ANNUAL SUMMARY REPORT

P9 Cost-effective Design of Thick Asphalt Pavements: High Modulus Asphalt Implementation

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SUMMARY

Current Austroads Guides indirectly specify that asphalt pavements in Queensland must be thicker than those in other Australian states owing to the prevailing environmental and traffic conditions. The incorporation of high modulus asphalt layers would increase overall pavement stiffness, at the same time maintaining the same structural performance. In this project the pavement structural design using the French class 2 high modulus asphalt (Enrobé à Module Élevé – EME2) is discussed. The EME2 concept significantly differs from the way dense graded asphalt is currently modelled in Australia.

A comprehensive discussion is provided on the French pavement design system, including modelling and performance assessment of pavement structures containing EME2 mixes. The mix design of EME2 asphalt is fully performance-based, which significantly differs to the methodology currently used for normal and heavy duty asphalt mixes in Australia. Mix design considerations are not discussed in this report, as a comprehensive overview, including developing tentative specification limits, will be provided in an Austroads report to be published later this year.

A successful EME2 demonstration trial was constructed in February 2014 at Cullen Avenue West, Eagle Farm. The pavement design of the trial section is provided in this report. Also, initial testing of the trial, including the control section is included.

For a successful technology transfer it is paramount that an applicable and reliable pavement design methodology should be available in Australia for designing pavements containing EME2 asphalt. There are three options identified and discussed in the report. The application will depend on the strategic directions given by TMR and the level of completeness of the framework required for these three options. While developing the strategy for implementation, it should be emphasised that the introduction of a new technology always requires periods of transition.

The three options are summarised as options 1, 2 and 3. Option 1 would utilise the current Austroads pavement design methodology, while option 2 and 3 are applicable for future developments and they require work to be completed before they can be partially or fully implemented. Performance monitoring of the trial section will also provide input into the development and validation of a suitable pavement design method.

The report provides a draft Amendment to the TMR Pavement Design Supplement for designing pavements containing EME2 mixes. This methodology (option 1) would enable the application of the EME2 technology. However, since option 1 – which is strictly an interim measure – disconnects the performance-based mix design and pavement design, further work is required in the implementation process to enable maximising the benefits of the EME2 technology.

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1 INTRODUCTION

1.1 Background to the TMR Asphalt Research Program

Whilst deep lift asphalt pavements make up only a small proportion of the Queensland state-controlled network, the cost of replacing them is much higher than an unbound granular pavement with a sprayed bituminous surfacing. Current Austroads Guides indirectly specify that asphalt pavements in Queensland must be thicker than those in other Australian states owing to the prevailing environmental and traffic conditions. This is mainly due to assumptions associated with asphalt properties (i.e. stiffness and fatigue) and durability effects in certain traffic and environmental climates. As a result, asphalt pavements in Queensland are thicker and therefore more expensive than pavements designed for the same traffic conditions in other Australian states. There are significant cost savings to be realised if Austroads assumptions are found to be conservative in this context – as some overseas research indicates. In addition, alternative products, such as high modulus asphalt, may likewise deliver real savings to TMR if the cost associated with the product and the design thickness result in cost reduction.

The TMR Asphalt Research Program aims to test overseas theories and alternative asphalt products under Queensland conditions in order to raise confidence levels with respect to their performance so that these efficiencies can be implemented to TMR's benefit.

1.2 Background to Cost-effective Design of Thick Asphalt Pavements: High Modulus Asphalt Implementation Project (TMR Project P9)

Full depth asphalt thicknesses in excess of 400 mm are being designed and constructed on urban heavily-trafficked roads in Queensland. This is, on average, thicker than that adopted in other states. This is mainly related to the 'temperature effect' assumptions adopted in the Austroads pavement design procedures. Queensland needs to investigate and consider options to reduce the thickness of these heavy duty asphalt pavements as this would deliver more cost-effective pavement structures without compromising the long-term performance and productivity. The application of high modulus asphalt, as an alternative to current practice, would support this approach.

TMR is currently following the national approach in terms of pavement design and modelling. This results in a low design modulus of dense graded asphalt layers having to be adopted owing to the high air and pavement temperatures in Queensland. The incorporation of high modulus asphalt layers would increase overall pavement stiffness, at the same time maintaining the same structural performance. In this project the pavement structural design using the French Class 2 high modulus asphalt (Enrobé à Module Élevé – EME2) was discussed. The EME2 concept significantly differs from the way dense graded asphalt is currently modelled in Australia. Clarification is required on its most appropriate use, including the possible use of transfer functions and design modulus in line with design reliability.

1.3 Objectives of TMR Project P9

The EME2 technology transfer is a complex issue and the complete procedure is covered by other research projects, in line with the objectives of this project. Other related projects are:

- TMR project P10 – *Characterisation of Asphalt Fatigue at Queensland Pavement Temperatures*
- Austroads project TT1826 – *Improved design methods for asphalt pavements*
- Austroads project TT1908 – *High modulus high fatigue asphalt (EME) technology transfer*.

Austroads project TT1908 is a continuation of project TT1353, which was closed in 2012–13. An output of TT1353 with interim findings and recommendations was published in *EME Technology Transfer to Australia: An Explorative Study* (Austroads 2013a). The objectives of TT1353 and TT1908 can be summarised as follows:

- investigate the mix design methodology of EME2 asphalt mix, based on available international literature
- investigate requirements and local availability of aggregate type, aggregate grading, and hard penetration grade binder
- provide input for implementation of the EME2 technology in Australia
- provide a comprehensive characterisation of EME2 mix using Australian test methods, including workability, moisture sensitivity, rutting resistance, stiffness and fatigue resistance
- develop tentative specification framework for road agencies for designing EME2.

The above Austroads projects therefore provide a comprehensive input into the mix design part of the EME2 implementation.

The overall scope of TMR project P9 covers the development of structural design procedures for pavements containing EME2, including:

- the positioning and function of EME2 layers in typical Queensland pavement designs
- procedures to predict the modulus of EME2 at different temperatures and loading conditions
- development and validation transfer functions for pavement structures containing EME2.

TMR project P9 is a three-year program, with 2013–14 being the first year. The objectives of the first year are:

- conduct a literature review of published data on EME2 material properties for pavement design purposes
- evaluate the suitability of the current methods in the TMR *Pavement Design Supplement* (Department of Transport and Main Roads 2013) for the design of asphalt pavements consisting of EME2
- prepare an experimental plan for laboratory testing and a field trial to assess the relative fatigue performance of asphalt base with EME2 compared to heavy duty asphalt and Class 600 bitumen
- construct a field trial on a heavily-trafficked road, designed to observe fatigue within 2–3 years (under-designed in thickness); the pavement will be instrumented to measure pavement temperature and strains
- develop laboratory testing for the year-2 program
- prepare a progress report and present to TMR.

1.4 Structure of the Report

In Section 2 the EME2 mix design is discussed in general and Section 3 provides a summary of the historical development of pavement design using EME2 in France, including design methodology and specification requirements. Section 4 expands on Section 3 and provides a detailed explanation of the catalogue system, while Section 5 covers the general mechanistic procedure (GMP).

Section 6 puts the EME2 pavement design into the context of the Australian pavement design system and Section 7 contains the pavement design of the demonstration trial using both the

French method and the Australian method. Section 8 provides an assessment of how to implement EME2 in pavement design in Australia.

2 EME MIX DESIGN

Road authorities are continually seeking better ways to design long-life or perpetual pavements. The search for value-for-money pavements in areas of high traffic loading has led the concrete and asphalt industries to develop cost-effective, high-performing alternatives to conventional pavement design.

EME was developed in the mid-seventies in France and provides a high-performance asphalt material for use in heavy duty pavements, specifically suitable in the following situations:

- pavements carrying large volumes of heavy vehicles and requiring strengthening to protect underlying layers
- where there are constraints to the allowable pavement thickness, especially in urban areas or motorways, where geometric constraints persist
- heavily trafficked areas, such as slow lanes, climbing lanes, bus lanes and airport pavements, where there is a need for increased resistance to permanent deformation.

The EME technology is predominantly used for structural asphalt layers, i.e. base layers, which are referred to as base and foundation layers in the French terminology.

The French mix design approach utilises various steps in general asphalt mix design. For AC-EME it requires the utilisation of all four steps, where the next following step should always be conducted once the previous step has been met or finished. Testing levels and associated requirements are listed in Table 2.1

Table 2.1: Testing levels and requirements for AC-EME

Step	Test method	Test type	Reference standard	Requirement
0	Grading and binder content (only for non-trafficked areas)	General + empirical	EN 12697-2 EN 12697-1 or EN 12697-39	According to specification requirements
1	Gyratory compaction	General + empirical	EN 12697-31	Gyratory compactor, % void at different gyrations
	Void content	General + empirical	EN 12697-6	Specifications on the percentage of voids based on the gyratory compactor test (direct height-based measurement) For cores EN 12697-6, C method (bulk density - sealed specimen)
	Water resistance	General + empirical	EN 12697-12	
2	Wheel tracking	General + empirical	EN 12697-22	Wheel tracking, large device (for asphalt mixes designed for axle loads greater than 13 tonnes), 30 000 cycles, 60 °C
3	Stiffness modulus	General + fundamental	EN 12697-26	Two-point bending test, complex modulus, 15 °C, 10 Hz
4	Fatigue	General + fundamental	EN 12697-24	Two-point bending test, 10 °C, 25 Hz

Source: Delorme, Roche and Wendling (2007) and the cited EN standards.

The demonstration study (Austroads 2013a) highlighted that for a successful technology transfer it is important to select corresponding Australian standardised test methods to measure the performance of the design mix. This would also be the basis of setting correct performance limits in specifications; the complexity of this issue was discussed in the study. As discussed in Section

1.3, the EME mix design process, test methods and specification limits are developed under Austroads project TT1908 – *High modulus high fatigue asphalt (EME) technology transfer.* Tentative specification limits and the technical background of this development are summarised in an Austroads report (Austroads forthcoming).

Stiffness and fatigue properties are input values into the mechanistic pavement design. It is important to highlight that in the current Austroads pavement design procedure the fatigue properties obtained from the mix design cannot be directly translated into transfer functions. Transfer functions used in Australia today are not considered suitable to use for EME mixes; the currently used transfer functions were developed for mixes which are completely different to EME mixes (Bonneaure et al. 1977) and the utilisation of these functions would introduce a disconnection between mix performance in the laboratory and field. The correlation between fatigue properties obtained from the laboratory mix design procedure and transfer functions requires long-term performance observations and performance monitoring.

3 FRENCH PAVEMENT DESIGN – GENERAL CONSIDERATIONS

In the French context, the design of pavement structures should be conducted according to the French Technical Guide (LCPC 1997) which has recently been updated and published in the French standard NF P 98-086. Both documents provide the background and general principles of the mechanistic pavement design. They cover the different aspects affecting pavement performance and the design; these are the traffic loading, environment, climatic conditions, underlying bearing capacity, pavement materials and the work quality considerations.

However, for typical pavement design configuration the mechanistic pavement design approach has been used to develop a catalogue of pavement structures (LCPC-Setra 1998) which provides an alternative comprehensive and straightforward thickness design option for relatively standard situations. It provides the same outcomes as the mechanistic pavement design; however, it saves the multi-step process.

The French *Catalogue of typical new pavement structures* (LCPC-Setra 1998) is used for pavement design in France. It reflects the road conditions in the country (climatic variations, frost etc.) and contains statistics to be used as guidelines, especially as far as winter weather conditions are concerned. This national data is expected to be adapted for regional climatic variations and, within a region, to the position, altitude, topography etc. of the area concerned.

As the catalogue applies to the whole country, the document cannot describe all the possible pavement variations. It only deals with the structures most commonly used on the national network and generally considered as optimal in economic and technical terms.

The 1998 version of the catalogue offers a wide range of techniques, some of which cannot be used everywhere because of the associated high costs. The main contractors have the option to submit variations to the road agency in terms of surface layer selection or the overall pavement structure.

If the main contractors consider drawing inspiration from the catalogue, they must acknowledge that pavements have been designed in accordance with an investment and maintenance strategy specifically designed for the French road network. It is therefore up to the user to adapt these specifications to the specific features of their own network and the investment and maintenance strategies they have defined. In line with these considerations, the *French design manual for pavement structures* (Laboratoire Central des Ponts et Chausées 1997) emphasises the relationship between the pavement design options and road management system. In this regard, the initial construction decisions should be considered in line with the required maintenance work that will be necessary after initial construction. This leads to defining the overall strategy for capital investment and maintenance, which can be assessed based on seeking an economically optimum solution, also taking budget constraints into consideration.

The structural design of the pavement has a direct influence on the nature and frequency of maintenance works needed to sustain the desired level of service. The design chosen for the base layers will determine the threshold of possible interruption to traffic associated with climatic conditions (Laboratoire Central des Ponts et Chausées 1997).

The catalogue (LCPC-Setra 1998) consists of multiple booklets. The booklet *Hypotheses and calculation data (*LCPC-Setra 1998) outlines and gives reasons behind the decisions that were made to arrive at the suggested structures. That document therefore contains all of the numerical values and hypotheses enabling the structures to be re-calculated, and provides a transparent process and the user is not constrained by a ‘black box’ tool.

3.1 Pavement Design and Allowable Strain

If using the general mechanistic pavement design method instead of the catalogue, the allowable strain (as a function of the mix type) and the calculated strain (as a function of the pavement model) should be compared in the design procedure. The properties and requirements discussed in this section are in line with Laboratoire Central des Ponts et Chausées (1997), LCPC-Setra (1998*)* and French standard NF P 98-086 (2011) *Road pavement structural design – Application to new pavements.* Since the latest publication for pavement design is NF P 98-086 (2011), in most cases this standard will be referenced in this report; for completeness and better understanding of the pavement design approach, the other two documents also will be referenced in some instances.

3.1.1 Subgrade and Unbound Granular Subbase

The allowable strain in the subgrade and untreated (unbound) granular layer for medium and heavy traffic (T ≥ T3 – refer to Table 3.4) $\varepsilon_{z,allow}$ is calculated according to Equation 1.

$$\varepsilon_{z,allow} = 0.012 \times NE^{-0.222} \qquad (1)$$

where

NE = traffic loading in equivalent standard axles.

3.1.2 Asphalt Base Layer

The allowable strain in the asphalt base layer $\varepsilon_{t,allow}$ is calculated according to Equation 2.

$$\varepsilon_{t,allow} = \varepsilon_6(10°\mathrm{C}; 25Hz) \times \sqrt{\frac{E(10°\mathrm{C}; 10Hz)}{E(\theta_{eq}; 10Hz)}} \times \left(\frac{NE}{10^6}\right)^b \times k_c \times k_r \times k_s \qquad (2)$$

where

$\varepsilon_6(10°\mathrm{C}; 25Hz)$	=	the fatigue resistance of the asphalt mix, determined at 10^6 loading cycles; in France the test is carried out according to NF EN 12697-24, Annex A at 10 °C and 25 Hz
b	=	is the slope of the fatigue line (-1 < b < 0)
$E(10°\mathrm{C}; 10Hz)$	=	stiffness of the asphalt material at 10 °C and 10 Hz, tested according to NF EN 12697-26, Annex F
$E(\theta_{eq}; 10Hz)$	=	stiffness of the asphalt material at the equivalent temperature θ_{eq} and 10 Hz, tested according to NF EN 12697-26, Annex F
NE	=	traffic loading in equivalent standard axles
k_c, k_r, k_s	=	coefficients as discussed below.

Coefficient k_c

The value k_c is a coefficient which adjusts the results of the computation model in line with the behaviour observed on actual pavements. The value of k_c depends on the asphalt type

(Table 3.1). The value of k_c can also be used to introduce new technologies; in this case a long-term observation or accelerated loading trial is conducted and the value of k_c is derived from the comparison.

Table 3.1: Coefficient k_c, adjustment between model and in situ performance

Material	k_c value
Road base asphalt concrete (GB in French terms)	1.3
Bituminous concrete (BB in French terms)	1.1
High modulus asphalt (EME in French terms)	1.0

Source: NF P 98-086.

Coefficient k_r

The value k_r is a coefficient which adjusts the allowable strain according to the calculated risk of failure. The value of k_r depends on the standard deviation (e.g. scatter) of the thickness (S_h) and the fatigue performance from laboratory testing (S_N). The value of k_r is calculated according to Equation 3.

$$k_r = 10^{-u \times b \times \delta} \qquad (3)$$

$$\delta = \sqrt{S_N^2 + \left(\frac{c \times S_h}{b}\right)^2}$$

where

u	=	variable associated with the risk r (normal distribution), selected from Table 3.9
b	=	the slope of the fatigue line (-1 < b < 0)
δ	=	standard deviation of the distribution of logN at failure
S_N	=	standard deviation of the fatigue test
S_h	=	standard deviation of the pavement thickness
c	=	coefficient linking the variation in strain to the random variation of the pavement thickness; with usual structures c is approximately 0.02 $\frac{1}{cm}$.

S_h is determined according to Table 3.2.

Table 3.2: Standard deviation of the layer thickness

Thickness of the asphalt layers (m)	h ≤ 0.10	0.10 < h < 0.15	h > 0.15
S_h (cm)	1.0	1+0.3x(h-10)	2.5

Source: NF P 98-086.

Coefficient k_s

The value k_s is a reduction coefficient to take into account the effect of a lack of uniformity in the bearing capacity of a soft soil layer (the foundation in French terms) underneath the treated or modified layers. The value of k_s depends on the bearing capacity (surface modulus) of the formation level (Table 3.3).

Table 3.3: Coefficient k_s, adjustment to the lack of uniformity of the formation

Modulus of the soil	E < 50 MPa	50 MPa ≤ E < 80 MPa	80 MPa ≤ E < 120 MPa	E ≥ 120 MPa
k_s	1/1.2	1/1.1	1/1.065	1.0

Source: NF P 98-086.

3.1.3 Reference Axle

In France the stresses and strains are calculated for the model of the pavement structure under the reference axle of 130 kN. Each half axle is comprised of:

- a single dual wheel configuration
- uniformly distributed contact stress of 0.662 MPa on two circular plates of a radius 0.125 m, with a centre distance of 0.375 m (NF P 98-086).

For comparison, the Australian reference load consists of a dual-wheeled single axle, applying a load of 80 kN:

- four uniformly-loaded circular areas
- the contact stress is assumed to be uniform, taken to be 750 kPa, with a centre distance of 0.330 m between the tyres of the half axle (Austroads 2012).

The French reference axle load is shown Figure 3.1 and the half of the Australian reference axle is shown in Figure 3.2

Figure 3.1: French reference standard axle

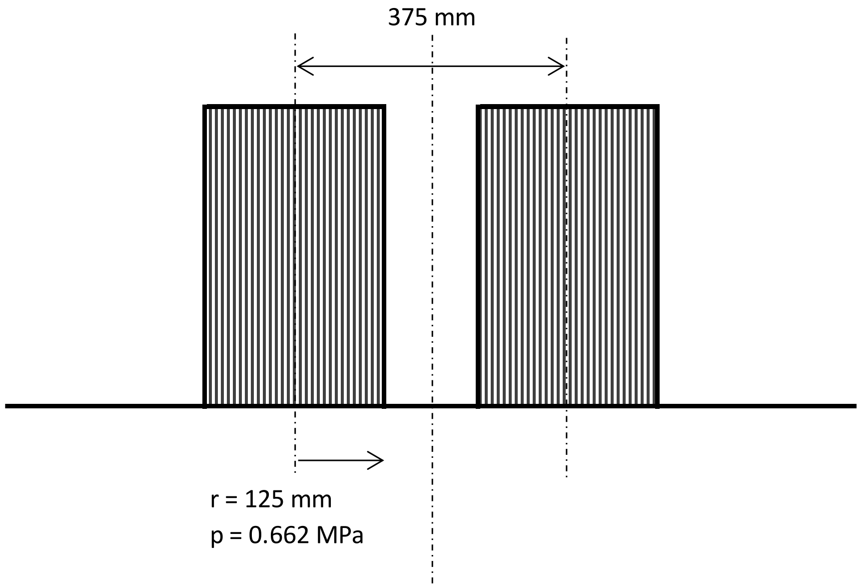
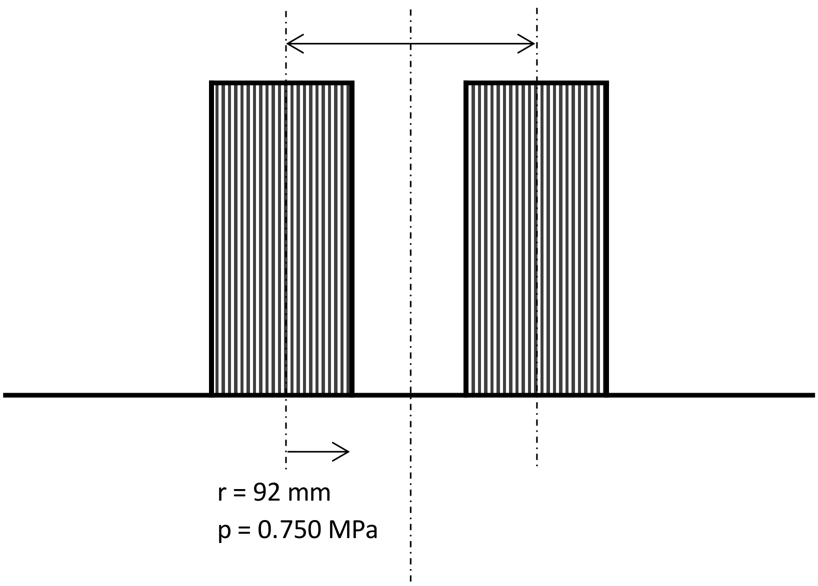


Figure 3.2: Half of the Australian reference standard axle



3.1.4 Traffic Loading

Information on the design traffic is very important to be able to choose the parameters to enter into the calculation and also for calculating the allowable strains and stresses. Heavy vehicles (denoted as PL in NF P 98-086) are defined as vehicles with a payload above 35 kN.

For calculating risk parameters according to Table 3.8, the calculation of the annual average daily traffic (AADT) is required. In France, the AADT is converted into traffic classes as outlined in Table 3.4.

Table 3.4: Traffic classes in France

<table>
<tr><th rowspan="2">Traffic class</th><th colspan="2" rowspan="2">T5</th><th colspan="2" rowspan="2">T4</th><th colspan="4">T3</th><th colspan="4">T2</th><th colspan="4">T1</th><th colspan="4">T0</th><th colspan="4">TS</th><th colspan="2" rowspan="2">TEX</th></tr>
<tr><td colspan="2">T3-</td><td colspan="2">T3+</td><td colspan="2">T2-</td><td colspan="2">T2+</td><td colspan="2">T1-</td><td colspan="2">T1+</td><td colspan="2">T0-</td><td colspan="2">T0+</td><td colspan="2">TS-</td><td colspan="2">TS+</td></tr>
<tr><td>Characteristic value (heavy vehicles)</td><td colspan="2">5</td><td colspan="2">35</td><td colspan="2">65</td><td colspan="2">115</td><td colspan="2">175</td><td colspan="2">245</td><td colspan="2">390</td><td colspan="2">615</td><td colspan="2">950</td><td colspan="2">1550</td><td colspan="2">2450</td><td colspan="2">3875</td><td colspan="2">5920</td></tr>
<tr><td>AADT (heavy vehicles)</td><td>1</td><td colspan="2">25</td><td colspan="2">50</td><td colspan="2">85</td><td colspan="2">150</td><td colspan="2">200</td><td colspan="2">300</td><td colspan="2">500</td><td colspan="2">750</td><td colspan="2">1200</td><td colspan="2">2000</td><td colspan="2">3000</td><td colspan="2">5000</td><td>7000</td></tr>
</table>

Source: NF P 98-086.

The value of AADT is used to calculate the number of heavy vehicles (N_{PL}) over the design period (Equation 4) which is then converted to equivalent design traffic (NE) according to Equation 6.

$$N_{PL} = 365 \times AADT \times C \qquad (4)$$

where

AADT = annual average daily traffic (0.5 times the number of heavy vehicles in two direction on a single-carriageway road, if the total width is greater than 6 metres or in one direction on a dual-carriageway road)

C = the cumulative growth factor over the design period, calculated according to Equation 5.

$$C = \frac{(1+\tau)^n - 1}{\tau} \quad (5)$$

where

n = design period (in years)

τ = annual growth rate (%).

$$NE = N_{PL} \times CAM \quad (6)$$

where

N_{PL} = the number of heavy vehicles over the design period

CAM = mean traffic aggressiveness coefficient.

The value of CAM can be calculated from the measured axle load distribution. If such a distribution is not known, pre-determined values can be used according to Table 3.5 for motorways and the trunk road network and according to Table 3.6 for low volume pavements.

Table 3.5: Traffic aggressiveness coefficient for motorways and the trunk road network

CAM	T2	T1	T0	TS	Tex
Bituminous layers	0.8				
Materials treated with hydraulically bound material and concrete	1.3				
Untreated gravel and soil	1.0				

Source: NF P 98-086.

Table 3.6: Traffic aggressiveness coefficient for low traffic pavements

CAM	T5	T4	T3-	T3+	T2, T1, T0
Bituminous layers	0.3	0.3	0.4	0.5	0.5
Materials treated with hydraulically bound material and concrete	0.4	0.5	0.6	0.6	0.8
Untreated gravel and soil	0.4	0.5	0.6	0.75	1.0

Source: NF P 98-086.

In Table 3.7, an example calculation was performed for traffic class TS+, for a design period of 30 years and 2.9% growth rate. A CAM value of 0.8 was used for a bituminous pavement structure.

Table 3.7: Example calculation for TS+ traffic

AADT (TS+)	**3875**
Design period - p (year)	30
τ (%)	2.9
C	47
CAM	0.8
N_{PL}	66 209 907
NE	**52 967 926**

3.1.5 Calculation of Risk

The pavement design calculations are based on a probabilistic approach and the designer has to choose the correct value for the probability of failure of the pavement after the design period. This is influenced by the design traffic (traffic class) as outlined in Table 3.4. The risk level r is selected according to the traffic class (Table 3.8). Based on the selected risk level the corresponding coefficient of u is selected from Table 3.9.

Table 3.8: Risk level associated with the traffic class (heavy traffic)

Traffic category	Tex	TS	T0	T1	T2
Risk (r%)	1.0	1.0	2.0	5.0	12.0

Source: NF P 98-086.

Table 3.9: The value of coefficient u, based on the risk level

r (%)	u
1	– 2.326
2	– 2.054
5	– 1.645
12	– 1.175

Source: NF P 98-086.

3.1.6 Pavement Support, Categories of the Subgrade (Formation)

The subgrade (referred to as formation in the French system) is categorised according to the bearing capacity measured by the plate loading device as summarised in Table 3.10. The subgrade properties are input into the selection of the k_c value (Section 3.1.2).

Table 3.10: Categories of the formation

Dynamic surface modulus (Ev2) (MPa)	20	50	80	120	200
Formation class	PF1	PF2	PF2qs	PF3	PF4

Source: NF P 98-086.

The calculations and parameters outlined in this section are used later for the pavement design of the Cullen Avenue West trial (Section 7).

4 FRENCH PAVEMENT DESIGN – THE CATALOGUE

This section summarises the French pavement design method using the catalogue. The specifications allow the utilisation of the general mechanistic pavement design procedure as outlined in Section 5, and the utilisation of the catalogue is getting less frequent. However, details on this methodology are summarised in this section in order to provide the historical development behind pavement designs using EME2 material and providing comparison on thickness reduction when EME2 is used instead of normal heavy duty asphalt (GB2 or GB3 in the French terminology).

4.1 Design and Structures

The following parameters are considered for pavement design purposes when utilising the catalogue system:

- initial duration of the pavement design and calculation risk
- climatic data
- traffic
- subgrade (formation) properties
- mechanical characteristics of the pavement materials.

In the French system the pavement structures are composed of several layers, as shown in Figure 4.1.

Figure 4.1: Composition of pavement structures

Surface layer (1)
Base layer
Foundation layer (2)
Subgrade (4)

Cement concrete slab
Foundation layer (3)
Subgrade layer

1: Surface layer = surface course + potential binder course.

2: Certain structures include only one sub-base layer.

3: Under continuously reinforced concrete: dense graded asphalt, under thick slab: open graded asphalt.

4: Potentially covered by a levelling capping layer of untreated gravel.

The groups of structures are as follows:

- thick bituminous pavements: these are composed of a bituminous surface layer on a sub-base of materials treated with bituminous binder
- pavements with sub-bases treated with hydraulic binders: they include a bituminous surface layer on a sub-base of materials treated with bituminous binder
- mixed structures: they include a surface layer and a base layer of bituminous materials on a foundation layer of materials treated with hydraulic binders; additionally, the ratio of the thickness of the bituminous materials to the total thickness of the pavement is 0.5
- cement concrete pavements: the cement concrete layer, which also serves as the surface course, rests on a foundation layer of materials treated with hydraulic binders
- flexible structures: they include a relatively thin bituminous covering, resting on one or more layers of untreated granular materials

- inverted structures: they are composed of a surface layer and a base layer of bituminous materials, on a thin layer of untreated gravel, itself resting on a subgrade layer treated with hydraulic binders that also plays the role of foundation layer.

4.2 Description of Structures and Classes of Materials Used

In this section only the thick bituminous pavements are discussed as only that section is related to EME. The following information is given for the six groups of structures:

- the materials used
- the reference structures retained, i.e. the combinations of different materials used in the sub-base layer
- the bonding conditions of the interfaces between the layers.

4.2.1 Materials Used

The materials used in thick bituminous structures are as follows:

- class 2 base asphalt concrete, GB2
- class 3 base asphalt concrete, GB3
- class 2 high-modulus mix, EME2.

These asphalt materials are used in France as structural layers, because of their strong resistance to fatigue. The GB3 demonstrates better resistance to fatigue than the GB2, but the latter is of interest in regions rich in gravels, or for structures with little traffic on a high-quality platform.

The selected reference structures exclude GB1, which is not permitted on the publicly operated national road network, and class 1 high modulus mixes (EME1) which have gradually been replaced by the class 2 high modulus asphalts, providing higher performance.

4.2.2 Reference Structures

The reference structures for deep lift asphalt pavements are summarised in Figure 4.2.

Figure 4.2: Reference bituminous structures

Surface layer	Surface layer	Surface layer
GB2	GB3	EME2
GB2	GB3	EME2

Source: LCPC-Setra (1998).

For all structures for which the total thickness of the sub-base of bituminous material is less than or equal to 12 cm, levelling of the platform at ± 2 cm is required. This may be achieved, for example, through the addition of a levelling-capping layer of at least 10 cm of crushed rock impermeable to water (fines content < 8%), laid by a guided vehicle and with a granularity appropriate to the thickness applied.

The use of the GB2/GB3 structure is not permitted, even though its pavement design is the same as that of the GB3/GB3 structure and it is therefore more economical. However, at the same thickness, a GB2 will not withstand increasing fatigue as much as a GB3.

4.2.3 Interface Conditions

For thick asphalt pavement design, all of the layers are considered to be bonded.

4.3 Determining the Nominal Thicknesses of the Base Layer

The nominal thicknesses of the base layer, which is referenced in the drawings, are defined on the right edge of the lane carrying the heaviest traffic. It is important to adopt the minimum and maximum technological thicknesses in order to ensure trafficability during construction, to guarantee correct compaction and to achieve satisfactory evenness. Table 4.1 summarises these thicknesses for base layer materials.

Table 4.1: Minimum and maximum nominal thicknesses for base materials

Layer thickness	GB		EME		
	0/14	0/20	0/10	0/14	0/20
Minimum (mm)	80	100	60	70	90
Maximum (mm)	140	160	80	130	150

Source: NF P 98-150-1.

When the nominal thickness is close to the allowable minimum or the maximum, the thickness of the finished layer may result in variation of the target thickness. Therefore for thick bituminous pavement structures the following applies:

- if the base comprises two layers, the thickness of the lower layer is equal to or 10 mm greater than the thickness of the upper base layer
- if the base comprises three layers, the thickness of the deepest layer is equal to or 10 mm greater than the thickness of the intermediate layer, which itself is equal to or 10 mm greater than that of the overlying layer
- for high-modulus mixes in lower base layers, a minimum thickness is set depending on the classification of the platform:
  - 10 cm on PF2s
  - 9 cm on PF3s
  - 8 cm on PF4s (for PF classification refer to Section 3.1.6).

In order to achieve good longitudinal evenness, the maximum thickness of a 0/14 EME2 layer has been set at 130 mm.

4.4 Surface Layers

The surface layer, annotated CS in the French pavement design catalogue, comprises the wearing course and potentially one or two binder (intermediate) courses. On the main (trunk) road network, as well as on non-structural network roads when traffic is greater than $TC5_{20}$ according to Table 4.4, the functions of the wearing and binder courses are necessarily separated:

- the function of the wearing course is to provide surface characteristics (riding comfort, skid resistance, noise, etc.) that comply with the intended objectives
- the binder course(s) protect the base from direct damage from traffic and environmental factors; the binder course is referred to as intermediate layer in the Australian terminology.

This provision is also advised for class $TC5_{20}$ traffic. It leads to the thickness of the wearing course being limited to 40 mm maximum.

When local conditions allow (available resources, supply of different aggregate size fractions, stock management, etc.), the use of aggregates of excellent quality is therefore limited to the wearing course alone, in a thin layer. On the other hand, the binder layer may be produced with lower requirements with regard to aggregates, the availability of which is greater and the cost is lower.

4.5 Design Input Parameters behind the Catalogue

4.5.1 Initial Duration of the Pavement Design and Calculation Risk

The base parameters for pavement design express the investment strategy, which corresponds to a low or moderate risk in structural weakness in the longer term. The concepts of calculating risk and initial duration of the pavement design are therefore introduced, defined as follows in Laboratoire Central des Ponts et Chausées (1997):

> A calculated risk of x% over an initial duration of the pavement design of p years is the probability of defects appearing during the years p, involving reinforcement work comparable to reconstruction of the pavement, in the absence of any structural maintenance intervention.

The values for the initial duration of the pavement design and the calculated risks outlined in the catalogue are given in Table 4.2. They express the quality and service objectives used by the road authority. For the trunk road network the design life is 30 years and it is associated with the low level of risk taken for heavy traffic. This approach enables minimal disruption to heavy traffic by maintenance work limited only to the renewal of the wearing course. It also enables the amount of associated work (repairing shoulders, recovery of safety devices, etc.) to be reduced.

The risks are distinguished according to structure type and traffic. Risk differentiation depending on traffic is also maintained in principle. This option considers that the number of maintenance interventions on the heaviest traffic areas should be limited in order to minimise disruption to users. This consideration, however, is qualified by the need to carry out periodic surface maintenance, regardless of the traffic, as a consequence of ageing of the surface course. Risk differentiation according to traffic thereby enables a satisfactory distribution of structural thicknesses between the classes of traffic, as well as a variation in maintenance thicknesses reflected in practice.

Table 4.2: Risk calculation parameters

Class of traffic		TC2	TC3	TC4	TC5	TC6	TC7	TC8
Initial duration of pavement design	Trunk road network	30 years						
	Secondary road network	20 years						
Risk (%)	Flexible and bituminous pavements	30	18	10	5	2	1	1
	Treated sub-bases and concrete pavements	12.5	10	7.5	5	2.5	1	1
	Foundation of mixed structures	50	35	20	10	3	2	1

Source: LCPC-Setra (1998).

4.5.2 Climatic Data

The water content of the subgrade is taken into account through the bearing capacity of the upper section of earthworks. The seasonal temperature cycles that influence the mechanical characteristics of the bituminous materials are taken into account through an equivalent temperature. The value retained for this equivalent temperature is 15° C in metropolitan France. The calculation hypotheses and pavement modelling for freeze-thaw verification should comply with the directives in Laboratoire Central des Ponts et Chausees (1997). The freeze-thaw calculations are not discussed in this report, as they are not relevant for the Australian climate.

4.5.3 Traffic

In the 1977 catalogue, a heavy vehicle was defined as a vehicle with a load capacity greater than 50 kN which, according to the profile of the heavy vehicles in France, corresponds to a total authorised gross weight of over 90 kN. In the new catalogue, the definition has changed.

Pursuant to French standard NF P 98-082, heavy vehicles are those with total authorised gross weight greater than 35 kN. Therefore a clear distinction should be made between traffic data expressed according to the old definition and according to that of the current standard.

The traffic count data distributed by SETRA since 1990 are relative to visual censuses of utility vehicle profiles with more than two axles, or with two axles with the rear axle carrying twin wheels. These vehicles are integrated with heavy vehicles with total authorised gross weight (TAGW) greater than 35 kN. If the engineer in charge of the project only has data available expressed as vehicles with load capacity greater than 50 kN, these can be converted into the number of vehicles with TAGW greater than 35 kN according to Equation 7, which is only valid in a rural environment.

$$N_{TAWG>35kN} = 1.25 \times N_{load\ capacity>50kN} \quad (7)$$

where

$N_{TAWG>35kN}$ = number of vehicles with total authorised gross weight greater than 35 kN

$N_{load\ capactity>50kN}$ = number of vehicles with load capacity greater than 50 kN.

This ratio expresses that, on average, 80% of vehicles with total authorised gross weight greater than 35 kN have a load capacity greater than 50 kN. This percentage is higher on motorways, where there are fewer small trucks, and lower on secondary roads.

4.5.4 Classes of Cumulated Traffic in the Structural Diagrams

To use the structural diagrams of the catalogue, the traffic data should be used as main entry data. Two series of cumulated traffic classes were defined:

- one for structural network roads, initial duration of pavement design 30 years, annotated TCi_{30}
- one for non-structural network roads, initial duration of pavement design 20 years, annotated TCi_{20}.

Between 1995 and 1996, when the structure calculations were carried out, the hypotheses retained for heavy vehicle traffic growth were as follows:

- for structural network roads: annual linear growth rate of 5% of traffic in the year of entry into service
- for non-structural network roads, annual linear growth rate of 2% of traffic in the year of entry into service.

In reality, the parameter associated with traffic that is used in the pavement design of a structure is the number of equivalent axles (NE) of 130 kN, which is calculated using Equation 6. The values for the average structural damage potential coefficient are given in Table 4.3. These values were derived from data collected by the heavy traffic analysis stations in France. Damage potentials on national roads have been obtained for all HGVs with authorised gross weight higher than 35 kN. The significant change in the composition of vehicle profiles, and in particular the increase in the number of tri-axles, is also considered in these numbers.

The sources of information concerning the composition of heavy traffic enable the average damage potential of heavy vehicles to be calculated according to French standard NF P 98-082.

Table 4.3: Average structural damage potential coefficient in the catalogue

Structure type	Road categories	
	Structural network roads (motorways and expressways	Non-structural network roads (interurban and other roads)
Thick bituminous (including inverted structure)	0.8	0.5
Unbound granular	N/A	1.0
Mixed	1.2	0.75
Semi-rigid and concrete	1.3	0.8

Source: LCPC-Setra (1998).

It should be noted that the values in Table 4.3 are in line with the overall values referenced in Table 3.5 and Table 3.6. These values were used for developing the standardised pavement structures referenced in the catalogue, which is summarised in this report in Table 4.8 for GB3 and EME2 structures. Table 4.4 indicates for each class of cumulated traffic, for each category of road and each type of structure, the number of equivalent axles that was used in the pavement design of the catalogue's structures, as well as the traffic upon entry into service.

Table 4.4: Number of equivalent axles used in the pavement design of the catalogue's structures (in millions)

Traffic upon entry into service (number of heavy vehicles/day/direction), refers to traffic classes		35	85	200	500	1200	3000	7000
Structural road network (motorways and expressways	**Class of cumulated traffic**	**$TC2_{30}$**	**$TC3_{30}$**	**$TC4_{30}$**	**$TC5_{30}$**	**$TC6_{30}$**	**$TC7_{30}$**	**$TC8_{30}$**
	Thick bituminous	0.5	1.3	3	7.5	18	45	106
	Semi-rigid, concrete	0.8	2	5	12	29	73	171
	Mixed	0.8	2	4.5	11	27	68	158
Non-structural road network (interurban and other roads)	**Class of cumulated traffic**	**$TC2_{20}$**	**$TC3_{20}$**	**$TC4_{20}$**	**$TC5_{20}$**	**$TC6_{20}$**	**$TC7_{20}$**	**$TC8_{20}$**
	Thick bituminous	0.1	0.3	0.8	2	5	13	30
	Semi-rigid, concrete	0.2	0.6	1.4	3.5	8.3	20	48
	Mixed	0.2	0.5	1.3	3.2	7.8	19	45
	Flexible	0.3	0.7	1.7	4.3	10.4	26	60

Source: LCPC-Setra (1998).

Table 4.5 provides the upper limit of the classes of cumulated traffic for the two types of roads, expressed as the number of equivalent axles for each type of structure. When the damage potential of the traffic is greater than that indicated in in Table 4.3, the engineer is required to calculate the number of equivalent axles with the chosen damage potential hypotheses and to compare the number obtained to the values given in Table 4.5.

Table 4.5: Upper limits of classes of cumulated traffic expressed in millions of equivalent axles

Structural network roads (motorways and expressways	**Class of cumulated traffic**	**$TC1_{30}$**	**$TC2_{30}$**	**$TC3_{30}$**	**$TC4_{30}$**	**$TC5_{30}$**	**$TC6_{30}$**	**$TC7_{30}$**
	Thick bituminous and inverted (1)	N/A	0.7	2.2	4.5	11.3	30	75
	Semi-rigid, concrete	N/A	1.2	3.6	7.3	18.4	49	122
	Mixed	N/A	1.1	3.4	6.8	17	45	113
Non-structural network roads (interurban and other roads)	**Class of cumulated traffic**	**$TC1_{20}$**	**$TC2_{20}$**	**$TC3_{20}$**	**$TC4_{20}$**	**$TC5_{20}$**	**$TC6_{20}$**	**$TC7_{20}$**
	Thick bituminous and inverted	0.1	0.2	0.6	1.3	3.2	8.6	21
	Semi-rigid, concrete	0.1	0.3	1	2	5.2	13.8	34
	Mixed	0.1	0.3	0.9	1.9	4.8	13.	32
	Flexible	0.2	0.5	1.5	2.5	6.5	17.5	43.5

Source: LCPC-Setra (1998).

1: Refer to Table 4.8 for classes of cumulated traffic.

4.5.5 Mechanical Characteristics

It should be noted that the material properties and performances used behind the catalogue structures comply with the current standards of NF P 98-086 and Laboratoire Central des Ponts et Chaussées (1997). Each sub-base material used in the structural diagrams is identified by mechanical performance as defined in the corresponding standard NF P 98-086. Table 4.6 summarises the values used for the pavement design calculations used for developing the catalogue.

Table 4.6: Bituminous materials (Poisson's ratio is taken as equal to 0.35)

Material	E in MPa (10°C; 10 Hz)	E in MPa (15°C; 10 Hz)	ε_6 (10^6) (10°C; 25 Hz)	-1/b	S_N	S_h (m) (2)	k_c
BBSG (1)	7200	5400	100	5	0.25	variable	1.1
GB2	12 300	9300	80	5	0.3	variable	1.3
GB3	12 300	9300	90	5	0.3	variable	1.3
EME2	17 000	14 000	130	5	0.25	variable	1.0

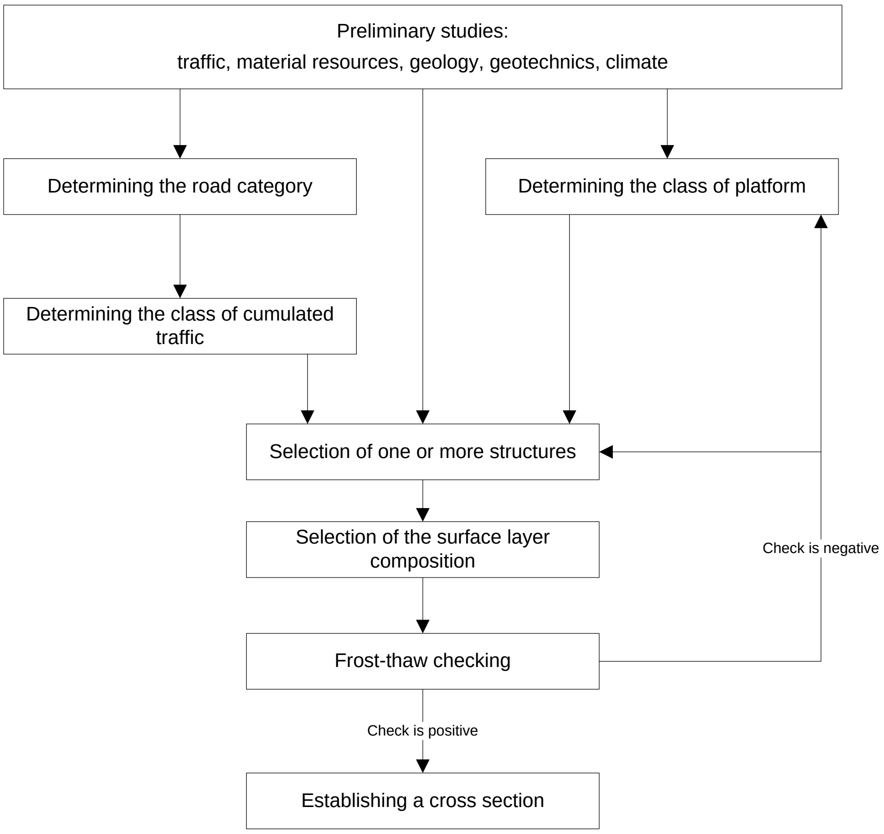
1: Calculations for the structures were carried out with surface and binder courses integrated into a BBSG of a thickness equal to the total mix thickness.

2: Sh (in metres) depends upon the total sub-base thickness (Section 3.1.2).

Source: NF P 98-086.

In general, the design flow chart outlined in Figure 4.3 is used to determine the design pavement thickness.

Figure 4.3: The process for determining a pavement structure



Source: LCPC-Setra (1998).

The thicknesses of the base layers are the nominal thicknesses of the right edge (edge side) of the lane of the pavement carrying the heaviest load. Input data in the selection of the pavement structure are discussed below:

TCi_{30}: class of cumulated traffic

This is determined by the number of heavy vehicles (authorised gross weight > 35 kN) accumulated over 30 years for the trunk road network, on the traffic lane carrying the heaviest load. The limits of these classes are indicated in Table 4.8.

PF_j: platform class

This is determined by the long-term modulus of the pavement formation. The limits of the platform classes are indicated in Table 4.8. The materials must comply with the standards in effect and with the standards' implementation guides.

Surface layer (CS)

This may include one or more layers of asphalt mix (surface course and one or two binder courses). The authorised combinations on the trunk road network are summarised in Table 4.7. In the French terminology the surface course consists of:

- the wearing course, which is the top layer of the pavement structure
- possibly, a binder course, between the base layers and the wearing course.

The characteristics taken into consideration are evenness, skid resistance, drainage, visibility and acoustic considerations; these properties are not discussed further in the report.

Another role of the surface course (wearing course and binder course) is to protect the structural integrity. The binder course, for example, could provide or complete the waterproofing when the wearing course is permeable or not entirely waterproof, like very thin bituminous concrete (béton bitumineux très mince – BBTM) or open graded asphalt (béton bitumineux drainant - BBDr). In a number of situations the need for a binder course must be studied.

In general, the thickness of the surface layer is determined not on the results of computation but essentially according to technological requirements and empirical considerations. There are three basic situations:

- flexible pavement with a granular base layer
- pavements with base layers treated with hydraulic binders
- other pavement types.

The EME mix, being a base layer material is considered an 'other pavement type', and only this is discussed in this section. The thickness of the surface course is determined by the technological limits of each material and the total thickness of the pavement structure, following the pavement design.

The binder course can range in thickness from 50 mm for a maximum aggregate size of 10 mm to 80 mm for a maximum aggregate size of 14 mm. Thickness of approximately 50 to 70 mm is considered most favourable from the point of view of the evenness. The acceptable surface layer compositions are summarised in Table 4.7.

Table 4.7: Permitted surface course variations for different traffic categories

Traffic category	
For TC6 and above	**For TC5 and below**
25 mm BBTM 60 mm BBSG or BBME (1)	25 mm BBTM
40 mm BBDr 60 mm BBSG or BBME (1)	40 mm BBDr
40 mm BBMa 40 mm BBM	

1: In the case of sites susceptible to rutting (slopes, ramps, etc.).
BBTM (béton bitumineux très mince) - very thin bituminous concrete.
BBSG (béton bitumineux semi-grenu) - semi-coarse bituminous concrete.
BBME (béton bitumineux à module élevé) - high-modulus bituminous concrete.
BBMa (béton bitumineux mince a) - class a thin bituminous concrete.
BBM (béton bitumineux mince) - thin bituminous concrete.
BBDr (béton bitumineux drainant) – draining bituminous concrete.

Table 4.8 compares the heavy duty (GB3) and high modulus (EME2) pavement structures as outlined in the catalogue (LCPC-Setra 1998). It can be seen that significant pavement thickness reduction can be achieved when using the EME2 mix type.

Table 4.8: Comparison of GB3 (heavy duty asphalt) and EME2 (high modulus asphalt) pavement structures based on the French catalogue

Traffic category	**Total pavement thickness (mm)**			
	PF3, Ev2 = 120 MPa		**PF4, Ev2 = 200 MPa**	
	GB3 pavement structure	EME2 pavement structure	GB3 pavement structure	EME2 pavement structure
TC8$_{30}$ 94 million HV (75 million ESA)	40 60 110 120 120 **Total 450**	40 60 130 130 **Total 360**	40 60 100 110 110 **Total 420**	40 60 120 120 **Total 340**

TC7$_{30}$ 38 million HV (30 million ESA)	40 60 100 100 110 **Total 410**	40 60 110 120 **Total 330**	40 60 140 140 **Total 380**	40 60 100 110 **Total 310**
TC6$_{30}$ 14 million HV (11.3 million ESA)	40 60 130 130 **Total 360**	40 60 90 100 **Total 290**	40 60 110 120 **Total 330**	40 60 80 90 **Total 270**
	Surface course; in this example the surface course consists of 40 mm BBDr (béton bitumineux drainant – draining bituminous concrete) and 60 mm BBSG (béton bitumineux semi-grenu – semi-coarse bituminous concrete)			
	Base course, GB3, heavy duty DGA			
	Base course, EME2, high modulus asphalt.			

Source: LCPC-Setra (1998).

Note: The traffic aggressiveness coefficient (CAM) is 0.8 according to Table 3.5.

5 FRENCH PAVEMENT DESIGN – GENERAL MECHANISTIC PAVEMENT DESIGN

5.1 Scope of the Design Procedure

French standard NF P 98-086 provides the background to calculations using the general mechanistic procedure (GMP) for pavement structures, including the EME2 pavements. This standard contains updated information compared to the catalogue (LCPC-Setra 1998). It applies to the mechanistic design of new pavements open to heavy vehicle traffic for minimum traffic of T5 or more than 50 000 equivalent axles.

The standard deals with the various pavement structure types: flexible, thick bituminous, semi-rigid, mixed, inverted, or composed of cement concrete. It only deals with standardised pavement materials for the sub-base and surface layers. Cases concerning the use of gravels stabilised with emulsion, special materials and cold materials are not dealt with in this standard. NF P 98-086 is limited to the calculation of the thickness of the layers: it does not deal with the longitudinal section or the cross-section of pavements, i.e. alignment design.

The standardised method can be used for designing urban road networks, except for pavements with the specific characteristics of city centres (presence of significant underground and branched networks, for example). Similarly, information enabling specific loads to be taken into account (aircraft, container carriers, segregated public transport with or without rails, static loads, etc.) is not considered in this standard.

5.2 Principles of the Pavement Design Process

The pavement design method includes a mechanical design component and a structural frost design component; however, in this report the frost design is not considered, as it is not applicable under Australian conditions. The mechanical design component consists of checking that the chosen structure is capable of bearing the cumulative heavy traffic, determined for the established duration of the pavement design. Traffic associated with light vehicles is assumed to have a negligible impact. The design is carried out by comparison of the following:

- the mechanical magnitudes (strains, deformations) representative of the behaviour of the structure of pavements under a reference axle, and calculated using a linear elastic model
- the allowable values of these same magnitudes, a function of the mechanical resistance of the materials bearing repeated loads with which various adjustment ratios are associated, taking into account, in particular, the probabilistic nature of the pavement design process and the discontinuities of the rigid pavements

The stresses calculated in the pavement must therefore be lower than or equal than the allowable stresses; the minimum thickness of the layers is determined by successive repetitions of calculations in order to adhere to this criterion. The structure resulting from the mechanical calculation is then subjected to a frost/thaw check; this is not considered in the Australian context. Also, the thicknesses of the layers are adjusted to incorporate the technological constraints of the layer thickness (Table 4.1).

The design of a pavement requires the project data and the selection of a certain number of pavement design parameters to be defined beforehand. In particular, it is necessary to establish the risk, the duration of the pavement in-service life and the traffic loading. The procedure is explained in Figure 4.3.

5.3 Calculating the Pavement Response

5.3.1 Modelling the Pavement Structure for Thick Bituminous Pavements

The structure is represented as an elastic multi-layered material, with the layers bonded to each other. The rules for determining the stiffness modulus of the materials and the Poisson's ratio to be taken into account are provided in Section 5.4.

The thickness and type of the surface layer and, potentially, the binder layer (layer between the wearing course and base course) are selected beforehand. The pavement design basically addresses the thicknesses of the asphalt base layer(s). For base layers composed of bituminous materials, the thickness of the lower base layer (if more than one) is equal or 10 mm thicker than the upper base layer.

5.3.2 Pavement Design Criteria

The pavement structures are evaluated using the following calculations:

- fatigue failure of the base of bituminous layers: strain ε_t at the base of the bituminous layers must remain lower than the allowable value $\varepsilon_{t,allow}$ calculated according to Equation 2
- the permanent deformation of the unbound layer (subgrade or improved subgrade, referred to as foundation): the reversible vertical deformation ε_z on the surface of the unbound layers must remain lower than the limit value $\varepsilon_{z,allow}$ (Equation 1).

5.4 Material Characterisation in the Design Procedure

High-modulus mixes (AC-EME) are characterised according to the fundamental approach. In the foreword to French standard NF EN 13108-1 concerning mixes, the minimum values for modulus E and for deformation ε_6 (at 10 °C and 25 Hz) are established by the class of the material, which is reproduced in Table 5.1. These values are used to carry out a pre-pavement design before obtaining the results of tests carried out in a laboratory on the material in question.

Characteristics greater than these minimum values for modulus E and for fatigue characteristic ε6 may be taken into account in the pavement design, provided that these characteristics were in fact obtained during the formulation study on materials developed with the worksite components, within the required air voids contents. These characteristics must not, however, exceed the maximum values for the class in question (Table 5.1).

Table 5.1: Minimum and maximum mechanical characteristics for EME to be retained for the pavement design within the context of the fundamental approach

Characteristic	Class	EME1	EME2
Minimum values and conventional values	Modulus at 15 °C – 10 Hz or 0.02 s (MPa)	14 000	14 000
	ε_6 (µdef)	100	130
Maximum values	Modulus at 15 °C – 10 Hz or 0.02 s (MPa)	17 000	17 000
	ε_6 (µdef)	115	145
Values to be applied inclusively	$-1/b$	5	5
	S_N	0.3	0.25
	k_c	1	1

Source: NF P 98-086.

For the pavement design calculations, for materials included in the empirical approach, the values for E in the pavement design calculations are provided in Table 5.2. When the empirical approach is used, the determination of stiffness and fatigue properties is not required in the mix design procedure.

Table 5.2: Characteristics of materials for binder and wearing course layers to be used for the pavement design

Material	Conventional calculation values, E (MPa) 15 °C – 10 Hz or 0.02 s
Thin BB (BBM)	5500
Very thin BB (BBTM)	3000
Draining BB (BBDr)	3000
Mastic road asphalts (ACR)	5500

Source: NF P 98-086.

5.5 Validation of the Calculation Method

Appendix L of NF P 98-086 provides case studies for validating stress and deformation calculations. The display of data and results is intended, for a group of various cases, to check the precision of the calculation method chosen to carry out the calculation of mechanical stresses (stresses, deformations) induced by the reference load in the pavement structure. It should be noted that the standard does not require using specific software, but it requires a multi-layered software to be used to perform the calculations in line with NF P 98-086. The term 'calculation method' refers both to the calculation tool used (calculation charts, semi-analytical calculation programs, finite element software, etc.) and its implementation conditions, which may be taken into account simultaneously in the calculations. In the event, for example, that the finite element method is used, the specification will essentially be bound to its implementation conditions through the fineness of the meshing used and the distance of its boundaries in relation to the load. In order to comply with NF P 98-086, the calculated pavement design stresses must be appropriately:

- identified; deformation or horizontal/longitudinal/transversal/vertical stress, depending on the nature of the permissible value criterion concerned
- localised; depending on the case, under the centre of the wheel or centre of the dual wheel.

The calculated pavement design stresses also must fall within the range of values indicated in the right-hand columns of the tables, under the French reference axle (Figure 3.1).

Table 5.3: Validation of strain calculation, thick bituminous structure, cumulated traffic TC4, platform PF4, non-structural network

Materials	Thickness (m)	Interface with layer below	Stiffness modulus (MPa)	Poisson's ratio	Required range of results
BB	0.025	Bonded	5400	0.35	
EME2	0.10	Bonded	14 000	0.35	ε_{longi} (μdef) wheel centres - 124 ± 1
PF4	Semi-infinite		200	0.35	ε_z (μdef) wheel centres 439 ± 3

Source: NF P 98-086.

Notes: Data set refers to record n° 3 from the Catalogue of Typical New Pavement Structures 1998 edition (LCPC-Setra 1998).
Value greater than 0 means compression; value smaller than 0 means tension.

Table 5.4: Validation of strain calculation, thick bituminous structure, cumulated traffic TC5, platform PF2, structural network

Materials	Thickness (m)	Interface with layer below	Stiffness modulus (MPa)	Poisson's ratio	Required range of results
BB	0.025	Bonded	5400	0.35	
EME2	0.11	Bonded	14 000	0.35	
EME2	0.12	Bonded	14 000	0.35	ε_{longi} (μdef) coupling centres - 66.0 ± 0.4
PF2	Semi-infinite		50	0.35	ε_z (μdef) coupling centres 251 ± 2

Source: NF P 98-086.

Notes: Data set refers to record n° 3 from the Catalogue of Typical New Pavement Structures 1998 edition (LCPC-Setra 1998).
Value greater than 0 means compression; value smaller than 0 means tension.

Table 5.3 and Table 5.4 provide the validation values if the designer wishes to use a specific software package for calculating stresses and strains for the design. CIRCLY could be specified for calculating pavement responses according to NF P 98-086, since the validation requirements are met for

- the pavement structure in Table 5.3
  - ε_{longi} (wheel centres): 124 μdef (microstrain)
  - ε_z (wheel centres) 439 μdef (microstrain)
- the pavement structure in Table 5.4
  - ε_{longi} (coupling centres): 66 μdef (microstrain)
  - ε_z (coupling centres) 250 μdef (microstrain).

5.6 The Effect of Temperature on the Pavement Design

5.6.1 Asphalt Fatigue Properties at a Given Equivalent Temperature

Although in metropolitan France the equivalent temperature of 15 °C is used, Laboratoire Central des Ponts et Chausees (1997) provides a general approach which can be utilised at any selected temperatures. This methodology can be used over a fairly broad range of positive temperatures, based on the calculation that the approximate value for the dependency of the modulus E and the strain ε_6 can be obtained from Equation 8 (Laboratoire Central des Ponts et Chausées 1997).

$$\varepsilon_6(\theta) \times E(\theta)^n = constant \qquad 8$$

where

$\varepsilon_6(\theta)$ = fatigue resistance of the asphalt mix, determined at 10^6 loading cycles at the equivalent temperature θ.

$E(\theta)$ = stiffness of the asphalt material at the equivalent temperature θ.

n = material constant.

In the absence of results of fatigue tests for a given material at different temperatures, a mean value of 0.5 can be selected for n and the equation can be re-organised as in Equation 9.

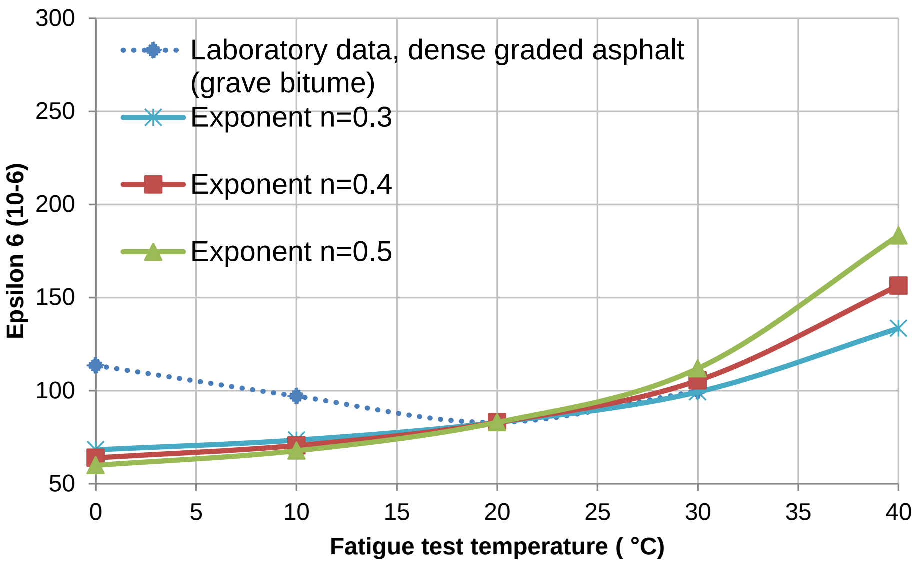
$$\varepsilon_6(\theta_i) = \varepsilon_6(10°C; 25Hz) \times \sqrt{\frac{E(10°C; 10Hz)}{E(\theta_i; 10Hz)}} \qquad 9$$

Equation 9 provides a good model and estimation of the fatigue properties at different temperatures. By using Equation 9, the fatigue properties at any given equivalent temperatures could be readily calculated, given that the standardised fatigue test at 10 °C, 25 Hz and a temperature-frequency sweep for flexural stiffness has been completed.

Bodin et al. (2010) presented a series of fatigue tests at different temperatures, using two different asphalt materials, which provides validation of the above model. An analysis was performed on the published data tested at different temperatures and Equation 8; the $\varepsilon_6(\theta_i)$, i.e. the fatigue properties at different temperatures were predicted from the measured stiffness $E(\theta)$. For demonstration, the prediction was performed using $\varepsilon_6(20\ °C; 25\ Hz)$ for dense graded asphalt and $\varepsilon_6(10\ °C; 25\ Hz)$ for high modulus asphalt (EME2). The predicted values using different exponents (0.3, 0.4 and 0.5) are shown in Figure 5.1 for dense graded asphalt (grave bitumen in French terminology) and for EME2 in Figure 5.2. The figures also show the laboratory test values for comparison.

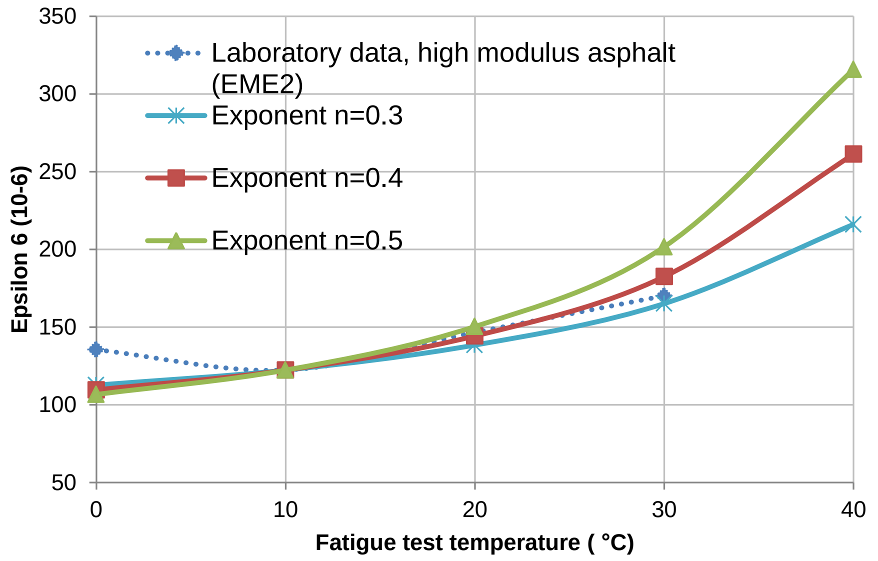
Based on the published data it can be seen that the exponent of 0.5 overestimates the fatigue behaviour at higher temperatures for the EME2 mix. An exponent of 0.4 seems to be closer to the laboratory test values. Unfortunately there is little information available on the high temperature fatigue properties of EME2 mixes and there is a need for local validation. This issue will be addressed in year 2 of the project.

Figure 5.1: Predicted fatigue properties and laboratory test results at different temperatures, dense graded asphalt



Source: based on Bodin et al. (2010).

Figure 5.2: Predicted fatigue properties and laboratory test results at different temperatures, EME



Source: based on Bodin et al. (2010).

5.6.2 *The Equivalent (Design) Temperature in the French Pavement Design Method*

Detailed calculation for the equivalent temperature is provided in Laboratoire Central des Ponts et Chausees (1997), which is discussed in this section. The equivalent temperature is defined according to Equation 10, which is based on the Miner hypothesis.

$$\sum_{i=1}^{n} n_i \times d_i = 1 \qquad (10)$$

where

n_i = the number of equivalent axle passages undergone by the pavement

d_i = the elementary damage.

The elementary damage is expressed in Equation 11.

$$d_i = \frac{1}{N_i} \qquad (11)$$

where

d_i = the elementary damage

N_i = the number of loadings causing fatigue failure at a strain level $\varepsilon(\theta_i)$.

By combining Equations 10 and 11, Equation 12 follows:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} = 1 \qquad (12)$$

Structural design is performed at a constant temperature, referred to as the equivalent temperature θ_{eq}. This temperature is such that the cumulative damage undergone by the pavement over a year, for a given temperature distribution, is equal to the damage that the pavement would undergo with the same traffic but for a constant temperature θ_{eq} (Laboratoire Central des Ponts et Chausées 1997). The equivalent temperature is determined by Equation 13, which is the Miner hypothesis (Equation 10).

$$\sum_i \frac{n_i(\theta_i)}{N_i(\theta_i)} = \frac{\sum_i n_i(\theta_i)}{N(\theta_{eq})} \qquad (13)$$

where

$N_i(\theta_i)$	=	is the number of loadings causing failure due to fatigue for the strain level $\varepsilon(\theta_i)$
$n_i(\theta_i)$	=	is the number of equivalent axle passes undergone by the pavement at a temperature (θ_i)
$N(\theta_{eq})$	=	is the number of loadings causing failure due to fatigue for the strain level $\varepsilon(\theta_{eq})$
θ_{eq}		is the equivalent temperature.

Equation 14 is derived from Equation 13 after re-organising the parameters:

$$\frac{1}{N(\theta_{eq})} = \frac{1}{\sum_i n_i(\theta_i)} \left[\sum_i n_i(\theta_i) \left\{ \frac{1}{N_i(\theta_i)} \right\} \right] \qquad (14)$$

Loading cycles $N_i(\theta_i)$ which cause failure can be deduced from the pavement response at a temperature $\varepsilon(\theta_i)$ and the laboratory test results $\varepsilon_6(\theta_i)$ according to Equation 15.

$$N_i(\theta_i) = \left\{ \frac{\varepsilon(\theta_i)}{\varepsilon_6(\theta_i)} \right\}^{1/b} \times 10^6 \qquad (15)$$

where

$\varepsilon(\theta_i)$	=	pavement response at a temperature
$\varepsilon_6(\theta_i)$	=	fatigue properties from laboratory test results.

The reciprocate of $N_i(\theta_i)$ as defined in Equation 15 equals, by definition, the elementary damage $d(\theta_i)$ at the strain level $\varepsilon(\theta_i)$ (Equation 16).

$$\frac{1}{N_i(\theta_i)} = d(\theta_i) = \left\{ \frac{\varepsilon_6(\theta_i)}{\varepsilon(\theta_i)} \right\}^{1/b} \times 10^{-6} \qquad (16)$$

Equation 17 can be derived by the combination of Equation 14 and Equation 16:

$$\frac{1}{N(\theta_{eq})} = \frac{1}{\sum_i n_i(\theta_i)}\left[\sum_i n_i(\theta_i)\left\{\frac{\varepsilon_6(\theta_i)}{\varepsilon(\theta_i)}\right\}^{1/b} \times 10^{-6}\right] \qquad 17$$

The total elementary damage at different temperatures (right side of Equation 17) is calculated; the equivalent temperature (design temperature) θ_{eq} is the temperature where the elementary damage for $\frac{1}{N(\theta_{eq})}$ equals to the total elementary damage at different temperatures. References to Equation 10 to 17 can be found in Laboratoire Central des Ponts et Chausees (1997) and in NFP 98 086-2011.

The value of $\varepsilon_6(\theta)$ can be obtained from laboratory testing or by using the correlation Equation 18 at 10^6 loading cycles (EN 12697-24–2012).

$$\lg(N) = a + \left(\frac{1}{b}\right) * \lg(\varepsilon) \qquad 18$$

where

N = number of load cycles

a = constant

b = slope of fatigue line

ε = strain (microstrain).

Table 5.5 illustrates an example used during the calculation according to Laboratoire Central des Ponts et Chausées (1997). The temperature distribution, expressed in 5 °C intervals, and with the relative duration of the designated temperature is shown in the table.

Table 5.5: Example calculation of the equivalent temperature

θ_i (°C)	-5	0	5	10	15	20	25	30
Duration (%)	10	12	18	14	18	18	8	2
$\varepsilon_t\ (10^6)$ – pavement response (microstrain)	24	27	32	40	51	68	98	149
$\varepsilon_6(\theta_i)\ (10^6)$ – fatigue performance (microstrain)	95	95	94	92	96	100	110	121

Source: Laboratoire Central des Ponts et Chausees (1997).

The sum of the weighted elementary damage $d(\theta_i)$ is 0.15 in this example. The equivalent temperature is determined by interpolation, where the single elementary damage is equal to this value; this results in an equivalent pavement temperature of 18.7 °C in this example.

The above calculation for the equivalent pavement temperature is based on fundamental mechanics and considers real pavement structure responses and asphalt fatigue properties. The calculation requires detailed input on the pavement temperature distribution. When real and accurate data can be obtained for the pavement structure for a certain climatic environment, it could provide reliable input into the mechanistic pavement design.

5.6.3 Australian Weighted Mean Annual Pavement Temperature

The weighted mean annual pavement temperature (WMAPT) is used in the Australian pavement design system to adjust the in-service modulus from the measured modulus in the laboratory. The

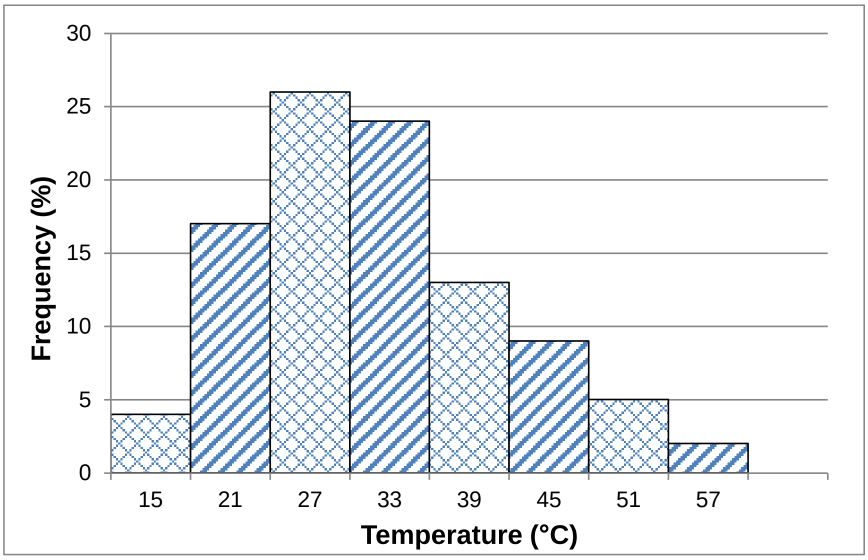
WMAPT value is derived from the weighted monthly average air temperatures following the Shell Pavement Design Manual (SPDM) method (Austroads 2012).

Although this method provides an appropriate approach in pavement engineering for general application, pavement engineers are always facing the challenge to extrapolate beyond the existing knowledge, and predict future pavement behaviour with increased traffic and/or improved material properties. The general mechanistic procedure (GMP) provides an excellent basic tool for this assessment. The level of confidence might be increased by providing more detailed material characteristics as an input for the pavement design by eliminating the 'average' material properties, allowing a more realistic performance prediction to be developed. Powerful computation devices are now available to perform these calculations, however sometimes the lack of input information limits the detailed assessment (Petho 2012).

5.6.4 Pavement Temperature Data

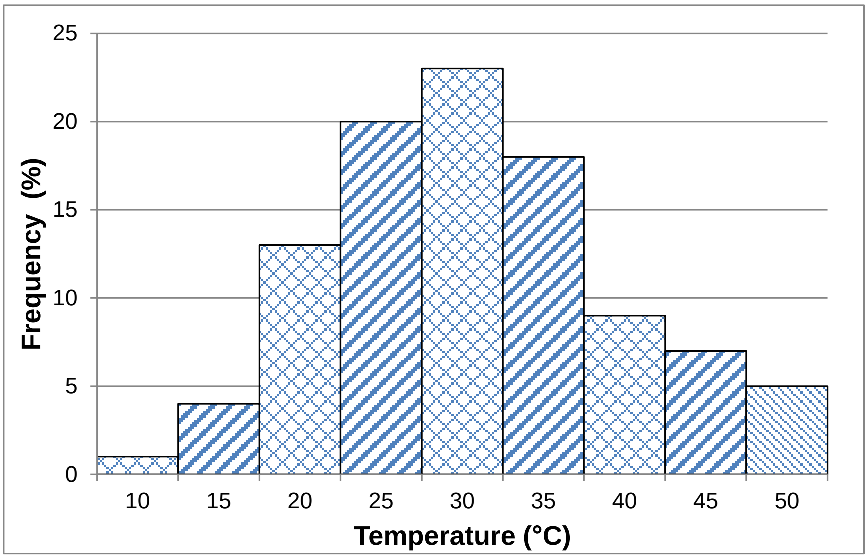
Based on detailed pavement temperature profiles, it is possible to construct the asphalt stiffness distribution for a better understanding of the material modulus changes due to temperature factors. Detailed pavement temperature measurement was conducted in Australia in the 1970s and the results are published in a series of documents (Dickinson 1981). Unfortunately the publications provide analysed data focusing mainly on minimum and maximum pavement temperatures, and with limited cumulative pavement temperature distribution. The recorded data set would be essential for a detailed analysis for determining the equivalent temperature, however the source data for these reports is thought to have been purged in the course of an earlier mainframe computer upgrade (Rickards 2011). The document which summarises the Australian pavement temperature measurement (Dickinson 1981) and the background documents for each Australian capital city (Dickinson 1971; Dickinson 1975; Dunstan 1967) provide a general analysis of the recorded temperatures. Histograms for Brisbane with wide range bins (6 °C) are available only at 50 mm depth for an asphalt pavement for a year and the distribution is summarised in Figure 5.3. For an easier application this distribution was transformed into a histogram with 5 °C bins (Figure 5.4).

Figure 5.3: Yearly pavement temperature distribution in an asphalt pavement, 50 mm depth (6 °C bins)



Source: based on Dickinson (1981).

Figure 5.4: Yearly pavement temperature distribution in an asphalt pavement, 50 mm depth (5 °C bins)



Source: based on Dickinson (1981).

By using the data from Figure 5.4, a series of calculations were performed for the Brisbane region for determining the equivalent temperature, which is 34 °C for n = 0.5 (Table 5.6 and Figure 5.5), 35 °C for n = 0.4 (Table 5.7 and Figure 5.6) and 36 °C for n = 0.3 (Table 5.8 and Figure 5.7). It should be noted that this calculation is valid only for the temperature distribution at 50 mm according to Figure 5.4.

Table 5.6: Calculation of the equivalent temperature, n = 0.5 in strain-stiffness conversion

Duration (%) – based on Dickinson (1981)	1	4	13	20	23	18	9	7	5
θ(°C)	10	15	20	25	30	35	40	45	50
$E\ (\theta_i); 10\ Hz$	17 118	13 862	10 878	8254	6046	4279	2946	2008	1397
$\varepsilon_6(\theta_i)$, n=0.5	130E-6	144E-6	163E-6	187E-6	219E-6	260E-6	313E-6	380E-6	455E-6
$\varepsilon_t\ (10^6)$ - pavement response (1)	30.0	35.3	42.7	52.9	66.7	85.9	111.5	144.0	180.6
$\varepsilon_6(\theta_i)\ (10^6)$ - fatigue performance	130	144	163	187	219	260	313	380	455
$d(\theta_i)(\times 10^6)$ - elementary damage	6.5E-04	8.7E-04	1.2E-03	1.8E-03	2.6E-03	3.9E-03	5.7E-03	7.9E-03	9.8E-03
$d(\theta_i)(\times 10^6)$ - weighted elementary	6.5E-06	3.5E-05	1.6E-04	3.6E-04	6.1E-04	7.1E-04	5.1E-04	5.5E-04	4.9E-04
$d(\theta_i)(\times 10^6)$ - total	3.4E-03								

1. 50 MPa + 300 mm EME structure and Australian standard axle load.

Table 5.7: Calculation of the equivalent temperature, n = 0.4 in strain-stiffness conversion

Duration (%) – based on Dickinson (1981)	1	4	13	20	23	18	9	7	5
θ(°C)	10	15	20	25	30	35	40	45	50
$E\ (\theta_i); 10\ Hz$	17 118	13 862	10 878	8254	6046	4279	2946	2008	1397
$\varepsilon_6(\theta_i)$, n=0.4	130E-6	141E-6	156E-6	174E-6	197E-6	226E-6	263E-6	306E-6	354E-6

$\varepsilon_t\ (10^6)$ - pavement response (1)	30.0	35.3	42.7	52.9	66.7	85.9	111.5	144.0	180.6
$\varepsilon_6(\theta_i)\ (10^6)$ - fatigue performance	130	141	156	174	197	226	263	306	354
$d(\theta_i)(\times 10^6)$ - elementary damage	6.5E-04	9.7E-04	1.5E-03	2.6E-03	4.4E-03	7.9E-03	1.4E-02	2.3E-02	3.4E-02
$d(\theta_i)(\times 10^6)$ - weighted elementary	6.5E-06	3.9E-05	2.0E-04	5.2E-04	1.0E-03	1.4E-03	1.2E-03	1.6E-03	1.7E-03
$d(\theta_i)(\times 10^6)$ - total	7.8E-03								

1. 50 MPa + 300 mm EME structure and Australian standard axle load.

Table 5.8: Calculation of the equivalent temperature, n = 0.3 in strain-stiffness conversion

Duration (%) – based on Dickinson (1981)	**1**	**4**	**13**	**20**	**23**	**18**	**9**	**7**	**5**
θ(°C)	10	15	20	25	30	35	40	45	50
$E\ (\theta_i); 10\ Hz$	17 118	13 862	10 878	8254	6046	4279	2946	2008	1397
$\varepsilon_6(\theta_i)$, n=0.3	130E-6	138E-6	149E-6	162E-6	178E-6	197E-6	220E-6	247E-6	276E-6
$\varepsilon_t\ (10^6)$ - pavement response (1)	30.0	35.3	42.7	52.9	66.7	85.9	111.5	144.0	180.6
$\varepsilon_6(\theta_i)\ (10^6)$ - fatigue performance	130	138	149	162	178	197	220	247	276
$d(\theta_i)(\times 10^6)$ - elementary damage	6.5E-04	1.1E-03	1.9E-03	3.7E-03	7.5E-03	1.6E-02	3.3E-02	6.7E-02	1.2E-01
$d(\theta_i)(\times 10^6)$ - weighted elementary	6.5E-06	4.3E-05	2.5E-04	7.4E-04	1.7E-03	2.8E-03	3.0E-03	4.7E-03	6.0E-03
$d(\theta_i)(\times 10^6)$ - total	1.9E-02								

1. 50 MPa + 300 mm EME structure and Australian standard axle load.

Figure 5.5: Demonstration of the calculated and allowable strain for calculating the equivalent temperature, n = 0.5 in strain-stiffness conversion

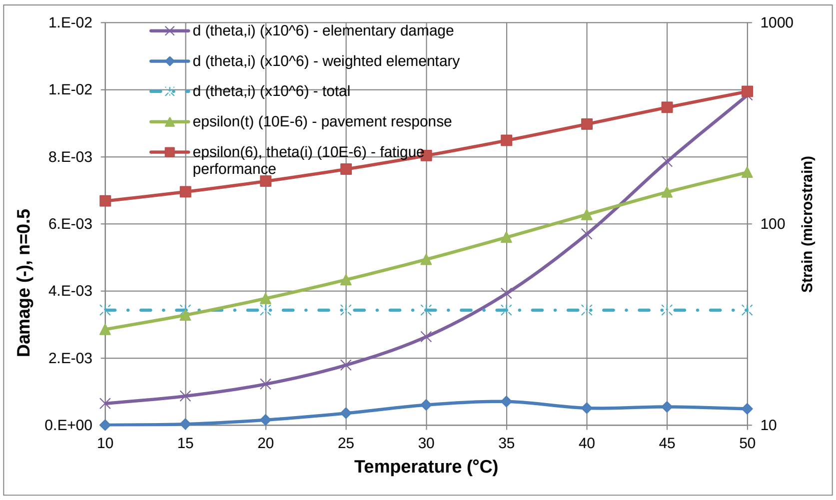


Figure 5.6: Demonstration of the calculated and allowable strain for calculating the equivalent temperature, n = 0.4 in strain-stiffness conversion

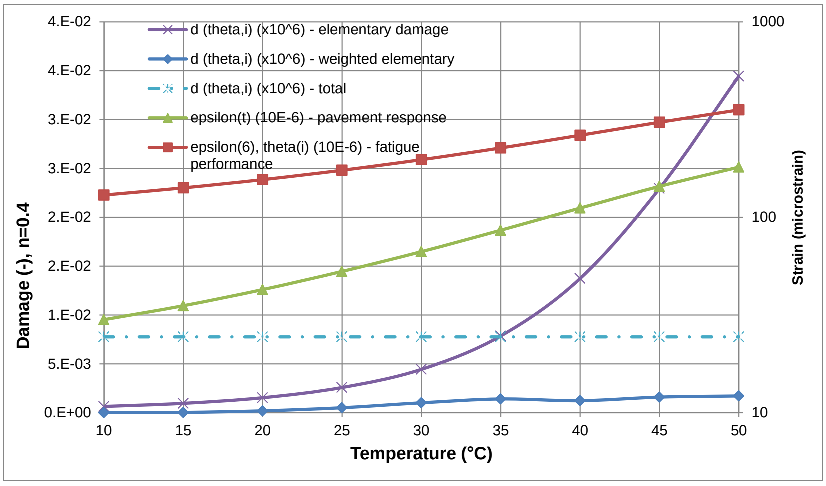
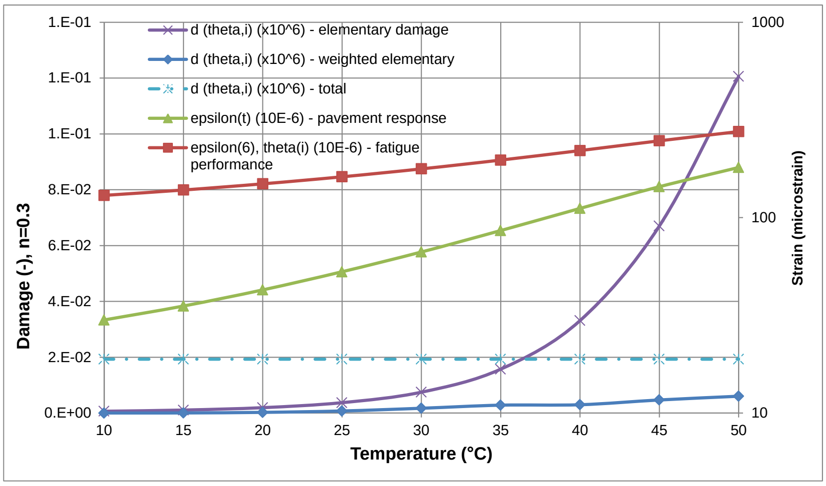


Figure 5.7: Demonstration of the calculated and allowable strain for calculating the equivalent temperature, n = 0.3 in strain-stiffness conversion



Unfortunately there is not enough information to decide:

- What is the real pavement temperature distribution in depth; currently there is information only at 50 mm depth, which most likely does not describe the temperature distribution in deep lift asphalts, where the thickness is above 250 mm – pavement temperature probes will be implemented in QLD pavements to answer this question.
- The fatigue characteristic of the material as a function of the temperature; this is not known and validation will be required in the subsequent years of the project.

Because of the above uncertainties, in the short-term it is suggested that the WMAPT value is used, noting that it most probably does not provide the required answer, and the pavement design should be updated once the above two questions can be answered confidently.

It should also be noted that as part of the demonstration trial, pavement temperature sensors were installed in a deep lift asphalt pavement as outlined in Austroads (forthcoming). Temperature data obtained from this road trial will provide crucial information for an accurate pavement design using EME2 mixes.

6 PAVEMENT DESIGN ACCORDING TO THE GUIDE TO PAVEMENT TECHNOLOGY: PART 2 – TRANSFER FUNCTIONS

6.1 Transfer Function for Subgrade

The limiting strain criterion for the subgrade is given in Equation 19, which limits the vertical compressive strain at the top of the layer (Austroads 2012).

$$N = \left[\frac{9300}{\mu\varepsilon}\right]^7 \qquad (19)$$

where

$\mu\varepsilon$ = the vertical compressive strain at the top of the subgrade (10^{-6} m/m)

N = the allowable number of repetitions of a standard axle at this strain before an unacceptable level of pavement surface deformation develops.

6.2 Transfer Function for Asphalt Fatigue

6.2.1 Background

The fatigue prediction method developed by Shell is well known and it is applied and used by many European countries and Australia. It has been successfully utilised by researchers and the industry for many infrastructure projects. The method used is based on the work reported by Bonneaure et al. (1977) and was developed on 12 typical asphalts from the 1970s. The method was developed based on laboratory testing using a 2-point bending apparatus for trapezoidal specimens. Twelve typical formulations of asphalt mixes were selected for the tests so as to cover a whole range of mixes for road, airfield and hydraulic applications as follows:

- five wearing course mixes comprising two asphaltic concretes, a German mastic asphalt, a British rolled asphalt and a British open-graded mix
- five basecourse mixes, including coarse asphaltic concrete, gravel sand and bitumen stabilised sands
- one asphalt grouting mix used in hydraulic structures and one filler/bitumen asphalt mastic for waterproofing (Bonneaure et al. 1977).

These mixes, vastly different in composition but all standard mixes for road applications in various countries, were studied and analysed. The complex relationships obtained from the laboratory test series formed the basis of the determination of the mix stiffness (S_{mix}) as described in Equation 20.

$$S_{mix} = f(S_{bitumen}; V_{bitumen}; V_{aggregate}) \qquad (20)$$

where

$S_{bitumen}$ = bitumen stiffness (measured or obtained from the van der Poel nomograph)

$V_{bitumen}$ = percentage by volume of the binder in the mix

$V_{aggregate}$ = percentage by volume of the mineral aggregate in the mix.

Bonneaure and his co-authors experienced the limitations of the method (the Shell method) and noted that the prediction model provided would not completely replace the laboratory measurement, but provides paving technologists with a fairly good approach to stress and strain distribution calculations in actual pavements. It should be noted that the determination of S_{bit} also has its limitations, as the van der Poel nomographs are only valid for plain bituminous binders and cannot be directly applied to polymer-modified binders.

The fatigue damage model as currently applied by Austroads (2012) is shown in Equation 21.

$$N = RF\left[\frac{6918 \times (0.856 \times V_b + 1.08)}{E^{0.36} \times \mu\varepsilon}\right]^5 \qquad 21$$

where

N = allowable number of repetitions of the load

$\mu\varepsilon$ = load-induced tensile strain at base of the asphalt (10^{-6} m/m)

V_b = percentage by volume of bitumen in the asphalt (%)

E = asphalt modulus (MPa)

RF = reliability factor for asphalt fatigue (Table 6.1).

Table 6.1: Desired project reliability

Reliability	80%	85%	90%	95%	97.5%
RF value	2.5	2.0	1.5	1.0	0.67

Source: Austroads (2012).

In Equation 21 the bitumen volume in asphalt (V_b) is considered as the total volume of the binder (Shell 1978), referred to as effective binder (V_B) in Section 6.2.3. It should be noted that the Shell manual does not consider any binder absorption as considered in Equation 25, accordingly the Shell fatigue prediction operates with more ideal fatigue properties due to the higher binder contents. As a guideline, when using average asphalt volumetric properties, 0.3% binder absorption could decrease the effective binder volume by 0.7%, which may have significant effect on the predicted fatigue properties when using Equation 21. Further reduction of the binder volume could be considered when using fillers with high Rigden voids, lowering the free binder content (V_F) even further, as calculated in Equation 23.

6.2.2 Typical Volumetric Properties of a Heavy Duty DG20HM Mix and EME2 Mix

It can be seen in Equation 21 that the one volumetric property of the mix, i.e. the total binder volume, and the mix modulus is considered in the fatigue equation.

A typical mix composition is shown for a DG20HM mix in Figure 6.1 and for an EME2 mix in Figure 6.2. The figures show that there is a remarkable difference between the binder volumes between the two mixes. The difference in binder content of the two mixes will have to be examined in the future cost analysis to establish whether the gain in stiffness and fatigue performance balances the required binder cost. The illustration in Figure 6.1 and Figure 6.2 applies an assumed binder density of 1.030 t/m^3, aggregate density of 2.725 t/m^3 and asphalt density of 2.429 t/m^3. The indicated air voids contents of 5.0% for the DG20HM and 3.0% for the EME2 mixes are in line with the mix design requirements of these mixes.

Figure 6.1: Volumetric properties of a DG20HM mix

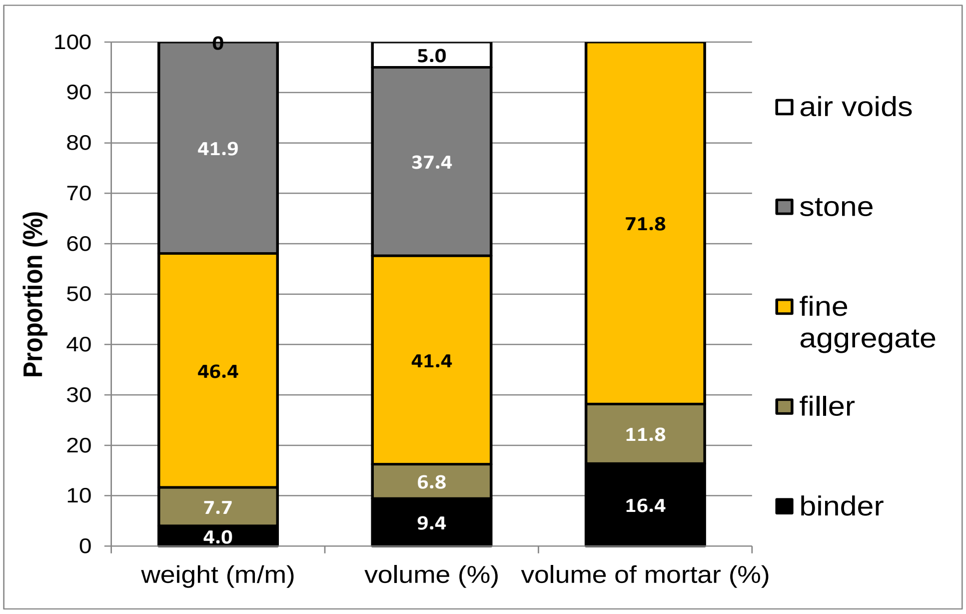
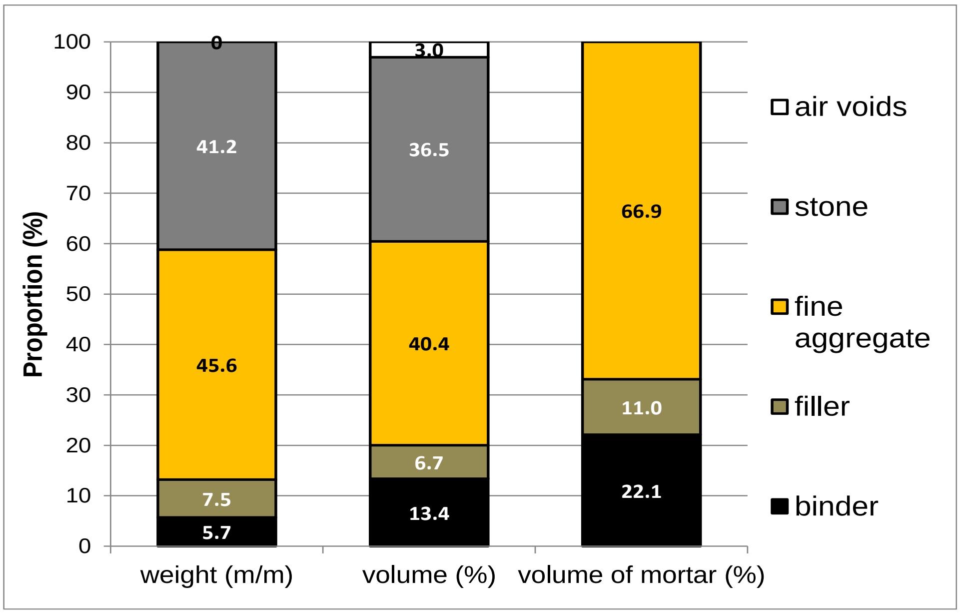


Figure 6.2: Volumetric properties of an EME2



6.2.3 *Sensitivity Analysis – Effective Binder Volume (V_B) and Free Binder Volume (V_f)*

In order to assess the applicability of the binder volume in the asphalt mix, as required in Equation 21, a sensitivity analysis was performed on this property. Austroads (2012) refers to binder volume; however, as discussed in Austroads (2013b), it is important to differentiate between the effective binder volume (V_B) and the free binder volume (V_f). The binder volume value, as an input into Equation 21, influences the fatigue prediction. The influence can be significant for mixes with high binder and filler content, i.e. the EME2 mixes. Also, it is well known that fillers with high Rigden voids are used for the production of asphalt mixes in Australia, which also have an impact on the V_B and V_f values.

For completeness, a sensitivity analysis was performed using Monte Caro Simulation (MCS) to gain an insight into the V_B and V_f values; the calculations were performed according to TMR Test Method Q321 (Department of Transport and Main Roads 2010) as outlined in Equations 22 to 25.

$$f_B = \frac{V_f}{V_B} \quad (22)$$

where

f_B = fixed binder fraction

V_f = fixed binder volume (% by volume of mix), according to Equation 24

V_B = effective binder volume (% by volume of mix), according to Equation 25.

$$V_F = V_B - V_f \quad (23)$$

where

V_F = free binder volume (% by volume of mix), according to Equation 24

V_B = effective binder volume (% by volume of mix), according to Equation 25

V_f = fixed binder volume (% by volume of mix).

$$V_f = \frac{P_{fill} * V * D_C}{\rho_{fill} * (100 - V) * 100} * (100 - B) \quad (24)$$

where

V_f = fixed binder volume (% by volume of mix)

P_{fill} = proportion of the combined filler (% by mass of aggregate and filler)

V = voids in dry compacted filler (%)

D_C = compacted density (t/m^3)

ρ_{fill} = apparent particle density of the combined filler (t/m^3)

B = total binder content (% by mass of mix).

$$V_B = \frac{D_C}{D_B} * (B - b) \qquad (25)$$

where

V_B = effective binder volume (% by volume of mix)

D_C = compacted density (t/m^3)

D_B = density of the binder (t/m^3)

B = total binder content (% by mass of mix)

b = binder absorption of the aggregate (% by mass of mix).

The variables in Equations 22 to 25 were selected according to Table 6.2. It was assumed that the variables have triangular distribution; the values are indicated in Table 6.2.

Table 6.2: Filler properties selected for the MCS analysis

Property	Selected values for sensitivity analysis		
	Minimum	Likeliest	Maximum
Specific gravity of the filler (t/m^3)	2.660	2.760	2.860
Rigden voids (V%)	20.0	30.0	40.0
Binder content in EME2 (m%) (1)	5.6	6.0	6.4

1: Mass % by total mass of asphalt mix.

For this demonstration, the properties of the asphalt mix were kept constant as outlined in Table 6.3 and the simulation was performed accordingly.

Table 6.3: Volumetric properties applied in the MCS simulation for V_B and V_F

Property	Applied value
Aggregate specific gravity (t/m^3)	2.800
Binder specific gravity (t/m^3)	1.030
Maximum density of the mix (t/m^3)	2.560
Compacted density (bulk density) of the mix (t/m^3)	2.485
Binder absorption (%)	0.3
Calculated air voids of the mix (V%)	2.9

The relative distributions are summarised in Figure 6.3 for V_B and in Figure 6.4 for V_F simulations. While the V_B is only a function of the binder content, V_F is influenced by the Rigden voids of the filler; the relative influence of each parameter on the simulation (sensitivity) is summarised in Table 6.4 for V_F.

Figure 6.3: Relative distribution V_B for EME2

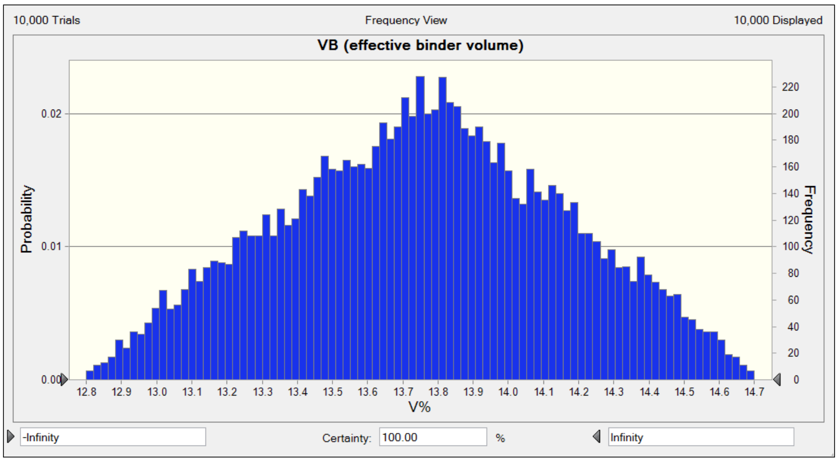


Figure 6.4: Relative distribution of V_F for EME2

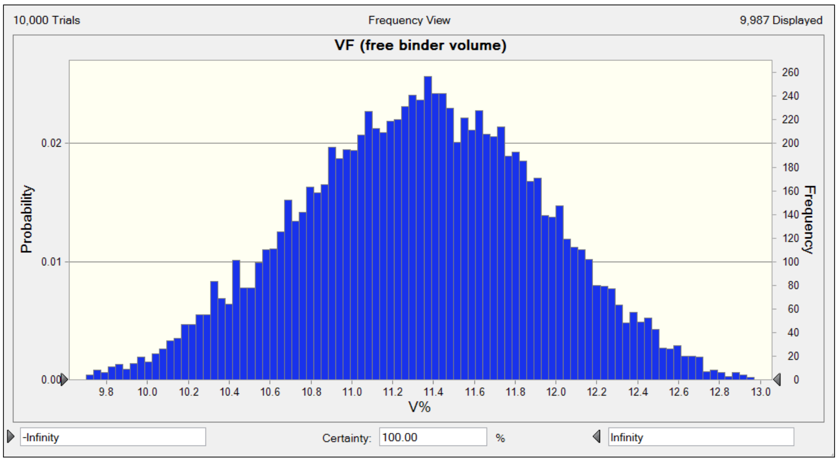


Table 6.4: Contribution to the variance of V_F (sensitivity)

Property	Contribution to variance (%)
Binder content (m%)	32.2
Filler apparent particle density (t/m^3)	0.3
Rigden voids of the filler (V%)	–67.5

Based on the MC simulation, it can be seen that not only the binder and filler content, but the Rigden voids of the filler influences the properties of the mix.

6.2.4 Sensitivity Analysis – Binder Content

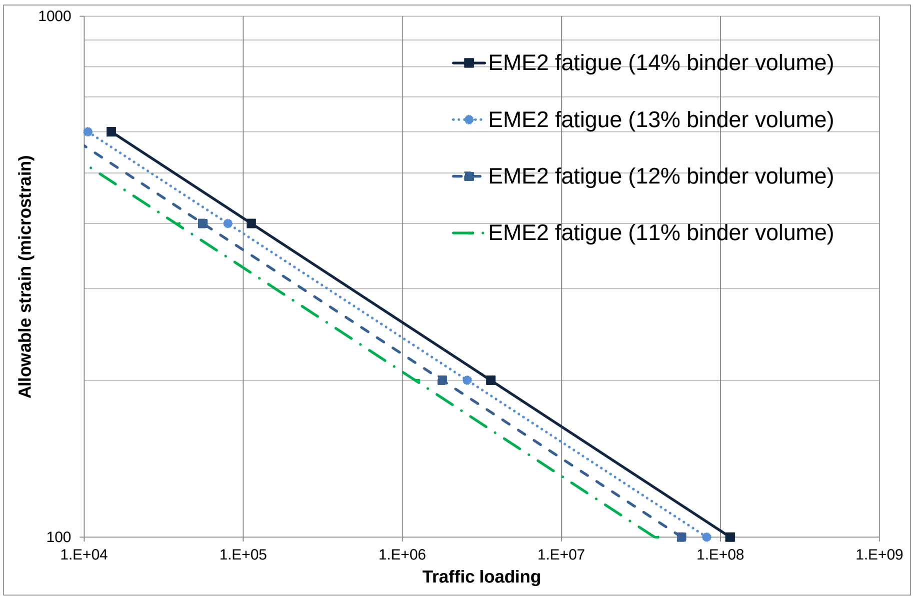
Another sensitivity analysis for the asphalt fatigue transfer function was performed for EME2, which is summarised in Table 6.5 and Figure 6.5. A modulus value of 5400 MPa was selected, which corresponds to the EME2 modulus assumptions (Section 7.3.2) at 32 °C, which is the weighted mean annual pavement temperature (WMAPT) for Brisbane. The binder content was assumed to be 4.9, 5.3, 5.7 and 6.1 % by mass of the total mix weight, which corresponds to 11, 12, 13 and 14 % effective binder volume, assuming the asphalt mix properties in Table 6.3.

Table 6.5: Prediction of the fatigue properties according to the Shell equation as a function of the binder content

RF	B% (by mass of the total mix)	V_b (%)	S_{mix}
1	6.1	14	5400
1	5.7	13	5400
1	5.3	12	5400
1	4.9	11	5400

It should be noted that the binder content affects the modulus of the mix; however, in this demonstration this influence was not considered as there has not been enough evidence collected in Australia to adjust the modulus according to the binder content.

Figure 6.5: Sensitivity analysis of the Austroads asphalt fatigue transfer function



It should be noted that for binder contents below 7% (by mass of the total aggregate), a 1% increase in bitumen content offers the potential of increasing the fatigue performance by 25μm/m (Delorme, Roche & Wendling 2007). Detailed discussion on the fatigue property prediction is provided in Section 8.

7 PAVEMENT DESIGN OF THE DEMONSTRATION TRIAL

7.1 Introduction

7.1.1 Background

It was part of the scope for this project to provide assistance in setting up the trial and undertake long-term monitoring. The pavement structural design provides the basis for establishing layer thicknesses for construction and modelling pavement performance and failure mode.

Boral Asphalt, and bitumen manufacturer SAMI, have developed an EME2 mix suitable for Australian conditions. Raw materials from Australia, including aggregates and binder were supplied and shipped overseas by Boral and SAMI to develop a mix design by Colas in France.

The purpose of the trial is to demonstrate the transfer of established technology and to prove the ability to successfully manufacture and lay EME2 in Australia (production and construction). Following extensive planning and preparation, an EME demonstration trial was constructed on 15 February 2014 at Cullen Avenue West, Eagle Farm. The EME2 mix is designed and tested by Colas France according to European standards, utilising Australian materials and Colas' local subsidiary SAMI Bitumen Technologies' EME binder.

The option to design a pavement to fail within a pre-defined period was discussed by the stakeholders at the progress meetings. This option could provide valuable information about failure modes and validation into shift factors. Due to the complex nature of re-allocating road rehabilitation budgets for an industrial road within a short period of time, it was decided to adopt a 40-years design life. This option also provides valuable information about long-term performance through non-destructive testing and pavement instrumentation, except that symptoms of deterioration and distress may not be observed during the monitoring.

Based on the above the pavement design provided in this section formed the basis for selecting layer thicknesses and overall pavement thickness. Also, the validation of the pavement design through long-term monitoring is one of the major objectives of this study.

7.1.2 Project Delivery Including Production and Paving

The project had a number of stakeholders (Table 7.1) contributing to maximise the knowledge gained from the trial; the combined contribution of all stakeholders led to a seamless production and construction of the trial section.

Table 7.1: Stakeholders that contributed to the EME trial

Organisation	Task or contribution
Boral Asphalt	▪ Manufacture and place asphalt ▪ Provide advice during mix design process ▪ Financial contribution to the construction of the trial
SAMI/Colas	▪ Manufacture and provide binder for the trial (financial contribution) ▪ Design EME2 mix using Australian materials ▪ Coordinate mix design process
Speedie Contractors Pty Ltd South East Profiling Pty Ltd	▪ Cold planing ▪ Material transport
Queensland Transport and Main Roads	▪ Financial contribution towards long-term monitoring through the TMR-ARRB research agreement ▪ Provide assistance for development of specifications

Organisation	Task or contribution
Brisbane City Council	▪ Provide the road section for trial purposes ▪ Supply preliminary design drawings ▪ Perform FWD testing at multiple stages ▪ Complete road construction, such as wearing course, line marking, shoulders, drainage (financial contribution)
Australian Asphalt Pavement Association (AAPA)	▪ Funding the weather station for pavement instrumentation ▪ Disseminating findings to authorities and industry ▪ Organising workshop to present the process and preliminary outcomes of the trial
Environdata	▪ Manufacture and supply the modified weather station to be used with pavement instrumentation; extensive consultation to achieve fit-for-purpose solution
ARRB Group	▪ Perform pavement design ▪ Coordinate site and laboratory testing during design phase ▪ Production and construction monitoring ▪ Installation and long-term operation of pavement instrumentation ▪ Long-term pavement performance monitoring ▪ Develop technical background for specifications and design methods at the national level ▪ Provide assistance in developing project-specific technical standard (PSTS) for TMR ▪ Comprehensive project documentation

Prior to the production and construction trial on 15 February 2014, stakeholders met on several occasions between September 2013 and January 2014. The regular meeting between parties was beneficial in agreeing on the objectives of the trial and execution of the works as the project involved a high number of tasks and multiple partners (Table 7.1).

7.1.3 Trial Objectives

The trial had many objectives to achieve, summarised as follows:

- General requirements for the trial
  - identify location and selection of a suitable test site for the laying of the first demonstration trial
  - define requirements and scope of work for the demonstration trial
  - develop an interim guideline for designing pavement containing EME, using the Austroads pavement design methodology (Austroads 2012)
  - develop a plan to enable the evaluation of the performance of EME against a standard heavy duty DG20HM asphalt material
  - design and construct pavement sections with different base layer thicknesses covered with a standard 30 mm DG10 wearing course (Type 2 asphalt with multigrade binder)
  - manufacture EME2 mix in line with the Colas mix design, which meets the French specifications and tested according to CEN standards
  - provide assistance for developing an inspection test plan (ITP) and conduct laboratory testing using a full range of mix and component materials, against both Australian and French test methods
  - report on the findings of the trial, specifically the manufacture and paving conditions and testing of the binder and asphalt properties.
- Assess feasibility of construction and production of EME using asphalt plants and road construction equipment available in Australia
  - supervise asphalt production control and variability

  - — analyse in situ air voids contents and check whether high level of compaction is achievable
  - — measure material response and compaction curve under Australian compaction equipment
  - — measure compactibility of EME at different layer thicknesses
  - — collect experience with surface characteristic and assess whether temporary trafficking is possible; also check the need for gritting
  - — validate the amount of tack coat to be used on top of EME.

- Provide input into benchmarking and mix design specification
  - — analyse in situ material performance and validation with mix design
  - — input into mix design/benchmarking
  - — develop technical background for specifications and design methods on the national level and provide assistance in developing project-specific technical standard (PSTS) for TMR.

- Pavement design
  - — develop pavement design case studies with EME and check underlying issues in the design procedure
  - — undertake long-term monitoring of in situ pavement performance and develop shift factors for pavement structural design
  - — validate realistic pavement design input values (stiffness and fatigue).

Data collected from the pavement instruments provide input into the validation of strain responses and temperature correction; the information also feeds into many other Austroads and TMR projects.

Mix-design-related issues are not discussed and reported in this study. Mix design, manufacturing and paving, installation of pavement instrumentation and material testing conducted on the trial section and the development of tentative specification limits related to EME2 mixes are discussed in detail in an Austroads report to be published later this year.

Based on the data and observations during production and paving and subsequent testing, it can be concluded that the EME2 mix was manufactured and laid according to the plans and it provided excellent performance in the first four months of trafficking. Short-, medium- and long-term observations on the mix characteristics will be reported in subsequent Austroads reports. It should be noted that this report focuses on the pavement structural performance.

7.2 Existing Pavement Evaluation

7.2.1 Falling Weight Deflectometer Test

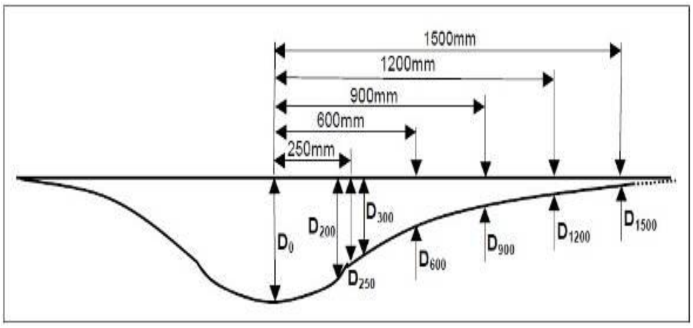
The pavement strength information was collected using Brisbane City Council's (Council) falling weight deflectometer (FWD). The FWD is a trailer-mounted non-destructive pavement testing device that provides pavement response (deflection) to a dynamic loading (Figure 7.1). The unit provides accurate data on the response of the pavement – specifically the surface deflection bowl – to dynamic loads by simulating actual wheel loads in both response and duration. Figure 7.2 shows, in schematic form, the deflection bowl derived from an FWD test.

A dynamic load is generated by the dropping of a mass from a pre-set height onto a 300 mm diameter plate. The magnitude of the load and the pavement response are measured by a load cell and nine geophones. One geophone is located immediately under the load, whilst the others are located at variable offsets from the centre of the load. It is usually recommended that a minimum of three drops per load level at each measuring point are recorded, excluding a small drop for setting the loading plate and checking the consistency of the readings (European Commission, Directorate General Transport 2000). The FWD measurement files contain the following structured data:

- loading for each test point
- measured contact pressure during each measurement process
- pavement surface and air temperature
- measured deflections at geophone offsets of 0, 200, 300, 450, 600, 900, 1200, 1500 and 2100 mm.

Figure 7.1: General view of FWD

Figure 7.2: FWD deflection bowl



Source: Austroads (2008).

Testing was conducted according to the following procedures:

- 40 kN on existing pavement and unbound granular base (cold planed surface)
- 50 kN on asphalt layers (on top of the new base and wearing course)
- FWD data was collected at 10 metre intervals, staggered in each wheel path in each traffic lane and on the unsealed shoulder
- there was no correction made to account for testing temperature
- seasonal effects were not considered in the analysis.

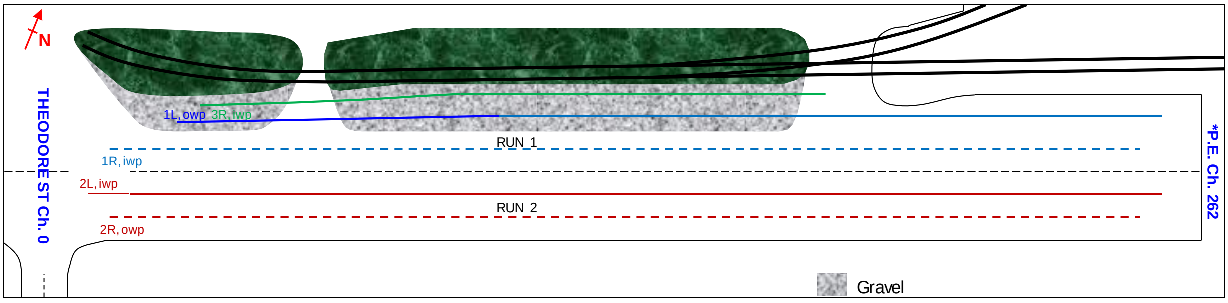
7.2.2 Delineation of Homogenous Sub-sections and Calculation of Characteristic Deflections

Usually the road is divided into homogeneous sub-sections and the structural adequacy of each pavement sub-section is estimated in accordance with the *Guide to pavement technology–part 5: pavement evaluation and treatment design* (Austroads 2011). However, due to the nature of the project and the short length of the road section, these calculations were not performed and only visual sub-sectioning was applied.

A general view of the trial site and the locality plan with the testing locations are outlined in Figure 7.3 and Figure 7.4. The maximum deflections are summarised in Figure 7.5.

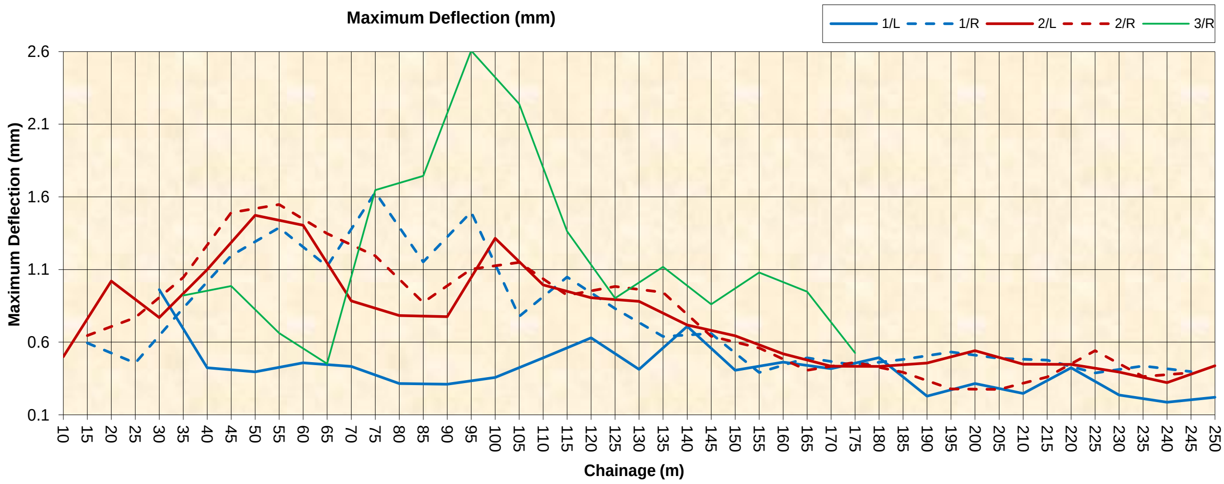
Figure 7.3: General view of the trial site – Cullen Avenue West, Eagle Farm

Figure 7.4: Locality plan and testing locations on the rehabilitated road



Source: Brisbane City Council.

Figure 7.5: Maximum deflection (D0) values on Cullen Avenue West



Source: Brisbane City Council.

1L and 1R are referenced as the northern traffic lane (N), whereas 2L and 2R are referenced as the southern traffic lane (S) in this report. The characteristic deflection was calculated and the back-calculated moduli were used in subsequent analysis.

7.2.3 Coring and Dynamic Cone Penetrometer (DCP) Testing

The asphalt layer thickness was determined at six locations based on cores extracted from the pavement, with the dry coring method being used. Following the coring, DCP testing was performed at three locations. The objective of performing DCP testing was to provide accurate determination of the thickness of the unbound granular base layer(s) and to collect input for the subgrade CBR value, which was used later as a seed value in the back-calculation.

The asphalt cores extracted from the pavement are shown in Figure 7.6 to Figure 7.12 a summary of the thicknesses and the locations of the coring are summarised in Table 7.2. The DCP values are summarised in Figure 7.13; it should be noted that the DCP test results were used for determining the layer thicknesses.

Figure 7.6: Core #1

Figure 7.7: Core #2

Figure 7.8: Core #3

Figure 7.9: Core #4

Figure 7.10: Core

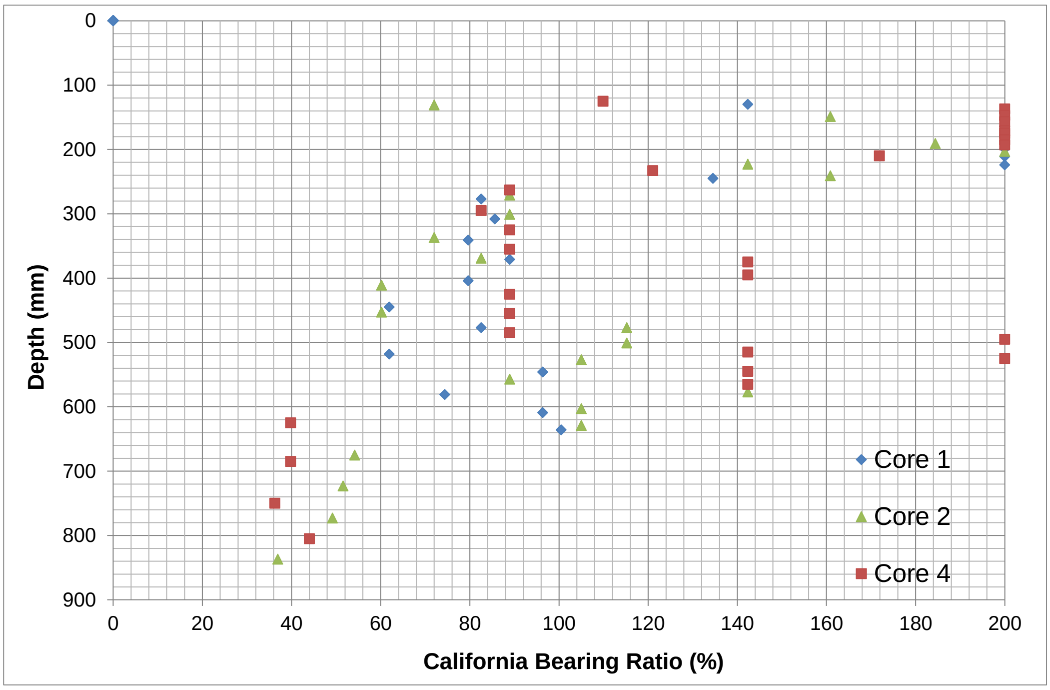
Figure 7.11: Borehole #5

Figure 7.12: Core #6

Table 7.2: Locations of coring and DCP testing and thickness of each pavement layer

Core number	Chainage (m)	Distance from southside kerb (m)	DCP	Thickness (mm)			
				Asphalt	Upper subbase	Lower subbase	Subbase total
1	214	3.0	Yes	110	150	380	530
2	165	5.0	Yes	95	160	390	550
3	150	1.6	N/A	100	N/A	N/A	N/A
4	126	4.5	Yes	100	150	350	500
5	84	1.0	N/A	110	N/A	N/A	N/A
6	45	1.9	N/A	170	N/A	N/A	N/A
Average				**114**	**153**	**373**	**527**

Figure 7.13: Estimated CBR values



7.2.4 FWD Data Analysis and Back-calculated Moduli

Initial uniformity assessment

To provide an overview of the pavement condition prior to back-calculation, an initial check of the surface modulus was performed using the FWD data. The surface modulus was calculated according to Equation 26 (Ullidtz 1998).

$$E_{surface} = \frac{2 \times p \times r \times (1 - \nu^2)}{d_0} \quad (26)$$

where

$E_{surface}$ = surface modulus (MPa)

p = contact stress (kPa)

r = radius of the loading plate (m)

ν = Poisson's ratio; a constant value of 0.35 was used in these calculations

d_0 = central deflection.

Figure 7.14 shows that the southern traffic lane had a weaker bearing capacity between chainages 0 to 150 m compared to the northern lane which was a result of the weak subgrade; this conclusion is based on the equivalent subgrade modulus, which is shown in Figure 7.15. The weak subgrade could be a direct result of the road geometry and drainage conditions, as the road section has a one-sided cross-fall towards the southern side where the water should be collected and channelled into the drainage system. However, water ponds on this road section and may penetrate into the deeper layers through the cracked asphalt layer(s). This theory is supported by the fact that the unbound granular layer was found to be wet underneath the asphalt layer (Figure 7.11).

It was important to identify and delineate the traffic lanes with different subgrade properties as it influences the overall pavement performance as is discussed in Section 7.4.

Figure 7.14: Surface modulus of the traffic lanes and the shoulder

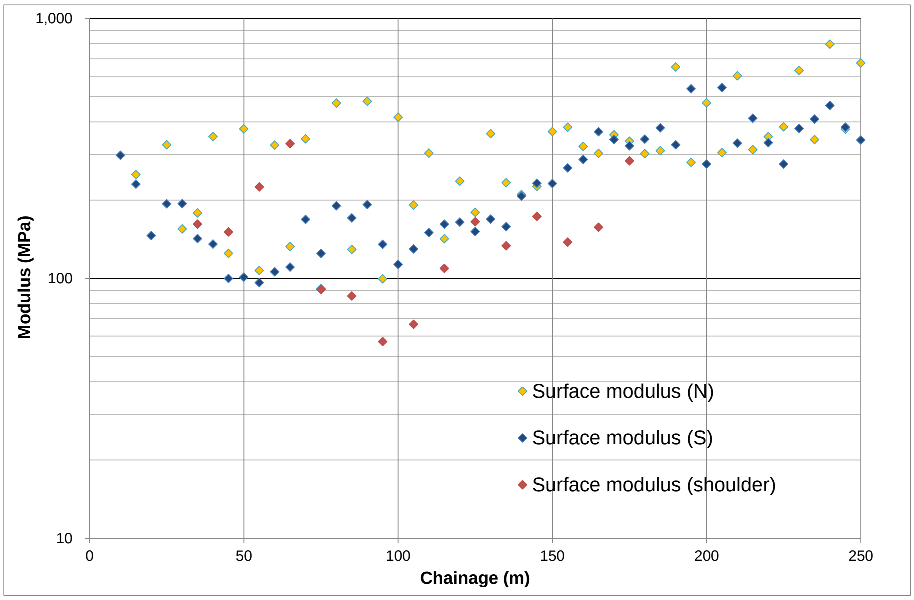
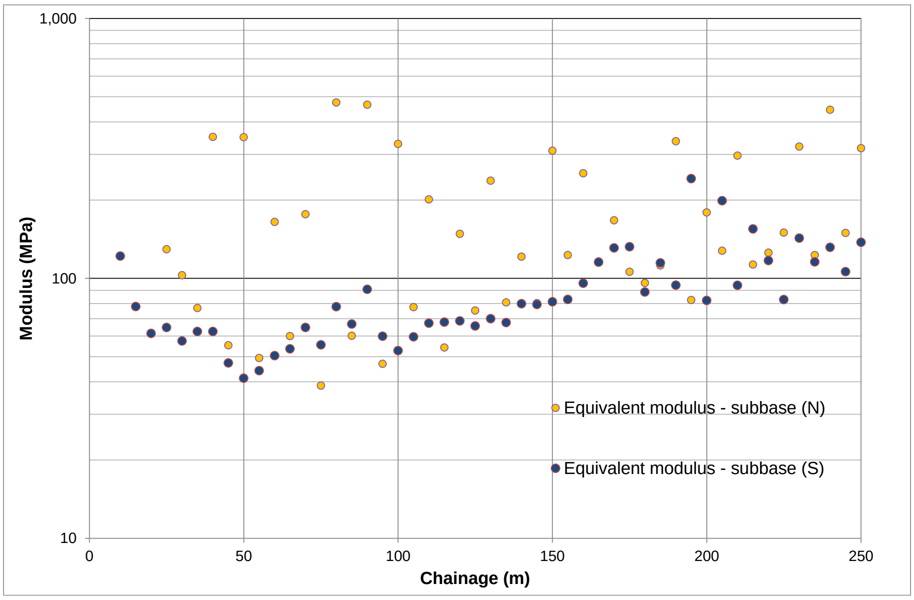


Figure 7.15: Equivalent modulus of the subgrade on the sealed traffic lanes



Back-calculated moduli

In the back-calculation process, the measured FWD deflection bowls were used to estimate the in situ elastic moduli for each pavement layer. It is particularly important to the accuracy of this procedure to know the different layer thicknesses. Incorrect pavement configuration input could lead to incorrect outcomes.

The process of back-calculating layer stiffness comprises of gathering information on the existing pavement structure and adoption of a pavement model for the calculation. Information on the layer type and quality is used to estimate likely values for the stiffness of the various pavement layers. An indication is obtained of the quality and type of the subgrade and the pavement layers, as well as of the thickness of the various pavement layers. In the calculations the design pavement thicknesses (obtained from the pavement design report) were adopted.

Only the second drop from the FWD test was used for back-calculation, with an applied load of 40 kN. Back-calculation was performed using the ELMOD 6 computer program. The back-calculation was performed by ARRB on the FWD test results provided by the Council in October 2013.

Assumptions applying to the back-calculation were:

- no layer thickness variance applies along the pavement
- no temperature and moisture adjustment applies to the test results
- stress dependency of the granular materials was not taken into consideration.

The existing pavement structure configuration model was based on three different layers:

- asphalt
- unbound granular base and subbase
- subgrade.

TMR pavement rehabilitation guidelines provide a procedure for back-calculation of the pavement layer properties derived from FWD measurement (Department of Transport and Main Roads 2007). According to TMR the following conditions apply:

- Pavement rehabilitation methodology practised in TMR differs to that of the Austroads method. The method takes into account the strength of the existing layers (bound or granular) but limits the maximum modulus values based on the actual field condition (significant cracking, minor cracking and so on) and the back-analysed data from deflection results.
- The pavement design package CIRCLY is then used to evaluate the critical strains and the allowable standard axle repetitions of the existing and new pavement layers.
- As the general mechanistic procedure (GMP) relies on the back-analysed modulus values, it is important to understand the strength and weakness of the technique of back-analysing.

TMR pavement rehabilitation guidelines also limit the maximum moduli values derived from back-calculation for granular flexible pavements. These limited values are summarised in Table 7.9. It should be noted that the pavement calculations utilised a value of 250 MPa, instead of the allowable maximum of 150 MPa.

The adopted thicknesses are shown in Table 7.3.

Table 7.3: Thickness of the existing pavement

Layer	Pavement files	DCP testing
	(mm)	(mm)
Asphalt	130	100
Unbound granular	Unknown	500
Subgrade	Unknown	Infinite

The average, standard deviation and minimum back-calculated modulus values are shown in Table 7.4 and Table 7.5, and the values are visualised for the sealed traffic lanes in Figure 7.16 and Figure 7.17.

Table 7.4: Sealed traffic lanes, back-calculated modulus

Lane	Chainage (m)	Property	Asphalt (MPa)	Subbase (MPa)	Subgrade (MPa)
Northern lane	0-150	Average	2311	189	96
		Standard deviation	2415	149	75
		Minimum	700	80	6
	150-250	Average	6663	206	147
		Standard deviation	2767	130	52
		Minimum	983	80	54
Southern lane	0-150	Average	2606	84	30
		Standard deviation	1748	13	23
		Minimum	700	80	7
	150-250	Average	7497	125	144
		Standard deviation	2166	44	48
		Minimum	3849	80	68

Figure 7.16: Back-calculated modulus, northern lane

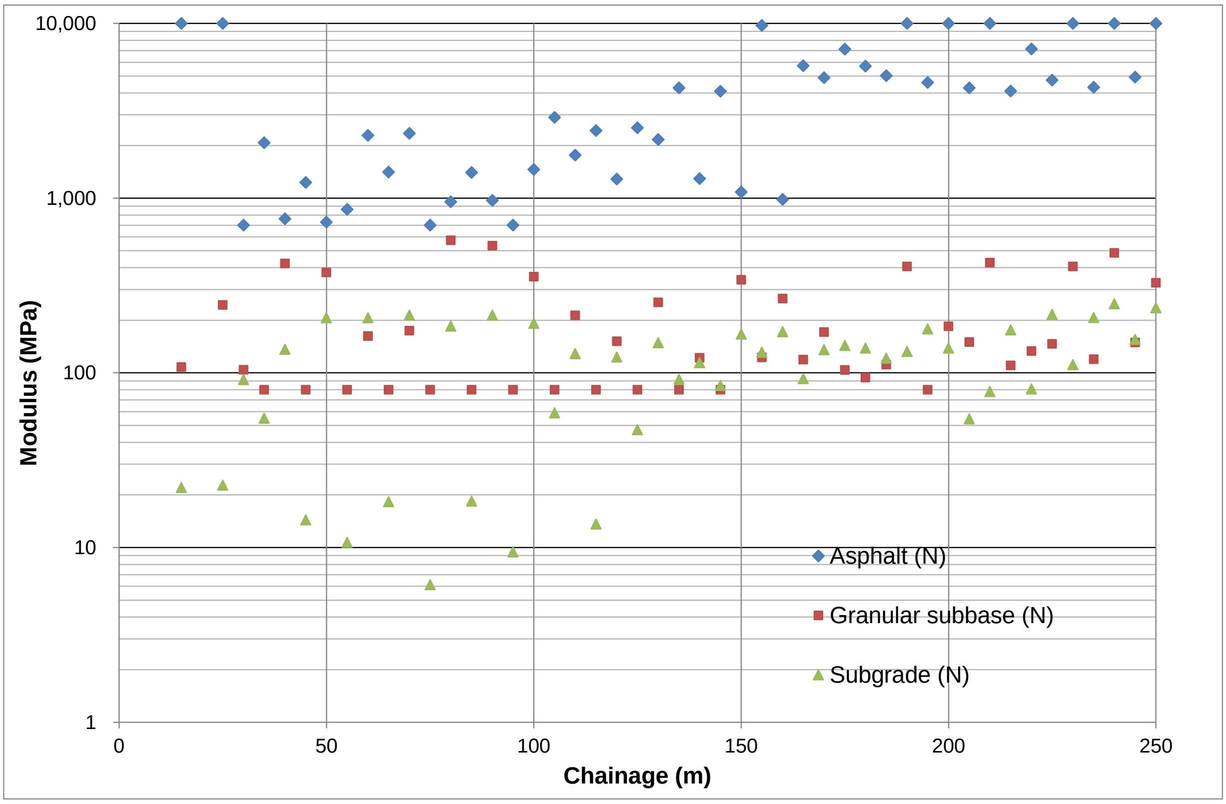
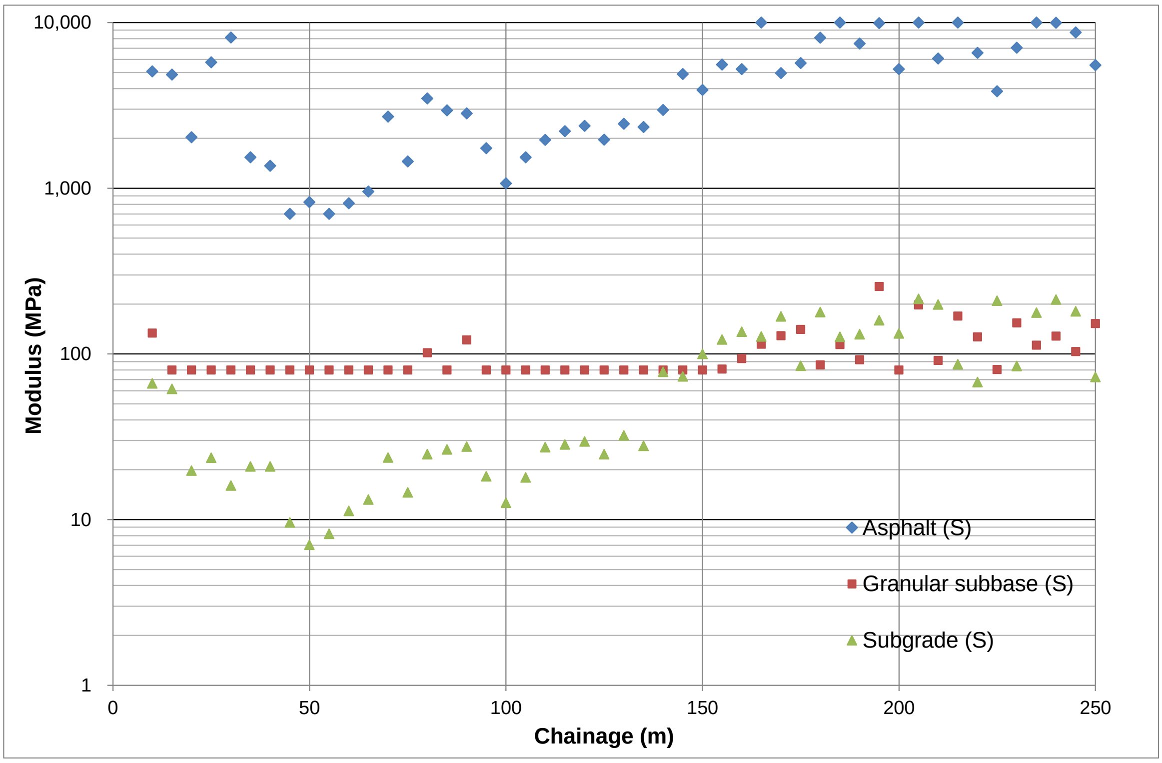


Figure 7.17: Back-calculated modulus, southern lane



Adequate performance assessment of the shoulder was necessary, as with the new layout this area became the eastbound traffic lane and the layer's moduli of this part of the future pavement were also determined.

The adopted model for back-calculation on the shoulder was a two-layer system; an unbound granular upper layer of 150 mm supported by the semi-infinite subgrade. Back-calculated moduli are showed in Table 7.5.

Table 7.5: Shoulder, back-calculated moduli

Lane	Chainage (m)	Property	Upper layer (MPa)	Subgrade (MPa)
Shoulder	35-175	Average	895	43
		Standard deviation	277	34
		Minimum	188	9

7.3 Pavement Design According to the Australian and French Pavement Design Methods

In France the structural pavement design should be performed in line with the *French design manual for pavement structures* (Laboratoire Central des Ponts et Chausées 1997), the *Catalogue of typical new pavement structures* (LCPC-Setra 1998) or standard NFP 98-086 (2011) *Road pavement structural design–Application to new pavements.*

Detailed background on the design process is provided in Section 5. For this study the general mechanistic procedure outlined in NF P 98-086 (2011) was used.

It should be noted that the current Austroads pavement design method is not suitable for designing pavements containing EME2 layers. The existing Austroads framework was used where possible, assuming the unknown input parameters from available information. It does not form a rigorous, validated thickness design but provides results for comparison purposes.

7.3.1 Design Assumptions for the New Trial Pavement

The pavement design package CIRCLY was used to evaluate the critical strains and the allowable standard axle repetitions. The following assumptions were adopted to perform the calculations (Table 7.6):

- number of heavy vehicles per year 10 000 (27 per day)
- heavy vehicle operating speed is 60 km/h
- the pavement is consistent within sub-sections
- there is restriction with respect to the increase in surface levels
- the subgrade characteristics are the same across the full width of each section of existing pavement lane.

Homogenous sections were formed based on the required number of trial sections and not based on statistical analysis of the existing pavement performance.

Table 7.6: Design parameters

Traffic parameter	Value
Design period	P = 40
One-way traffic volumes (number of heavy vehicles per year)	NHV = 10 000
Direction factor	DF = 1.0
Heavy vehicle growth rate (%)	R= 2%
Lane distribution factor (LDF)	LDF = 1.00
Number of heavy vehicle axle groups per heavy vehicle (presumptive)	N_{HVAG} / HV = 2.8
Equivalent standard axle per heavy vehicle axle group (presumptive)	ESA / HVAG = 0.9
Standard axle repetition per equivalent standard axle – asphalt (presumptive)	SAR5 / ESA = 1.1
Standard axle repetition per equivalent standard axle – subgrade (presumptive)	SAR7 / ESA = 1.6

Based on the values summarised in Table 7.6, the design traffic values outlined in Table 7.7 were used in the pavement design.

Table 7.7: Design traffic values

Number of heavy vehicles (per year)	NDT	DESA	DSAR5	DSAR7
10 000	1.69 E+06	1.47 E+06	1.66 E+06	2.41 E+06

Also, the following assumptions were made for the pavement design:

- design thicknesses did not take into account the construction tolerance
- the pavement layer interface is rough between the pavement layers
- in this calculation 85% reliability level was adopted, in line with Austroads (2012) (other roads, lane AADT < 500).

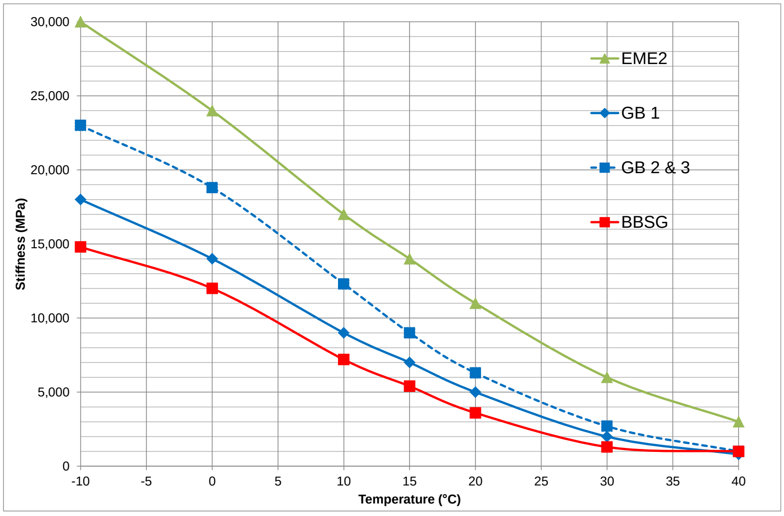
CIRCLY version 5.0 software from MinCad Systems was used to calculate the critical strains.

7.3.2 Thickness Evaluation According to the Austroads Method

The WMAPT for Brisbane is 32 °C, and the stiffness values were selected accordingly. At the time the pavement structural design was performed, laboratory studies, conducted by ARRB and industry partners, for characterising EME were underway in Australia. Due to the limited information available at that time, modulus values were adopted from different sources.

The design modulus value for EME2 mix was adopted from Laboratoire Central des Ponts et Chaussées (1997), which is in line with the material library of the software package ALIZÉ. Although it was known that stiffness is measured by using a different test method in France (2-point bending flexural stiffness) than in Australia (indirect stiffness test – ITS), a value of 5400 MPa was selected for 32 °C (Figure 7.18).

Figure 7.18: Temperature dependency of different asphalt types (complex modulus at 10 Hz, 2-point bending)



Source: Laboratoire Central des Ponts et Chausées (1997).

The design input parameters and assumptions for the asphalt mixes are shown in Table 7.8.

Table 7.8: Design input parameters for the Australian design procedure

Asphalt type	Design stiffness (MPa)	V_b	k	Exponent	Poisson value
DG20HM (C600)	2360	10.5	0.004253	5	0.4
EME2	5400 (1)	14.0 (3)	0.004096	5	0.4
Type 2 (BCC)	1800 (2)	10.0	0.004489	5	0.4

1. Refer to Figure 7.18.

2. There is no published data for type 2 asphalt stiffness with M1000/320 binder; the closest asphalt type is type 3, C320 material which has E = 1900 MPa, V_b = 10%, at v = 50 km/h.

3. Assumptions: binder content 6.4 mass %, binder absorption 0.3%, mix air voids 4.1%, maximum density 2.450 t/m^3; at the time of the pavement design the sensitivity analysis of the binder volume (Section 6.2.3) was not performed and the mix design properties, including design binder content, were not provided.

In Table 7.10 and Table 7.11 different design scenarios were adopted for the existing pavement properties, as the design subgrade and existing pavement properties varied along the road section.

There was uncertainty about the design subgrade modulus, therefore three scenarios are calculated (i.e. 50, 100 and 150 MPa).

Table 7.9: Utilised vertical moduli for the subgrade underneath the asphalt base layer

Vertical moduli	Adopted Poisson's ratio
50 MPa (CBR 5%)	0.45, anisotropic

Vertical moduli	Adopted Poisson's ratio
100 MPa (CBR 10%)	0.45, anisotropic
150 MPa (CBR 15%)	0.45, anisotropic

Table 7.10: Initial design thickness of EME2 pavement for different scenarios

Layer	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)
Pavement reference	**1**		**2**		**3**	
Asphalt type 2	30	1800	30	1800	30	1800
Asphalt EME2	100	5400	100	5400	100	5400
Subgrade	infinite	50	infinite	100	infinite	150

Note: The design thicknesses indicated in this table do not contain the construction tolerances as required in the TMR Supplement.

Table 7.11: Initial design thickness of DG20HM pavement for different scenarios

Layer	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)	Thickness (mm)	Modulus (MPa)
Pavement reference	**4**		**5**		**6**	
Asphalt type 2	30	1800	30	1800	30	1800
Asphalt DG20HM	100	2360	100	2360	100	2360
Subgrade	infinite	50	infinite	100	infinite	150

Note: The design thicknesses indicated in this table do not contain the construction tolerances as required in the TMR Supplement.

The results of the calculations are summarised in Table 7.12 (EME2) and in Table 7.13 (DG20HM) for the Austroads design method. The optimised pavement layer thicknesses and thickness reductions are highlighted in Table 7.14.

Table 7.12: Pavement thickness design according to the Australian method, EME2

AGPT Part2 / CIRCLY	Asphalt base layer		Subgrade		Design controlled by	Optimised base layer thickness (mm)
EME2 / E = 5400	**Calculated strain (microstrain)**	**CDF**	**Calculated strain (microstrain)**	**CDF**		
CBR 5%	270.0	1.01	872.0	0.15	EME2 layer	101
CBR 10%	220.0	0.36	641.0	0.02	EME2 layer	80
CBR 15%	193.0	0.18	530.0	0.01	EME2 layer	67

Table 7.13: Pavement thickness design according to the Australian method, DG20HM

AGPT Part2 / CIRCLY	Asphalt base layer		Subgrade		Design controlled by	Optimised base layer thickness (mm)
DG 20 HM / E =2360	**Calculated strain (microstrain)**	**CDF**	**Calculated strain (microstrain)**	**CDF**		
CBR 5%	448.0	10.50	1250.0	1.82	DG 20 HM layer	156
CBR 10%	357.0	3.36	907.0	0.20	DG 20 HM layer	129
CBR 15%	307.0	1.59	745.0	0.05	DG 20 HM layer	110

Table 7.14: Pavement thickness reduction according to the Australian method

Subgrade property	EME2 thickness (mm)	DG 20 HM thickness (mm)	Thickness reduction (%)
CBR 5%	110	160	31
CBR 10%	80	130	38
CBR 15%	70	110	36

Note: Wearing course: 30 mm Type 2 asphalt.

7.3.3 Thickness Evaluation According to the French Method

The outcomes of the French design method are summarised in Table 7.15 (EME2) and Table 7.16 (DG20HM). The optimised pavement layer thicknesses and thickness reduction according to the French method are summarised in Table 7.17. It should be noted that the design is controlled in some cases by the subgrade strain criteria.

It should be noted that Table 7.16 and Table 7.17 refer to GB3, which is a heavy duty asphalt type in France and it is considered to be similar to the DG20HM mix. The CIRCLY calculations and allowable strain calculations are summarised in Appendix B and Appendix C.

Table 7.15: Pavement thickness design according to the French method, EME2

NF P 98-086 / ALIZE	Asphalt base layer		Subgrade		Optimised base layer thickness (mm)
EME2 / E = 5400	Permissible strain (microstrain)	Calculated strain (microstrain)	Permissible strain (microstrain)	Calculated strain (microstrain)	
EV2 50 MPa	255.8	256.3	974.9	929.5	100
EV2 100 MPa	264.2	208.9	974.9	683.8	80
EV2 150 MPa	281.4	182.5	974.9	566.4	70

Table 7.16: Pavement thickness design according to the French method, GB3

NF P 98-086 / ALIZE	Asphalt base layer		Subgrade		Optimised base layer thickness (mm)
GB3 / E =2360	Permissible strain (microstrain)	Calculated strain (microstrain)	Permissible strain (microstrain)	Calculated strain (microstrain)	
EV2 50 MPa	288.8	425.1	974.9	1336.2	150
EV2 100 MPa	298.3	336.4	974.9	974.6	120
EV2 150 MPa	317.7	287.4	974.9	801.8	100

Table 7.17: Pavement thickness reduction according to the French method

Support category	Surface modulus, EV2 (MPa)	EME2 thickness (mm)	GB3 thickness (mm)	Thickness reduction (%)
PF2	50	100	150	33
PF2qs	100	80	120	33
PF3	150	70	100	30

Note: Wearing course: 30 mm Type 2 asphalt.

7.3.4 Discussion of the Pavement Designs

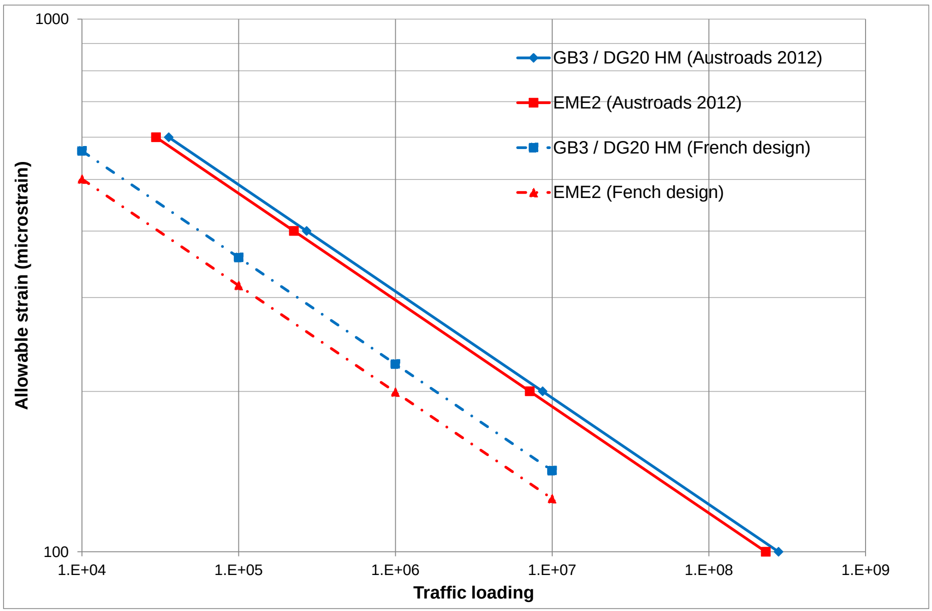
For the Australian design, the fatigue equation was used as described in Austroads (2012). A binder volume of 14% was assumed which was derived from an assumed asphalt volumetric property of binder content of 6.4% by mass, binder absorption of 0.3%, mix air voids of 4.1% and a maximum density of 2.450 t/m^3.

For the calculation of the allowable strains, the following should be considered and noted:

- It is theoretically incorrect to apply the fatigue equation as described in Austroads (2012) directly to EME2 mixes. EME2 is based on a fully performance-based mix design and the fatigue equations in Austroads (2012) are not validated for EME2. However, it was decided as an interim solution to utilise this approach, while the information and conclusions derived from running Austroads and TMR projects would be available. In the longer term, the performance-based mix design of EME2 should be directly connected to the mechanistic pavement design procedure as is outlined in NF P 98-086 (2011).
- For the French design, the fatigue properties were calculated according to NF P 98-086 (2011). The methodology of calculation is explained in detail in Section 5. It should be noted that NF P 98-086 (2011) is available only in the French language.
- For the pavement design calculations, the minimum mix performance requirements were taken into account, i.e. 14 000 MPa stiffness at 15 °C, 10 Hz and 130 microstrain at 10 °C, 25 Hz. By applying the real mix design properties within the ranges as allowed in France (Table 5.1), it may be possible to provide further optimisation of the pavement configuration.

The calculated allowable strains for the Cullen Avenue West trial are summarised in Figure 7.19, where the GB3 (DG20HM) asphalt has a higher tolerable strain. It should be noted that the fatigue properties in Figure 7.19 are predicted (calculated) values, based on the Shell equation, and mix-specific laboratory fatigue testing may show different results.

Figure 7.19: Comparison of the allowable strain for the QLD trial using the French and Australian transfer functions



Note: This chart is valid only for the Cullen Avenue West pavement design.

It should be noted that the Australian and French pavement design methods cannot be directly compared; although they utilise the same methodology, the amplitude of traffic loadings, the shift factors, the reliability factors and the fatigue properties are calculated and determined in very different ways. The differences are discussed in detail in Section 5, and the major differences for the above design procedure are summarised in Table 7.18. It should be noted that the Australian and the French method uses the same method for traffic growth calculation; however, due to minor differences in rounding there is a difference of 8 757 vehicles over a 40 years design period which is considered negligible.

Table 7.18: Comparison of the French and Australian pavement design input for the Cullen Avenue demonstration trial

Input	French method	Australian method
Number of vehicles	595 262	604 019
NDT	N/A	1 690 000
$NE_{pavement}$	178 578	N/A
ESA	N/A	1 470 000
Calculated pavement responses (i.e. strains)	Similar	
Fatigue equations	Different	
Pavement design	Very similar	

Based on the above pavement design procedure, combined with considerations on constructability and construction sequences, it was decided to adopt CBR5% for the entire road section and the following pavement structures were constructed:

- 100 mm thick EME2 base layer, constructed in one paving run
- 150 mm thick EME2 base layer, constructed in one paving run
- 150 mm thick dense graded asphalt for heavy duty application (DG20HM), constructed in two paving runs, as the control section.

Cold planing at various depths was performed before construction of the base layers and the top of the base layers were paved in level. After a short period of monitoring of the base, the wearing course, a 10 mm nominal size dense graded asphalt with multigrade bitumen, was placed at a uniform thickness of 30 mm.

The Austroads report (forthcoming) provides detailed information on the following:

- construction, profiling, paving sequences, rolling and surface gritting
- construction quality assurance, including material production, in situ density testing, temperature monitoring and sand patch testing
- post-construction testing, including coring, British Pendulum Testing (BPT), SCRIM testing
- pavement instrumentation, including strain gauges, pavement temperature sensors and installation of the weather station
- construction of the wearing course, including production control, temperature measurements and validation of the residual binder for the tack coat between the base layer and wearing course.

7.4 Assessment of the Pavement Before and After Construction

7.4.1 Falling Weight Deflectometer (FWD) Testing Program

The FWD testing was undertaken at different stages of the construction. Initial testing was performed on the top of the existing granular layer, before placement of the asphalt base. Then FWD testing was completed after the asphalt base construction before and after the wearing course was laid.

The first FWD testing took place after the completion of the surface profiling (Figure 7.20); it should be noted that any subsequent testing on the pavement layers was carried out at the same offset from the kerb, for each FWD testing line as follows:

Traffic lanes:

- Line 1: 3.3 metres from the kerb (denoted as 2R)
- Line 2: 4.3 metres from the kerb (denoted as 2L)
- Line 3: 6.6 metres from the kerb (denoted as 1R)
- Line 4: 7.8 metres from the kerb (denoted as 1L).

Testing frequency is 10 metres per line:

- Line 1 and Line 2 are 5 metres staggered
- Line 3 and Line 4 are 5 metres staggered.

Parking lanes (testing frequency is 10 metres per line)

- Line 5: 0.6 metres from the kerb (denoted as 4R)
- Line 6: 10.5 metres from the kerb (denoted as 3L).

Figure 7.20: Falling weight deflectometer testing on the profiled surface

Following construction of the base, FWD test was carried out on the top of the DG20HM and EME2 layers on 21 February 2014; FWD test was performed on top of the wearing course on 13 May 2014.

7.4.2 Temperature Correction

Variations in temperature can cause significant changes in the stiffness of flexible pavements containing asphalt and, therefore, the measured pavement deflection. The temperature of the asphalt was recorded by the weather station, implemented as part of this project.

Conditions varied during the deflection survey and therefore the deflections had to be adjusted accordingly. It should be noted that no deflection testing should be conducted when the asphalt surface temperature exceeds 60 °C, as the results may become unreliable. Some references suggest that deflection testing should not be conducted if the pavement temperature exceeds 30 °C (European Commission, Directorate General Transport 2000).

The average working temperature is referred to as the weighted mean annual pavement temperature (WMAPT). The WMAPT value for the Brisbane region is 32 ºC.

The deflection test results (D_0) were multiplied by adjustment factors to correct for the differences between the measured field temperatures and the WMAPT. The representative adjustment factors (Table 7.19) were calculated according to Equation 27 (Austroads 2008).

$$AF = 1 + a * (TAF - 1)^2 + b * (TAF - 1) \tag{27}$$

where

AF = deflection at the WMAPT divided by the deflection at the measurement temperature

TAF = WMAPT divided by the asphalt temperature during deflection measurements

a, b = regression coefficients.

Table 7.19: Regression coefficients for calculating the temperature adjustment factors

Asphalt thickness (mm)	T_{meas} < 25 °C		T_{meas} > 25 °C	
	a	b	a	b
100	-0.0613	0.2106	-0.2045	0.2678
150	-0.0622	0.271	-0.2205	0.3684

Source: Austroads (2008).

The measured pavement temperatures are summarised in Table 7.20 and Table 7.21 and they are visualised in Figure 7.21.

Table 7.20: Temperature for each FWD test line on 21 February 2014

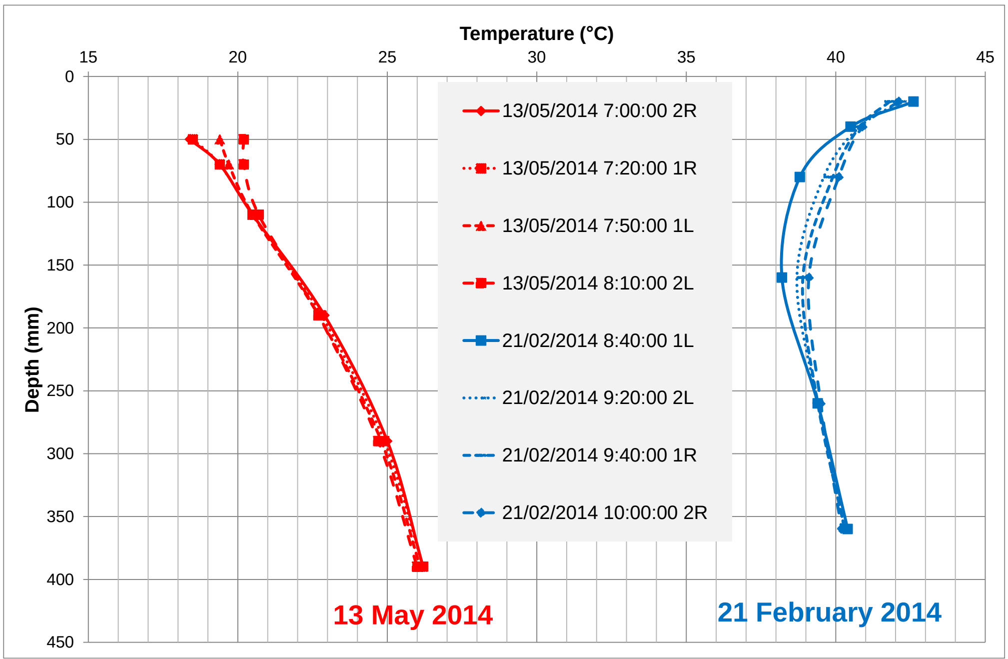
Depth (mm)	Temperature for each FWD test line (°C)			
	10:00:00	9:20:00	9:40:00	8:40:00
	2R	2L	1R	1L
20	42.1	42.2	41.8	42.6
40	40.9	40.8	40.8	40.5
80	40.1	39.6	39.9	38.8
160	39.1	38.7	38.9	38.2
260	39.5	39.4	39.4	39.4
360	40.2	40.3	40.3	40.4
Average temperature of the top 100 mm (°C)	41.0	40.9	40.8	40.6
Average temperature of the top 150 mm (°C)	40.6	40.3	40.4	40.0

Table 7.21: Temperature for each FWD test line on 13 May 2014

Depth (mm)	Temperature for each FWD test line (°C)			
	7:00:00	8:10:00	7:20:00	7:50:00
	2R	2L	1R	1L
50	18.4	20.2	18.5	19.4
70	19.4	20.2	19.4	19.7

Depth (mm)	Temperature for each FWD test line (°C)			
	7:00:00	8:10:00	7:20:00	7:50:00
	2R	2L	1R	1L
110	20.5	20.7	20.5	20.5
190	22.9	22.7	22.8	22.7
290	25.0	24.7	24.9	24.8
390	26.2	26.0	26.2	26.1
Average temperature of the top 100 mm (°C)	19.4	20.4	19.5	19.9
Average temperature of the top 150 mm (°C)	20.3	21.0	20.3	20.6

Figure 7.21: Pavement temperatures, Cullen Avenue West



7.4.3 *Initial Assessment of Pavement Construction Uniformity*

Subsequent back-calculation will be undertaken to determine the different layer modulus in the road-bed. However, a preliminary assessment of the construction quality uniformity was completed based on the temperature-corrected surface modulus (Figure 7.22 to Figure 7.28). It should be noted that FWD testing were performed using different maximum loads; in order to remove the differences in the maximum deflection (d0) due to the different loading and provide a comparison on the same basis, the surface modulus was calculated.

Figure 7.22: Calculated surface modulus of the eastbound traffic lane

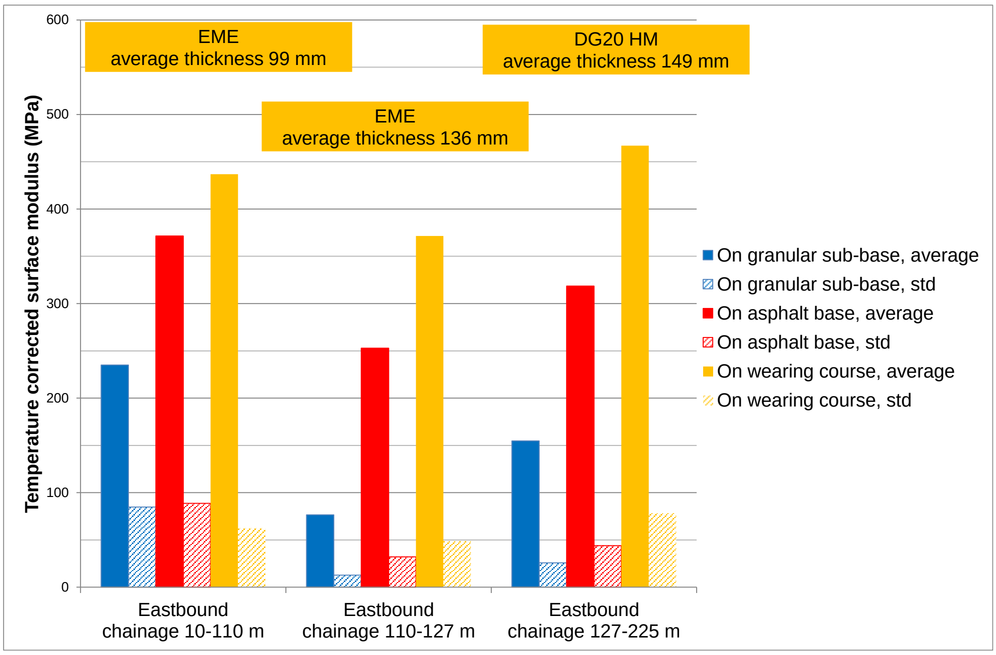


Figure 7.23: Calculated surface modulus of the westbound traffic lane

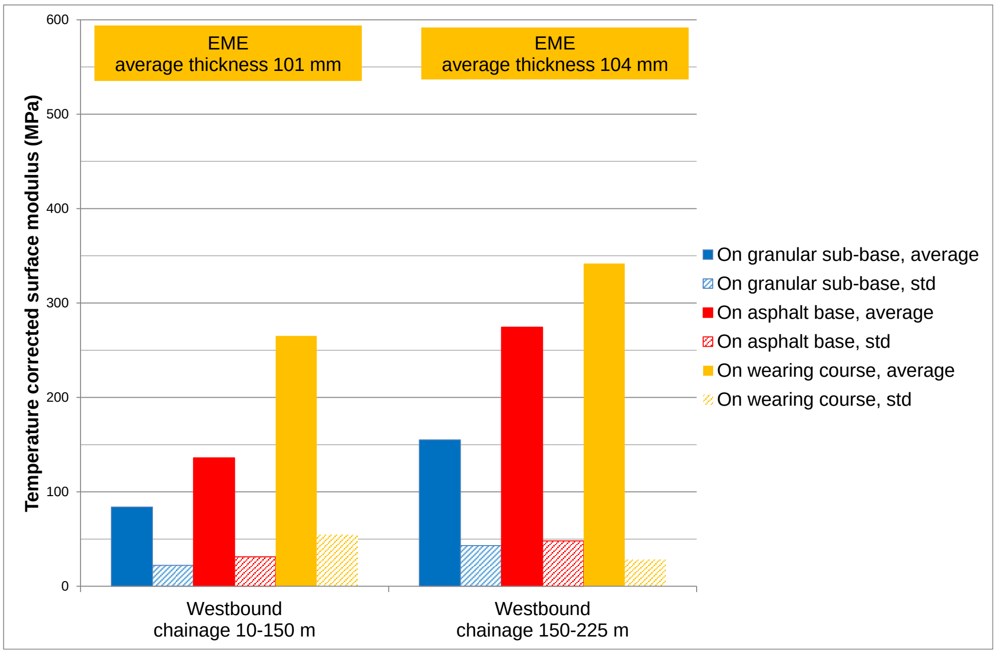


Figure 7.24: Temperature-corrected surface modulus, Cullen Avenue West, eastbound traffic lane (1L)

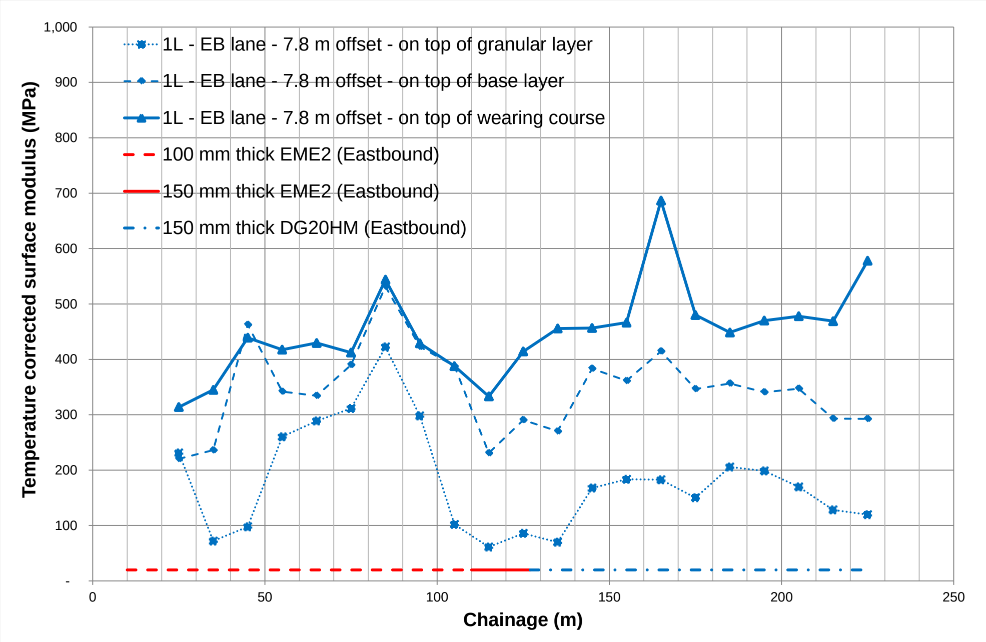


Figure 7.25: Temperature-corrected surface modulus, Cullen Avenue West, eastbound traffic lane (1R)

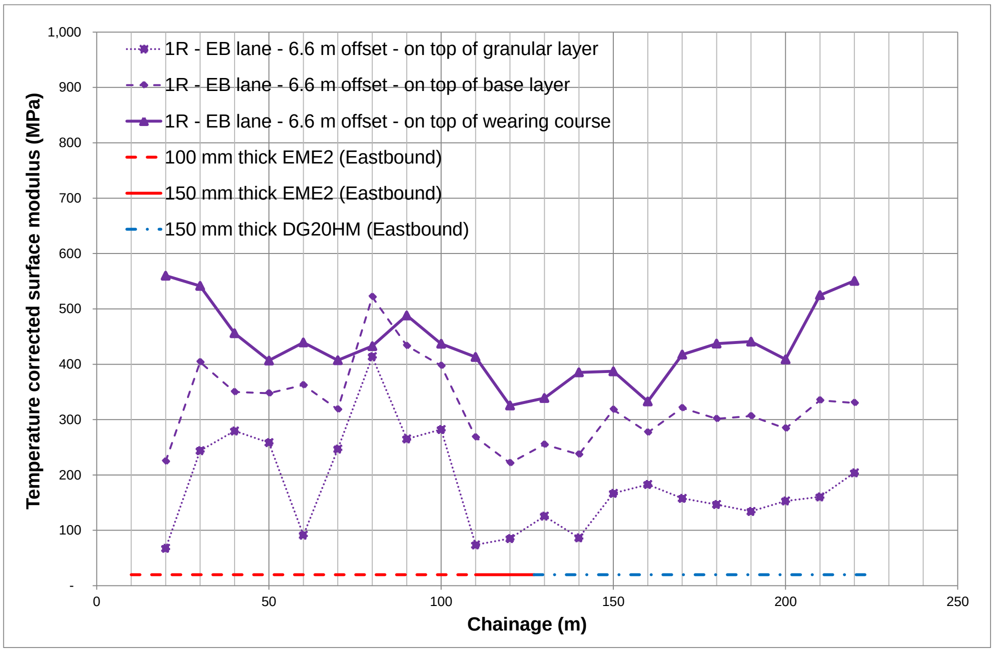


Figure 7.26: Temperature-corrected surface modulus, Cullen Avenue West, westbound traffic lane (2R)

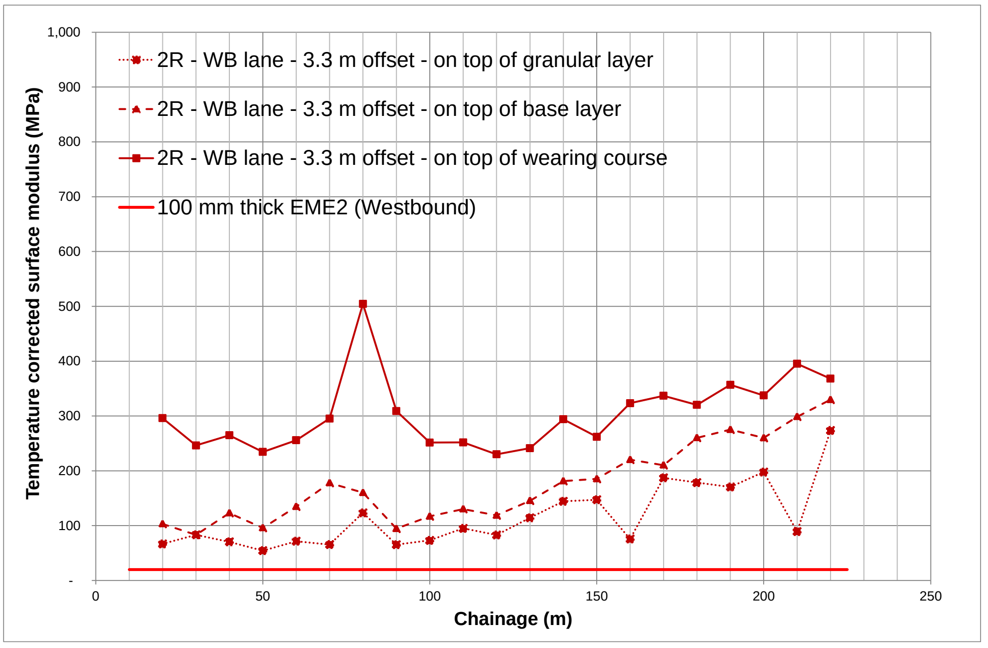


Figure 7.27: Temperature-corrected surface modulus, Cullen Avenue West, westbound traffic lane (2L)

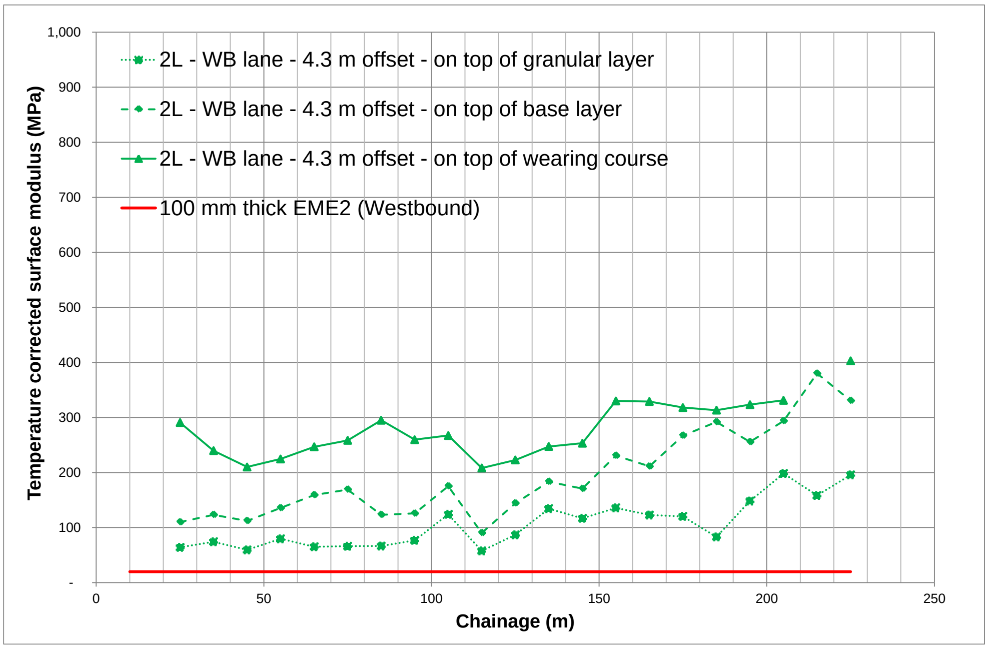
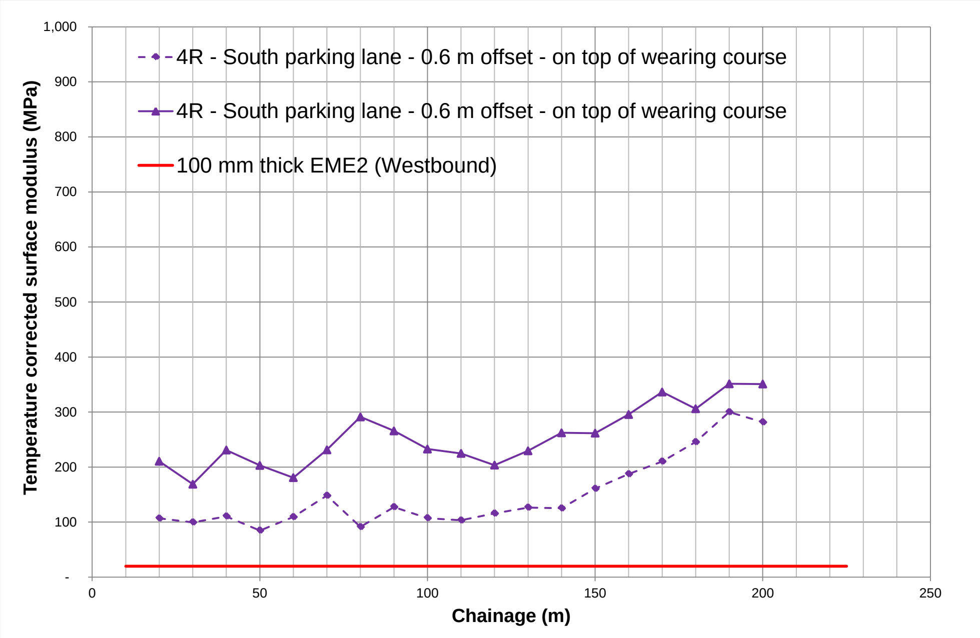


Figure 7.28: Temperature-corrected surface modulus, Cullen Avenue West, south parking lane (4R)



8 IMPLEMENTING THE EME2 TECHNOLOGY – PAVEMENT DESIGN CONSIDERATIONS

8.1 The Genesis of EME2 in France

When it was developed in France, the EME2 technology was typically based on the existing performance-based approach relying on both stiffness and fatigue properties; it should be noted that workability and wheel-tracking tests are part of the mix design process.

Thickness reductions are realised through both the higher stiffness, which reduces the critical strain, and the increase of the tolerable strain, from the improved laboratory fatigue performance.

When compared with classic asphalt base layer material (grave bitumen in French) the gain in terms of pavement thickness reduction can be analysed from the typical configurations proposed in the catalogue. Typical examples are shown in Table 4.8; the asphalt base can be decreased resulting in equivalent design life. It should be kept in mind that one of the French design approaches is based on a minimum subgrade stiffness of 50 MPa (e.g. CBR 5% in the Australian context) for roads with medium to heavy traffic.

8.2 Managing the Technology Transfer in Australia

Based on the information provided and discussed in this report, it is obvious that a successful EME2 technology transfer requires the development of tentative specification limits for the mix design using Australian test methods. Also, there has to be an applicable and reliable pavement design methodology available in Australia so that the full benefits of the EME2 technology can be realised.

There were three options identified in Sections 8.3, 8.4 and 8.5 for pavement design using EME2. The application will depend on the strategic directions given by TMR and the level of completeness of the framework required for these three options. While developing the strategy for implementation, it should be emphasised that the introduction of a new technology always requires periods of transition.

The three options are summarised as options 1, 2 and 3. Option 1 would utilise the current Austroads pavement design methodology, while options 2 and 3 are applicable for future developments and they require work to be completed before they can be partially or fully implemented. The utilisation of option 2 will become redundant, if the framework of option 3 will become available in a timely manner.

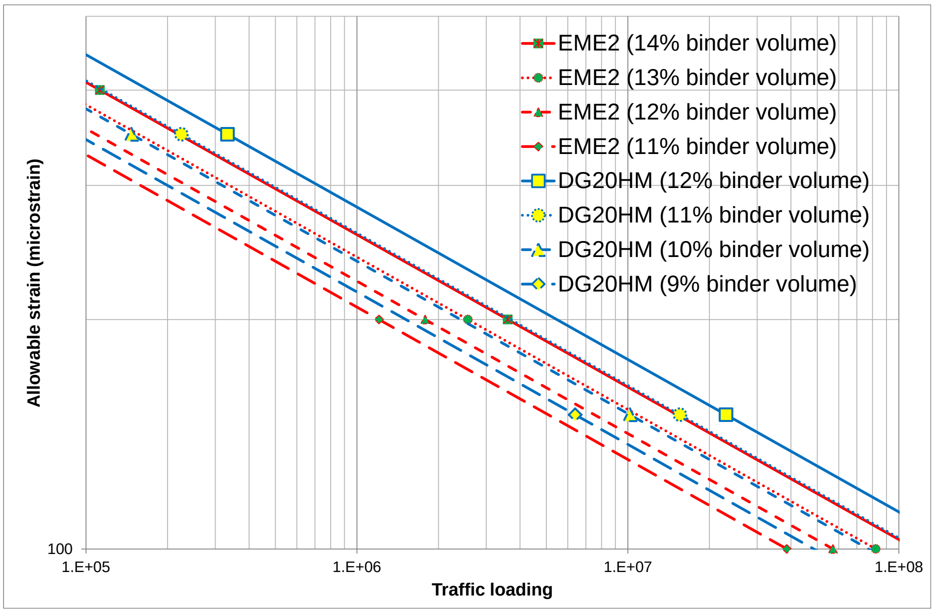
8.3 Option 1: Pavement Design using the Current Austroads Method

Option 1 uses the current pavement design method as outlined in Austroads (2012) and TMR *Pavement Design Supplement* (Department of Transport and Main Roads 2013). It should be noted that by using this methodology, all of the advantages of the improved EME2 mix properties may not be fully realised. This methodology would only utilise the volumetric properties and one performance parameter, i.e. the stiffness of the EME2 mix, and the designer may not be prompted to develop the most cost-effective mix *and* pavement design. Consequently the benefits of the EME2 technology may not be maximised.

However, by the application of the current Austroads (2012) methodology, as an interim measure, the utilisation of EME2 – within the existing pavement design framework – would be possible. This would, however, require that realistic and reliable asphalt stiffness (indirect tensile stiffness) values have to be selected and applied. The Austroads methodology and the TMR approach estimate the design moduli based on the resilient modulus measured using the standard indirect tensile test (ITT) adjusted to the in-service temperature (i.e. the WMAPT), in-service air voids and the rate of traffic loading in the roadbed. There is currently work underway in Austroads project TT1908 to determine the ITT modulus, also referred to as indirect tensile stiffness (ITS) in the relevant Australian Standard AS 2891.13.1, for EME2 mixes. Results collected so far indicate that at 40 ms rise time EME2 has an ITS value at least of 4500 MPa at 32 °C. This is a much higher value compared to a DG20HM, heavy duty asphalt with C600 binder, which has a design value of 2900 MPa for the same test conditions according to Department of Transport and Main Roads (2013).

The EME2 and DG20HM mix fatigue properties, if predicted according to the Austroads methodology (i.e. the Shell equation), may have very similar values as shown in Figure 8.1.

Figure 8.1: Comparison of EME2 and DG20HM fatigue properties (Shell prediction)

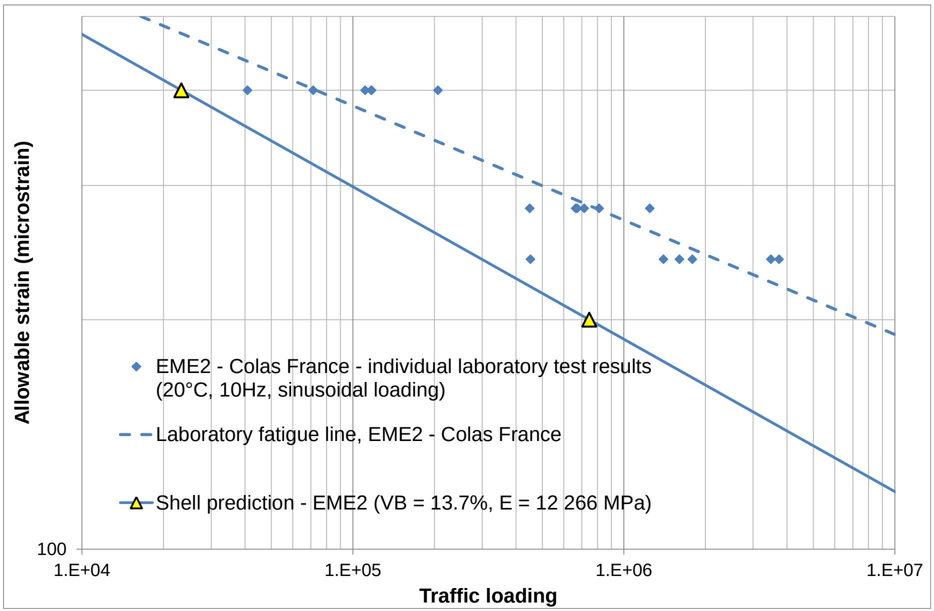


Since EME2 has higher stiffness, the benefits of designing thinner asphalt pavements by using the EME2 technology could be readily realised. However, the benefits are heavily dependent on the correct selection of the mix binder volume content as discussed in Section 6.2.3 and the selection of a realistic presumptive modulus value. Therefore the following should be considered:

- Although it seems reasonable that the binder volume (as required in Equation 21) should be selected according to the volumetric properties of the design mix, it is suggested that a pre-defined binder content is applied. A sensitivity analysis is provided in Section 6.2.3. By selecting an appropriate V_B value, in the interim, an additional safety factor may be built into the pavement design.
- It should also be noted that selecting an unrealistically low pre-defined binder volume for the EME2 mix – below 12% – may not provide all the benefits of this technology. There is currently work underway in project TT1908, where the EME2 mix and the control DG20HM mix sampled at the Cullen Avenue West trial site are subject to full fatigue characterisation in the laboratory and this information may provide input for selecting a realistic binder volume for EME2 mixes.
- According to the French pavement design system, different shift factors should be used for dense graded asphalt and EME2 (Table 3.1). However, when predicting the fatigue properties according to the Austroads pavement design methodology (as outlined in Equation 21) there is no difference in the reliability factor (RF) for different asphalt types, or asphalt mixes with different binder types. Therefore, the same reliability factors should be applied for EME2 mixes, when using option 1 for pavement design.

- The Shell equation as outlined in Equation 21 does not predict the fatigue properties for EME2 mixes accurately. Figure 8.2 provides a comparison of laboratory fatigue data obtained according to a 4-point bending test at 20 °C and 10 Hz, and predicted fatigue properties according to the Shell equation. Both the laboratory data and the prediction relate to a conforming EME2 mix imported from France. It can be seen that for the EME2 mix the Shell equation under-predicts the fatigue performance, resulting in a conservative pavement design. It should be noted that the laboratory fatigue test results show a different slope compared to the Shell prediction; the laboratory data predicts better field performance under very heavy traffic loading.
- The presumptive design modulus value for EME2 at 32 °C could be determined based on the work currently underway in project TT1908. A large number of cores were extracted from the finished trial pavement, which are being tested at the ARRB laboratory according to the ITS test (AS 2891.13.1). Upon completion of this study, the variability of the mix modulus could be assessed and a realistic design value could be selected for EME2. It is suggested that the modulus correction for loading speed as discussed in Austroads (2012) is utilised.
- It should be noted that the performance-based mix design process and the general mechanistic pavement design procedure would remain disconnected if the above methodology, i.e. option 1, is utilised. Austroads (2013a) details that the performance parameters of the EME2 mix cannot be accurately predicted using the volumetric properties. The mix performance is influenced by the binder and aggregate properties and it is a complex task to meet the stiffness and fatigue criteria, while also maintaining the high resistance to plastic deformation.

Figure 8.2: Comparison of laboratory fatigue data and predicted fatigue properties according to the Shell equation for a conforming EME2



Source: Austroads (2014 forthcoming).

In order to ensure that the EME2 mix performs long-term as expected, the full performance-based mix design procedure should be conducted; guidelines for this test series will be provided in an Austroads report published later this year. The tentative specification limits for the EME2 mix should be met using the Australian test methods.

As an interim measure, the ITS value should also be determined for the EME2 mixes. It should be noted that determination of the ITS value is not a specification requirement for EME2, and there is no intention to maintain this requirement for testing the ITS value in the future either. However, by collecting the ITS value along with the volumetric properties, EME2 mixes could be readily compared to existing asphalt mixes in the short-term.

Appendix D provides draft content for the Technical Note for designing pavements containing EME2 mixes.

In the longer-term, the mix design values, i.e. flexural stiffness and fatigue, should be introduced and utilised in the pavement design procedure (Sections 3 and 5 of this report). As an interim measure, when using the current Austroads pavement design method, the mix design properties would not be directly used in the pavement design. Although the mix design and pavement design process would be disconnected for the time being, it is paramount that the full performance characterisation is performed for the EME2 mix, ensuring that only complying mixes are used on projects. This approach would also prevent situations where by simply increasing the binder content of the EME2 mix and balancing the stiffness properties – for example by adding RAP – it would theoretically provide the best solution. Such a situation could prevent any further optimisation of the mix or, in extreme cases, would deliver a non-conforming mix and jeopardise the successful implementation of the EME2 technology.

8.4 Option 2: Pavement Design using the French Pavement Design Methodology

Option 2 would be an interim measure to introduce the performance-based mix design into the pavement structural design. If the framework of option 3 would become available in a timely manner, option 2 would become a redundant method. This option utilises the French mix design method as outlined in Sections 3, 5 and 7.3 and can be summarised as follows:

- conduct and complete the full performance-based design of the EME2 mix, including the temperature-frequency sweep for flexural stiffness
- based on the flexural stiffness modulus values, determine the pavement model
- calculate the pavement response by utilising the CIRCLY software package
- calculate the allowable strain values by using the French methodology as outlined in Sections 3, 5, 7.3 and Appendix C and compare the strain responses rather than calculate the continuous damage factor (CDF).

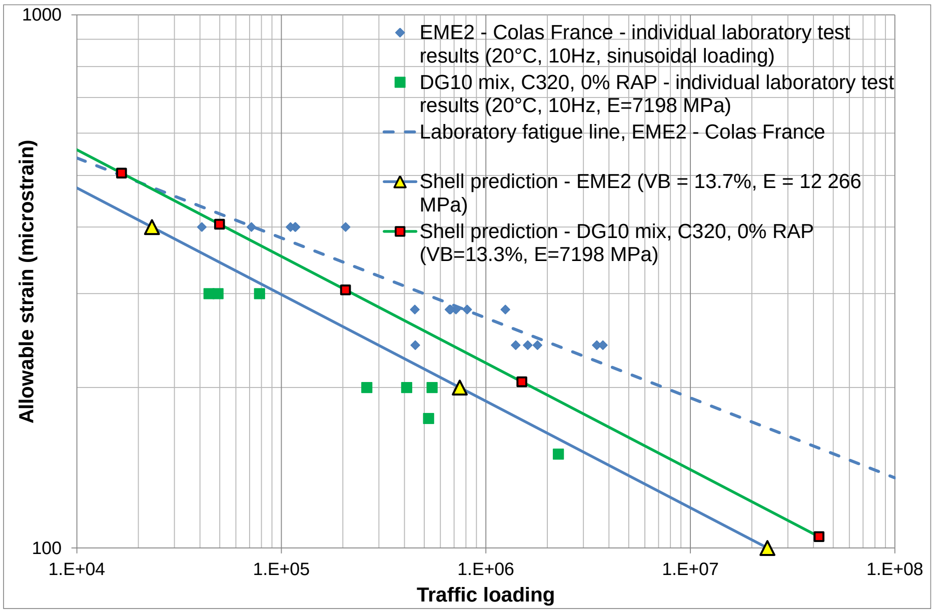
8.5 Option 3: Pavement Design using the Improved Austroads Pavement Design

Option 3 recommends utilising the pavement design method described in Austroads (2012) with an updated transfer function for asphalt fatigue. In Austroads project TT1826: *Improved design procedures for asphalt pavements* there is work currently underway for an updated methodology. In the current Austroads method the design reliability is incorporated in the reliability factor (RF), which represents a combination of a laboratory-to-field shift factor and a material safety factor (Denneman & Moffatt 2014). These two measures are clearly separated in the French design method.

The first step in coupling the asphalt mix design and pavement design is that the default general laboratory fatigue life model should be replaced with a mix-specific model (Denneman & Moffatt 2014). A possible utilisation of this methodology is provided in Equation 2. However, the laboratory-to-field shift factors, as indicated in Equation 2, have to be validated for Australian conditions. Also, the provided equation has to be simplified and re-arranged to be compatible with CIRCLY calculations.

Figure 8.3 demonstrates the need for individual laboratory fatigue testing. The fatigue properties of the EME2 mix (Figure 8.2) are compared with the fatigue properties of a DG10 mix using C320 binder and 0% RAP. The EME2 mix has a much better fatigue performance despite the extremely hard binder compared to the C320 bitumen. It should also be noted that the Shell fatigue prediction over-estimates the fatigue properties of the aforementioned DG10 mix.

Figure 8.3: Comparison of laboratory fatigue data and Shell fatigue prediction for EME2 and DG10 (C320) asphalt



The development of this methodology, i.e. option 3, would also require conducting a sensitivity analysis of the design methodology, especially related to the changed testing conditions suggested for EME2, which will be provided in an Austroads report published later this year, namely:

- a 4-point bending test at 15 °C, 10 Hz (the Australian method), instead of a 2-point bending test, 15 °C, 10 Hz (the French method) for determining the flexural modulus properties
- a 4-point bending test at 20 °C, 10 Hz (the Australian method), instead of a 2-point bending test, 10 °C, 25 Hz (the French method) for determining the fatigue properties.

The coefficient k_c, for adjusting the results of the computation model, is in line with the behaviour observed on actual pavements (shift factor) and coefficient k_r for adjusting the allowable strain according to the calculated risk of failure, should be validated. Coefficient k_s for accounting for the effect of a lack of uniformity in the bearing capacity of a soft soil layer can be considered as a shift factor and could be combined with and incorporated into coefficient k_r. By applying the two separate factors for k_c (shift) and k_r (reliability), the proposed improvements by Denneman and Moffatt (2014) could be applied. The k_c value equals to 1.0 for EME2 mixes in France (Section 3.1.2), which may be different in Australian climatic and loading conditions and this value has to be validated based on the data collected during the demonstration trials.

In the longer term this methodology would enable innovation and development of high-performing asphalt mixes.

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APPENDIX A DETERMINING A SHIFT FACTOR IN FRANCE (k_C VALUE)

Purpose

A pavement design system requires validation of the in situ behaviour of a pavement. The requirements and procedures for such a validation process are outlined in NF P 98-086 and it is summarised in this section. The process of validating the results from models and in-service performance is as follows:

- validate the calculation model by comparing the calculated stress, strain and deflection with the values measured on site
- determine the corrections to be applied to the allowable stress, strain and deflection in order that the predicted in-service life of the structure is consistent with that observed on site, when applying the same risk level.

These two operations may be carried out using a series of comparisons – theoretical pavement design/observations of the behaviour of real pavements – spread over time, sometimes over several years.

Methodology

For each pavement type, validation of the model is carried out on real pavement sections by measuring parameters corresponding to the chosen design criteria and by comparison with control sections. These sections may be fitted with sensors. Failing this, the measurement of relative movements through the use of accelerated pavement testing (APT) is permissible for pavements treated with hydraulic or bituminous binders. For flexible pavements, deflection and curvature measurements are taken at the level of the reference load.

The control sections are:

- either a group of sections integrated into an existing road network; the test series on these sections should include documented evidence concerning the effects of climate and ageing of the material
- or full-scale accelerated pavement test sections, subject to these sections having sufficient dimensions.

The length of these control sections should be long enough and representative in order to replicate normal manufacturing and paving conditions (30 m long and 2.5 m wide). For each of these control sections, the following information should be collected:

- the number of equivalent axles or reference load, *NE*
- the type, thickness and characteristics of the layers comprising the pavements, in particular the composition and mechanical performance of the materials in the various layers and the sub-grade
- the type of bond between the various layers
- the results of the production control and in situ testing.

For completely new techniques, the number of control sections should be sufficiently high (from 5 to 15) to allow for statistical interpretation.

APPENDIX B CIRCLY REPORTS

EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME2 Title: Cullen Ave

   Load  Load         Movements
    No.  ID
    1    ESA75-Full   1.47E+06

Details of Load Groups:

   Load  Load         Load        Load            Radius    Pressure/    Exponent
    No.  ID           Category    Type                      Ref. stress
    1    ESA75-Full   SA750-Full  Vertical Force     92.1     0.75         0.00

   Load Locations:
   Location   Load         Gear          X          Y      Scaling     Theta
    No.       ID           No.                              Factor
    1         ESA75-Full    1         -165.0        0.0    1.00E+00      0.00
    2         ESA75-Full    1          165.0        0.0    1.00E+00      0.00
    3         ESA75-Full    1         1635.0        0.0    1.00E+00      0.00
    4         ESA75-Full    1         1965.0        0.0    1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:      0

Details of Layered System:

   ID: EME2-1 Title: EME2-1

   Layer  Lower    Material     Isotropy   Modulus    P.Ratio
    No.   i/face   ID                      (or Ev)    (or vvh)   F          Eh         vh
    1     rough    Type 2       Iso.       1.80E+03   0.40
    2     rough    EME2         Iso.       5.40E+03   0.40
    3     rough    Sub_CBR5     Aniso.     5.00E+01   0.45       3.45E+01   2.50E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.   Traffic
    No.            ID                      Constant   Exponent   Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000      1.100
    2     bottom   EME2         ETH         0.004096    5.000      1.100
    3     top      Sub_2004     EZZ         0.009300    7.000      1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -5.62E-05      2.49E-04
    2      100.00    EME2         ESA75-Full    -2.70E-04      1.01E+00
    3        0.00    Sub_CBR5     ESA75-Full     8.72E-04      1.50E-01
```

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EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME21 Title: 2

   Load   Load         Movements
    No.   ID
    1     ESA75-Full   1.47E+06

Details of Load Groups:

   Load   Load        Load        Load            Radius    Pressure/    Exponent
    No.   ID          Category    Type                      Ref. stress
    1     ESA75-Full  SA750-Full  Vertical Force     92.1    0.75         0.00

   Load Locations:
   Location   Load        Gear          X          Y      Scaling     Theta
    No.       ID          No.                             Factor
    1         ESA75-Full   1          -165.0        0.0   1.00E+00      0.00
    2         ESA75-Full   1           165.0        0.0   1.00E+00      0.00
    3         ESA75-Full   1          1635.0        0.0   1.00E+00      0.00
    4         ESA75-Full   1          1965.0        0.0   1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:     0

Details of Layered System:

   ID: EME2-11 Title: 2

   Layer  Lower    Material     Isotropy  Modulus    P.Ratio
    No.   i/face   ID                     (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Type 2       Iso.      1.80E+03   0.40
    2     rough    EME2         Iso.      5.40E+03   0.40
    3     rough    Sub_CBR10    Aniso.    1.00E+02   0.45      6.90E+01   5.00E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.  Traffic
    No.            ID                      Constant   Exponent  Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000     1.100
    2     bottom   EME2         ETH         0.004096    5.000     1.100
    3     top      Sub_2004     EZZ         0.009300    7.000     1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -3.86E-05      3.80E-05
    2      100.00    EME2         ESA75-Full    -2.20E-04      3.65E-01
    3        0.00    Sub_CBR10    ESA75-Full     6.41E-04      1.73E-02
```

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EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME2 Title: Cullen Ave

   Load   Load         Movements
    No.   ID
    1     ESA75-Full   1.47E+06

Details of Load Groups:

   Load   Load        Load        Load            Radius    Pressure/    Exponent
    No.   ID          Category    Type                      Ref. stress
    1     ESA75-Full  SA750-Full  Vertical Force     92.1    0.75         0.00

   Load Locations:
   Location   Load        Gear          X          Y      Scaling     Theta
    No.       ID          No.                             Factor
    1         ESA75-Full   1         -165.0        0.0  1.00E+00      0.00
    2         ESA75-Full   1          165.0        0.0  1.00E+00      0.00
    3         ESA75-Full   1         1635.0        0.0  1.00E+00      0.00
    4         ESA75-Full   1         1965.0        0.0  1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:     0

Details of Layered System:

   ID: EME2-111 Title: 3

   Layer  Lower    Material     Isotropy   Modulus    P.Ratio
    No.   i/face   ID                      (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Type 2       Iso.       1.80E+03   0.40
    2     rough    EME2         Iso.       5.40E+03   0.40
    3     rough    Sub_CBR15    Aniso.     1.50E+02   0.45      1.03E+02   7.50E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.  Traffic
    No.            ID                      Constant   Exponent  Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000     1.100
    2     bottom   EME2         ETH         0.004096    5.000     1.100
    3     top      Sub_2004     EZZ         0.009300    7.000     1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -3.11E-05      1.28E-05
    2      100.00    EME2         ESA75-Full    -1.93E-04      1.88E-01
    3        0.00    Sub_CBR15    ESA75-Full     5.30E-04      4.58E-03
```

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EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME2 Title: Cullen Ave

   Load   Load         Movements
    No.   ID
    1     ESA75-Full   1.47E+06

Details of Load Groups:

   Load   Load        Load        Load            Radius    Pressure/    Exponent
    No.   ID          Category    Type                      Ref. stress
    1     ESA75-Full  SA750-Full  Vertical Force     92.1    0.75         0.00

   Load Locations:
   Location   Load        Gear          X          Y      Scaling     Theta
    No.       ID          No.                             Factor
    1         ESA75-Full   1         -165.0        0.0   1.00E+00      0.00
    2         ESA75-Full   1          165.0        0.0   1.00E+00      0.00
    3         ESA75-Full   1         1635.0        0.0   1.00E+00      0.00
    4         ESA75-Full   1         1965.0        0.0   1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:     0

Details of Layered System:

   ID: EME2-1111 Title: 4

   Layer  Lower    Material     Isotropy   Modulus    P.Ratio
    No.   i/face   ID                      (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Type 2       Iso.       1.80E+03   0.40
    2     rough    DG20HM       Iso.       2.36E+03   0.40
    3     rough    Sub_CBR5     Aniso.     5.00E+01   0.45      3.45E+01   2.50E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.  Traffic
    No.            ID                      Constant   Exponent  Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000     1.100
    2     bottom   DG20HM       ETH         0.004253    5.000     1.100
    3     top      Sub_2004     EZZ         0.009300    7.000     1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -6.02E-05      3.50E-04
    2      100.00    DG20HM       ESA75-Full    -4.48E-04      1.05E+01
    3        0.00    Sub_CBR5     ESA75-Full     1.25E-03      1.82E+00
```

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EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME2 Title: Cullen Ave

   Load   Load         Movements
    No.   ID
    1     ESA75-Full   1.47E+06

Details of Load Groups:

   Load   Load        Load        Load            Radius    Pressure/    Exponent
    No.   ID          Category    Type                      Ref. stress
    1     ESA75-Full  SA750-Full  Vertical Force     92.1    0.75         0.00

   Load Locations:
   Location   Load        Gear          X          Y      Scaling     Theta
    No.       ID          No.                             Factor
    1         ESA75-Full   1         -165.0        0.0   1.00E+00      0.00
    2         ESA75-Full   1          165.0        0.0   1.00E+00      0.00
    3         ESA75-Full   1         1635.0        0.0   1.00E+00      0.00
    4         ESA75-Full   1         1965.0        0.0   1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:     0

Details of Layered System:

   ID: EME2-11112 Title: 5

   Layer  Lower    Material     Isotropy   Modulus    P.Ratio
    No.   i/face   ID                      (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Type 2       Iso.       1.80E+03   0.40
    2     rough    DG20HM       Iso.       2.36E+03   0.40
    3     rough    Sub_CBR10    Aniso.     1.00E+02   0.45      6.90E+01   5.00E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.  Traffic
    No.            ID                      Constant   Exponent  Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000     1.100
    2     bottom   DG20HM       ETH         0.004253    5.000     1.100
    3     top      Sub_2004     EZZ         0.009300    7.000     1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -4.04E-05      4.78E-05
    2      100.00    DG20HM       ESA75-Full    -3.57E-04      3.36E+00
    3        0.00    Sub_CBR10    ESA75-Full     9.07E-04      1.98E-01
```

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EME2.txt

```
CIRCLY Version 5.0u (18 December 2012)

Job Title: EME2 Cullen Ave

Damage Factor Calculation

Assumed number of damage pulses per movement:
   One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

   ID: EME2 Title: Cullen Ave

   Load   Load         Movements
    No.   ID
    1     ESA75-Full   1.47E+06

Details of Load Groups:

   Load   Load        Load        Load            Radius    Pressure/    Exponent
    No.   ID          Category    Type                      Ref. stress
    1     ESA75-Full  SA750-Full  Vertical Force     92.1    0.75         0.00

   Load Locations:
   Location   Load        Gear          X          Y      Scaling     Theta
    No.       ID          No.                             Factor
    1         ESA75-Full   1         -165.0        0.0   1.00E+00      0.00
    2         ESA75-Full   1          165.0        0.0   1.00E+00      0.00
    3         ESA75-Full   1         1635.0        0.0   1.00E+00      0.00
    4         ESA75-Full   1         1965.0        0.0   1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin: -165   Xmax:  1800   Xdel:  20
   Y:     0

Details of Layered System:

   ID: EME2-11111 Title: 6

   Layer  Lower    Material     Isotropy   Modulus    P.Ratio
    No.   i/face   ID                      (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Type 2       Iso.       1.80E+03   0.40
    2     rough    DG20HM       Iso.       2.36E+03   0.40
    3     rough    Sub_CBR15    Aniso.     1.50E+02   0.45      1.03E+02   7.50E+01   0.45

   Performance Relationships:
   Layer  Location Performance  Component  Perform.   Perform.  Traffic
    No.            ID                      Constant   Exponent  Multiplier
    1     bottom   Type 2       ETH         0.004489    5.000     1.100
    2     bottom   DG20HM       ETH         0.004253    5.000     1.100
    3     top      Sub_2004     EZZ         0.009300    7.000     1.600

   Reliability Factors:
   Project Reliability: Austroads 85%
   Layer  Reliability  Material
    No.   Factor       Type
    1       2.00       Asphalt
    2       2.00       Asphalt
    3       1.00       Subgrade (Austroads 2004)


Results:

   Layer  Thickness  Material     Load          Critical       CDF
    No.              ID           ID            Strain
    1       30.00    Type 2       ESA75-Full    -3.29E-05      1.72E-05
    2      100.00    DG20HM       ESA75-Full    -3.07E-04      1.59E+00
    3        0.00    Sub_CBR15    ESA75-Full     7.45E-04      5.00E-02
```

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APPENDIX C CALCULATION OF ALLOWABLE STRAIN – FRENCH PAVEMENT DESIGN METHOD

Details of the calculations and abbreviations are provided in Sections 3 and 5.

Pavement structure	Property	EME2	EME2	EME2	GB2&3	GB2&3	GB2&3
Formation support	MPa	50	100	150	50	100	150
Traffic	AADT (traffic class T4)	27	27	27	27	27	27
	Design period - p (year)	40	40	40	40	40	40
	τ (%)	2	2	2	2	2	2
	C	60	60	60	60	60	60
	CAM (pavement)	0.3	0.3	0.3	0.3	0.3	0.3
	NE (pavement)	178 578	178 578	178 578	178 578	178 578	178 578
	NPL	595 262	595 262	595 262	595 262	595 262	595 262
Subgrade vertical strain	Light traffic	0.016	0.016	0.016	0.016	0.016	0.016
	CAM (subgrade)	0.5	0.5	0.5	0.5	0.5	0.5
	NE	297 631	297 631	297 631	297 631	297 631	297 631
	Exponent	-0.222	-0.222	-0.222	-0.222	-0.222	-0.222
	$\varepsilon_{vertical}$	**975E-6**	**975E-6**	**975E-6**	**975E-6**	**975E-6**	**975E-6**
Allowable asphalt strain	$T_{equivalent}$	32	32	32	32	32	32
	E (10 °C, 10Hz) (MPa)	17 000	17 000	17 000	12 300	12 300	12 300
	E (32 °C, 10Hz) (MPa)	5400	5400	5400	2360	2360	2360
	ε_6 (10 °C, 25Hz)	130E-6	130E-6	130E-6	90E-6	90E-6	90E-6
	ε_6 (15 °C, 10Hz)	326E-6	326E-6	326E-6	290E-6	290E-6	290E-6
	Pavement thickness (cm)	10	10	10	10	10	10
	Formation support (MPa)	50	100	150	50	100	150
	Risk (%)	12	12	12	12	12	12
	u	-1.175	-1.175	-1.175	-1.175	-1.175	-1.175
	b	-0.2	-0.2	-0.2	-0.2	-0.2	-0.2
	c	0.02	0.02	0.02	0.02	0.02	0.02
	sh	1.0	1.0	1.0	1.0	1.0	1.0
	SN	0.25	0.25	0.25	0.3	0.3	0.3
	δ	0.269	0.269	0.269	0.316	0.316	0.316
	kr	0.864	0.864	0.864	0.843	0.843	0.843
	kc	1.0	1.0	1.0	1.3	1.3	1.3
	ks	0.91	0.94	1.00	0.91	0.94	1.00
	$\varepsilon_{t, allow}$	**255.8 E-6**	**264.2 E-6**	**281.4 E-6**	**288.8 E-6**	**298.3 E-6**	**317.7 E-6**

APPENDIX D DRAFT CONTENT FOR THE TECHNICAL NOTE

D.1 Pavements Containing EME2 Mixes

Determination of design modulus

For pavements containing EME2 mix the presumptive design modulus should be used; the applied Poission's ratio is 0.4.

The EME2 mix should comply with the requirements outlined in Project Specific Technical Specification (PSTS) 107 - *EME2 (Enrobé à Module Élevé) – High Modulus Asphalt*. As an interim measure the ITT value of the complying mix should be determined according to AS 2891.13.1 at 32 °C. The purpose of this additional testing is to ensure that minimum pavement design values are met for the design mix. It is not allowed to use a higher design modulus than the presumptive value.

The following presumptive values for elastic characterisation of asphalt mixes at a WMAPT of 32 °C should be used for EME2 mixes (extension of Table Q6.5 of the TMR Supplement).

Asphalt mix type	Binder type	Volume of binder (%) [1]	Asphalt modulus at heavy vehicle operating speed (MPa) [2] (1)			
			10 km/h	30 km/h	50 km/h	80 km/h
EME2	EME binder (15/25 pen)	13	E = 0.45 * 4500	E = 0.65 * 4500	E = 0.8 * 4500	4500

1: Modulus correction for loading speed is calculated according to Austroads (2012).

Pavements containing EME2 asphalt mix should be designed as full depth asphalt (FDA) pavement according to Table Q2.1 of the TMR pavement design supplement. The application of EME2 is not restricted for traffic categories and may be designed for any average daily ESA value for both a rural and urban environment. Minimum bearing capacity of the pavement should exceed CBR5% (50 MPa) at all points on top of the improved layer; also, care should be exercised that the improved layer is not heavily modified or stabilised. Ensuring minimum bearing capacity under the lowest EME2 layer is critical for achieving adequate compaction and long-term performance.

The performance-based mix design, which is required for EME2 mixes, and the general mechanistic pavement design procedure is disconnected, by utilising the above approach. It is discussed in details in Austroads (2013a) that the performance parameters of EME2 mixes cannot be accurately predicted using the volumetric properties. The mix performance is influenced by the binder and aggregate properties and it is a complex task to meet the stiffness and fatigue criteria, while also maintaining the high resistance to plastic deformation. In Austroads and TMR projects there is currently work underway to introduce fully performance-based mix design properties into the pavement structural design. Until this work is completed, the above procedure should be utilised for designing EME2 pavements as an interim measure.

[1] The binder content is provided in this table as an example.

[2] The presumptive modulus values are provided in this table as an example.