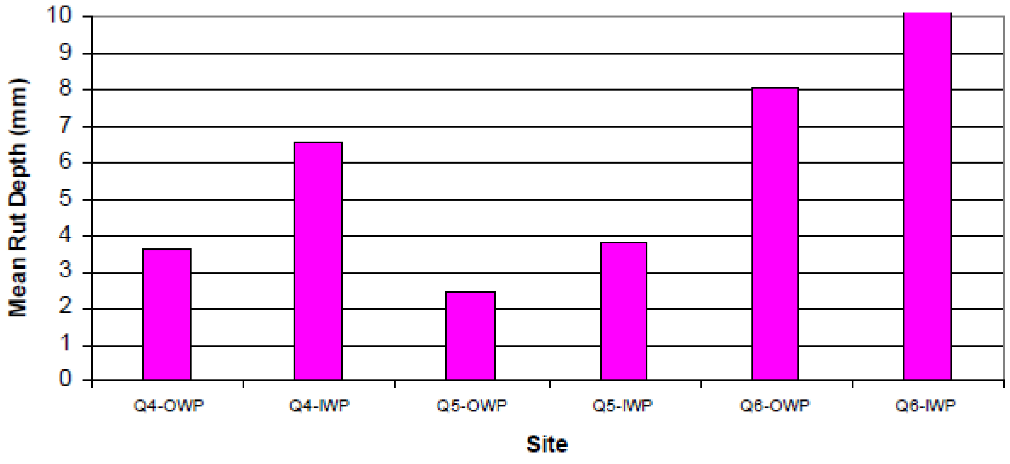
Site ID	Description	Average maximum deflections from FWD (mm)	
		Outer wheel path	Inner wheel path
Q5	Bruce Highway (1.3 to 1.5 km south of Boundary Road)	0.30	n/a
Q6	Bruce Highway (2.2 to 2.4 km south of Boundary Road)	0.30	n/a

Source: Youdale (2004).

B.2.4 Rutting and Textures

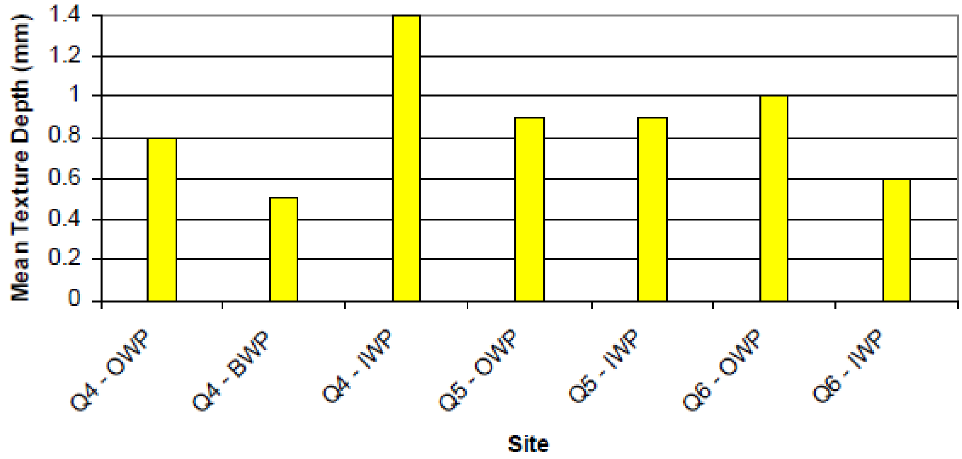
Average rut depth and texture depth was reported by Youdale (2004) (refer details presented in Figure B 6 and Figure B 7).

Figure B 6: Average rut depth across all Queensland sites



Source: Youdale (2004).

Figure B 7: Texture depth across all Queensland sites



Source: Youdale (2004).

B.2.5 Material Testing – Asphalt

A comprehensive set of field and laboratory testing has been carried out at the Q6 (ARRB 2001, Youdale 2004). Details of the testing results for the asphalt layers can be found in the above publications.

B.2.6 Visual and Functional Condition

ARRB Transport Research (2001) provided a description of the site condition in 2001. The relevant comments are summarised in Table B 7.

Table B 7: Summary of visual condition of Q5 and Q6 sites in 2001

Site	Visual and functional condition description
Q5	▪ Moderate longitudinal cracking Sum of crack lengths: 32.5 m (16.3% of total section length) ▪ Slight rutting, mean rut depths of 2.4 mm in outer wheel path and 3.8 mm in inner wheel path ▪ Mean maximum deflection of about 0.2 mm with values varying between 0.1 and 0.35 mm The deflection is very low and uniform ▪ PCI of 90 (i.e. deterioration since construction barely discernible visually)
Q6	▪ Pavement failure in inner wheel path (1 m x 5 m) ▪ Moderate rutting, with several subsections having rut depths greater than 10 mm Mean rut depths of 8.0 mm in outer wheel path and 11.1 mm in inner wheel path Examination of an asphalt slab taken from the outer wheel path indicated that the rutting was confined to the 40 mm wearing course layer ▪ Maximum deflections ranging between 0.2 and 0.35 mm The deflection is low and uniform ▪ PCI value of 68 much lower than the other test site

Source: ARRB (2001).

Youdale (2004) indicated that both Queensland sites were in very sound structural condition, they met the current functional requirements and appear to be stable with time. It is unclear from the report whether an additional visual survey was conducted in 2004.

B.3 Pacific Motorway near Loganholme/Shailer Park

B.3.1 Introduction

This section describes the level of structural damage taking place over time on heavy duty asphalt pavement along a section of the Pacific Motorway in Loganholme/Shailer Park. In order to determine any changes the findings of previous studies by Bryant (2005), Foley (2008) and Radar Portal Systems (2015) were reviewed.

B.3.2 Traffic

The design traffic for the *Queensland Heavy Duty Asphalt Trial* was 2.3 x 10^7 ESAs (33 000 vpd, 8% HVs, 3% annual growth, 20 year design life, 1.1 ESAs/HV and a lane utilisation factor of 0.7) (Bryant 2005). Table B 8 and Table B 9 show the cumulative traffic loading up to 2007, indicating that the pavement had experienced approximately half of its predicted lifetime ESAs between 1997 and 2008. Based on the recent traffic count data, the estimated cumulative traffic was calculated and presented in Table B 10.

Table B 8: Summary of traffic loading up to 2007

Date opened to traffic	Design life	Design traffic (ESA)	Cumulative ESAs and % life to December 2002		Cumulative ESAs and % life to December 2007	
1997	20 years	2.30E+07	6.4E+06	28%	1.11E+07	48%

Source: Foley (2008).

Table B 9: Detailed traffic loading

Year	AADT (two-way)	Southbound (one-way)	% HV	HVs/day	ESA/HV	Lane distribution	ESA per annum
1997	70 767	38 922	7.0	2700	2.1	0.58	9.00E+05
1998	72 349	39 792	6.5	2600	2.1	0.58	1.16E+06

Year	AADT (two-way)	Southbound (one-way)	% HV	HVs/day	ESA/HV	Lane distribution	ESA per annum
1999	76 227	41 925	6.0	2500	2.1	0.58	1.11E+06
2000	77 102	42 406	5.5	2300	2.3	0.55	1.06E+06
2001	81 491	44 820	5.0	2200	2.7	0.5	1.08E+06
2002	86 751	47 713	4.5	2100	2.7	0.5	1.03E+06
2003	91 717	41 423	4.0	1700	3.0	0.49	9.12E+05
2004	95 007	42 747	4.0	1700	3.0	0.49	9.12E+05
2005	98 188	44 380	4.0	1800	3.0	0.49	9.66E+05
2006	99 494	45 224	4.0	1800	3.0	0.49	9.66E+05
2007		45 700	4.0	1800	3.0	0.49	9.66E+05
						Cumulative traffic	**1.11E+07**

Source: Foley (2008).

Table B 10: Cumulative traffic estimation for Shailer Park site

Year	AADT (one-way)	Percentage of HVs (%)	Lane distribution factor (assumed)	ESA per HV (assumed)	ESA per annum (estimated)
1997–2007	Cumulative traffic during this period (Foley 2008)				1.11E+07
2008	49 246	8.4	0.49	3.0	2.21E+06
2009	50 470	8.4	0.49	3.0	2.26E+06
2010	50 522	8.2	0.49	3.0	2.23E+06
2011	51 113	8.3	0.49	3.0	2.28E+06
2012	50 937	8.5	0.49	3.0	2.33E+06
2013	51 875	8.1	0.49	3.0	2.26E+06
2014	53 550	8.3	0.49	3.0	2.39E+06
				Cumulative traffic	**2.71E+07**

Note:

AADT from recent traffic data (Site 140054). Percentage of HVs were obtained from the closest available source. It is noted that there is significant difference in the percentage of HVs assumed when compared with the earlier report from Foley (2008).

B.3.3 Description of Pavement

The section was established in 1996–97 as the Queensland Heavy Duty Asphalt Trial by the Queensland Department of Main Roads. It consists of 730 m of pavement divided into four sections with differing pavement designs. Bryant (2005) provides an in-depth description of the design and construction of the trial pavements.

The three studies that were surveyed for this report use different descriptions and chainages for the sections of the pavement (see Table B 11). The pavement length studied was initially defined by Byrant (2005) and thus in this report, all sections will be referred to by their ‘designation’ (see first column in Table B 11).

Table B 11: Labelling of the pavement sections between reports

Designation (Bryant 2005)	Chainage (Bryant 2005)	Length (m) (Bryant 2005)	General description (Bryant 2005)	Surface description (Foley 2008)	RPS designation (RPS 2015)	RPS chainage (m) (RPS 2015)
Control	1580–1800	220	Unbound granular pavement with DG10 surface	N/A	Section A	0–197
Alternative 1	1800–1970	170	Deep strength asphalt pavement with OG14 surface	OGA	Section B	197–367(2)
Alternative 2	1970–2140	170	Full depth asphalt pavement with OG14 surface	DGA(1)	Section C	367(2)–541
Alternative 3	2140–2310	170	Deep strength asphalt pavement with SM14 surface	SMA	Section D	541–713

1 This surface description is believed to be incorrect for Alternative 2.
2 This chainage 367 is inferred from the RPS report (i.e. it is not documented in Radar Portal Systems (2015)).

Source: Bryant (2005), Foley (2008), RPS (2015).

The Queensland Heavy Duty Asphalt Trial road was opened to traffic in 1997 with a design life of 20 years and design traffic of 2.3 x 10^7 ESAs. The trial site is currently still in service.

B.3.4 Summary of Data Collected – Deflection and Curvature

Table B 12 summarises the deflection and curvature means presented in Bryant (2005) and Foley (2008). No deflection data has been found for subsequent years.

Table B 12: Deflection and curvature

Test section	Section description	Mean maximum deflection at 1132 kPa (microns) (estimated from chart)						
		1997 late	1998 late	1999 late	2001 mid	2002 late	2004 mid	2006 (calculated)
Alt 1	DSA with 1st and 2nd life (175 mm structural asphalt)	250	250	250	200	250	200	172
Alt 2	FDA (250 mm structural asphalt)	320	320	320	250	350	300	247
Alt 3	DSA with 1st life (215 mm structural asphalt)	200	200	200	170	200	200	159
		Mean deflection curvature at 1132 kPa (microns) (estimated from chart)						
		1997 late	1998 late	1999 late	2001 mid	2002 late	2004 mid	2006 (calculated)
Alt 1	DSA with 1st and 2nd life (175 mm structural asphalt)	80	75	70	65	75	55	43
Alt 2	FDA (250 mm structural asphalt)	100	80	90	80	100	85	59
Alt 3	DSA with 1st life (215 mm structural asphalt)	75	65	70	60	70	60	44

Source: Foley (2008).

B.3.5 Summary of Data Collected – Roughness

Table B 13 shows the two sets of road roughness data available. Bryant (2005) data is the mean over four runs and over all chainages (measured at 20 m intervals) taken at 80 km/h. Radar Portal Systems (2015) data was taken in a single run by laser scan at 10 m intervals at 60–80 km/h and the values below are averaged over all chainages. NAASRA roughness values (instead of IRI) are presented in Table B 13 as they were the units used in Bryant (2005).

The results show an overall increase in roughness in all sections, except for Alternative 2. It is worth noting that the middle lane of Alternative 2 had 15% surface patching (Section B.3.8), which may have contributed to the increased smoothness of that section.

Table B 13: Road roughness

Section	Mean roughness 1997 (Bryant 2005) (NAASRA roughness count)			Mean roughness 2014 (RPS 2015) (NAASRA roughness count)		
	Outer lane	Middle lane	Inner lane	Outer lane	Middle lane	Inner lane
Control	36	38	44	45	43	52
Alternative 1	47	44	46	52	50	54
Alternative 2	42	58	56	48	55	43
Alternative 3	45	46	54	57	53	49

Source: Bryant (2005), Radar Portal Systems (2015).

B.3.6 Summary of Data Collected – Rutting

Table B 14 shows mean rut depth data collected in 2006 and 2014. The 2006 data was collected at 10 m intervals for the three 170 m long sections in both wheel paths (Foley 2008). More complete data is available in Foley (2008).

The data shows an increase overall increase in rutting for all the pavement sections, although given the approximate nature of the 2006 data, further validation maybe required.

Table B 14: Rut depth

Section	Mean (rounded) rut depths 2006 (mm) (Foley 2008)	Mean rut depths 2014 (mm) (RPS 2015)		
		Outer lane	Middle lane	Inner lane
Control – OWP	Nil data	0.8	2.2	1.5
Control – IWP	Nil data	4.0	4.0	7.3
Alt 1 – OWP	1	1.7	1.9	2.7
Alt 1 – IWP	1	2.1	2.5	5.2
Alt 2 – OWP	2	1.9	1.1	5.8
Alt 2 – IWP	3	3.9	3.4	6.2
Alt 3 – OWP	0	2.7	1.7	4.6
Alt 3 – IWP	0	2.5	5.0	7.3

B.3.7 Summary of Data Collected – Texture

Table B 15 shows mean texture depths. The 2006 data was measured using the sand patch test method at three chainages in each pavement section. Data from 2014 was collected by laser and was extracted for every 10 m (Radar Portal Systems 2015).

The results show an overall reduction in texture depth as expected, except for Alternative 2, which remains the same or higher.

Table B 15: Texture depth

Pavement	Mean texture depths 2006 (mm) (Foley 2008)	Mean texture depths (SMTD[(1)]) 2014 (mm) (RPS 2015)		
		Outer lane	Middle lane	Inner lane
Control – OWP	Nil data	0.4	0.3	0.4
Control – BWP	Nil data	0.4	0.4	0.3
Alt 1 – OWP	1.9 and 2.2	0.7	0.6	0.7
Alt 1 – BWP	2.4	0.7	0.8	0.7
Alt 2 – OWP	0.6 and 0.7	0.7	0.6	0.7
Alt 2 – BWP	0.5	0.7	0.7	0.7
Alt 3 – OWP	Nil data	0.5	0.4	0.5
Alt 3 – BWP	Nil data	0.6	0.5	0.5

3 Sensor Measured Texture Depth (SMTD). Median and Tilted MPD are also available.

Source: Foley (2008), Radar Portal Systems (2015).

B.3.8 Top Surface Analysis

Table B 16 provides a summary of the top surface analysis. For 1997–2006, the observations come from visual inspections of the pavement. The 1997–99 comments in Table B 16 are summarised from cracking diagrams presented in Bryant (2005). The 2006 comments are taken directly from Foley (2008) and give no indication of location within the pavement sections. For 2014, the observations have been made by analysing surface imaging data that was assessed using the ROCOND 90 method (Radar Portal Systems 2015). The surface imaging data reports on crocodile cracks, longitudinal cracks, transverse cracks, block cracks, potholes, delamination and surface patches (Radar Portal Systems 2015).

Due to the differences in method and resolution of data collection it is difficult to make conclusive comparisons between the surface conditions of the pavement at different dates. Overall it appears that cracking has increased over time in all the pavements, with Alternative 2 having the most cracking (and surface patching was reported in the middle lane).

Table B 16: Summary of top surface analysis

Designation	Visual report 1997 (Bryant 2005)	Visual report 1998 (Bryant 2005)	Visual report 1999 (Bryant 2005)	Visual report 2006 (Foley 2008)	Surface imaging 2014 (RPS 2015)
Alternative 2	1. Excess bitumen Ch. 400 2. Ch. 490–510 clear of defects	1.Rim mark continues for 50 m 2. Excess bitumen worn away 3. Crane movement derived stone loss, ‘gouging’; oil spills and tyre-rim marks. 4. Ch. 490–510 clear of defects	1. Rim mark less defined in trafficked areas 2. Crane marks worn away 3. Ch. 490–510 clear of defects	1. Good condition 2. Several short transverse cracks 3. One short longitudinal crack-sealed	1. Approx. 14% longitudinal cracks in outer and inner lanes 2. 15% surface patch in middle lane 3. < 1% crocodile and transverse cracking

Source: Bryant (2005), Foley (2008), Radar Portal Systems (2015).