Asphalt Pavements Solutions for Life

Technical Background to the Development of the AAPA Long Life Asphalt Pavement Design Procedure

Bevan Sullivan, Fulton Hogan Ltd
Geoff Youdale, Australian Asphalt Pavement Association
Ian Rickards, Australian Asphalt Pavement Association
Saeed Yousefdoost, Swinburne University of Technology

Editor Dougall Broadfoot, Australian Asphalt Pavement Association
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Nigel Preston, Viva Energy
Ian Rickards, Consultant
Greg Stephenson, Brisbane City Council
Bevan Sullivan, Fulton Hogan
Geoff Youdale, Consultant

Peter Armstrong, Fulton Hogan
Trevor Distin, Boral Asphlat

Australian Asphalt Pavements Association Head Office: Level 2, 5 Wellington Street, KEW Victoria 3101
Tel: (03) 9853 3595 Fax: (03) 9853 3484 E-mail: info@aapa.asn.au Web: www.aapa.asn.au
**Executive Summary**

In 2011 the Australian Asphalt Pavement Association (AAPA) commenced the Asphalt Pavement Solutions for Life (APS-fL) project. This project was initiated to address the concerns of clients, consultants and industry that current pavement design procedures were producing overly conservative asphalt thickness requirements.

It was believed that these overly conservative design outcomes were due to a number of issues, such as;

- limited data on material characterisation of Australian mixes which is resulting the use of overly conservative material properties,
- the lack of a calibrated and validated shift factor between laboratory observations and field performance,
- the lack of incorporation of the healing mechanism in asphalt mixes, and
- the lack of recognition of a `threshold strain or Fatigue Endurance Limit (FEL) below which no damage occurs.

Most importantly, it was felt that if a FEL concept could be incorporated into a pavement design procedure, Long Life Asphalt Pavements (LLAPs) could be introduced to Australia. The result of which would be that the maximum thickness of asphalt could be determined, beyond which any increase in design thickness will result in little to no increase in the structural capacity of the pavement. This aspect was conducted as part of the APS-fL project, with the intent of developing a framework for the incorporation of the LLAP design into the Austroads, Mechanistic Empirical Design procedures.

In addition to the lack of a FEL one of the primary issues identified by industry was the limited data on material characteristics of typical Australian production mixes. Because of this limited dataset it was believed the values being adopted for the purpose of pavement design were overly conservative. Additionally they were not validated to field measurements. To obtain actual information and fill this gap, the APS-fL project set out to characterise typical Australian production mixes using the dynamic modulus test. The dynamic modulus test was selected;

- to be consistent with the direction being taken internationally,
- to enable measurement of modulus over the full time temperature space (through the development of master curves), and
- most significantly, to enable linking of Australian mix characterisation with overseas research and test track data.

As a result, the material characterisation study of 28 Australian mixes has been used to develop a full set of dynamic modulus master curves. This will enable, for the first time, the calculation of
modulus of Australian production mixes at any load frequency and temperature applicable to Australian field conditions.

There is still significant debate about the exact conversion between frequency in the dynamic modulus tests and loading time in a pavement structure. Hence this study used the results of field measured modulus from the FWD testing on the NCAT, Westrack and MnRoads test tracks sites to develop and validate a direct inter-conversion between the dynamic modulus test and field stiffness. The conversion was then used to validate strain results obtained at the NCAT test track against strains predicted by the use of layered elastic analysis and the converted stiffness.

This study then investigated a number of procedures for the incorporation and modelling of the FEL using both Australian and overseas laboratory data. The procedures were then compared against the performance of LLAP, by examining the effects of temperature, modulus and strain levels at the NCAT test track and a recommended and calibrated modelling approach was developed. The recommended NCAT model was then validated against Australian LLAP sites and was adjusted to match the field performance of Australian and UK pavements incorporating a confidence based FEL modelling approach.

The study then made recommendations on the model form, number of seasons and vehicle classification required to model performance based on calibration and validation against actual performance. This provides an effective and efficient methodology to design LLAP.

By reviewing the strategies for designing and maintaining long-life pavements in Australia, the UK, France, Netherlands and several states in the US, it was found LLAP could be achieved with a maximum thickness of 300-350mm and a minimum of 200mm was required for LLAP performance.

Examination of the gradation and volumetric properties of SRA Australian production mixes shows that despite the variability in the design methods (Marshall, gyratory and Superpave) and the differing compaction efforts, all mixes fit into a very small volumetric window. Additionally it was found the design gradation of all standard Australian production mixes closely follow the maximum density line, with nearly all mixes being slightly coarse graded, indicating that distinguishing between Australian mixes based on gradation and volumetric properties may be difficult.

The comparison of the results obtained by AAPA and NCAT on two un-aged samples showed that the difference in the compaction method between the AAPA study and US Superpave test method had no influence on the measured dynamic modulus results, with identical results obtained. Based on this finding, it was concluded that the APS-fL project could utilize the results of NCAT testing for modulus and performance data with confidence, to;

- correlate dynamic modulus estimates from back analysis of deflection data,
- validate measured strain and predicted strain using linear elastic analysis, and
- develop fatigue endurance limits.
Examination of the master curves of standard Australian production mixes suggested the minimum modulus value appeared to be the best at distinguishing between binders and nominal aggregate size. As there is little difference in the volumetrics of Australian mixes, no significance could be found in air void levels or binder contents within sub mix types. Unexpectedly, no correlation was found between the minimum modulus and RAP content, indicating that at current RAP levels, RAP has little effect on the minimum modulus value and therefore overall modulus values. The results showed that, most likely because of the small variance in aggregate gradation and volumetric properties, there was no change in the shape of the master curve within groups of the same binder types and nominal aggregate size.

Because of the consistency of the master curves for a given binder type and nominal mix size, it was found for practical implementation that it was not necessary to develop complex master curve equations for routine pavement designs. The results of grouping of Australian mixes showed that Australia can achieve a higher degree of accuracy by grouping common mixes, than from the use of complex model such as the Witzczak or the Hirsch models.

It was found that because the variability in the prediction of modulus followed a normal distribution with the standard deviation equal to that of the standard error, confidence could be established from the grouped data by simply varying the minimum modulus data to move the dynamic modulus curve down the modulus scale. By doing this it was shown that confidence level master curves could be established for the nominal 14 and 20mm mixes and the three primary binder classes used in Australia, C320, AR450 and C600.

The analysis of dynamic modulus test results from NCAT, MnRoads and WesTrack, against field stiffness determined from FWD testing, found that frequency in the dynamic modulus test should be considered as an angular frequency and that a shift of 1/2π on the frequency axis will allow the use of dynamic modulus values to determine the modulus resulting from a pulse load in the field. Using this conversion it was found that dynamic modulus results at 5.3Hz (1/(2π*0.03)) could be used to accurately predict the modulus determined from FWD loading with a pulse width of approximately 0.03seconds over a wide range of temperatures. The results of the optimisation found that the use of the mid-layer depth resulted in a slight underestimation of the effective asphalt layer modulus for day time testing and that if mid-layer depth is used a correction of +2°C is required to correct for the average temperature within the asphalt layers. Therefore, if modulus calculations were to be undertaken at times of day other than typically mid-day, more work on modelling the full temperature with depth profile in the pavement structure would be needed to determine the effective temperature of the asphalt layers.

Using the pulse frequency conversion and temperature correction obtained from comparison with FWD testing, the multi-variable optimisation found that dynamic modulus could be used to accurately predict strain under a moving load using layer elastic analysis when time of load is corrected for the effective load length. It was found that when computing strain under a moving
load, contrary to some published recommendations, the thickness of the asphalt layer was insignificant in determining strains. It was also found that the time of loading is more related to the length of the deflection response than the current approach of the use of a stress pulse.

The results of the optimisation on the thick asphalt sections of Phase II NCAT support the recommendation of Coffman (1968) that a vehicle acts as a cyclic load with a wave length of six feet, with the optimisation determining the wave length of 1.8m. Based on these findings the following frequencies in the dynamic modulus test are recommended for use in pavement design with an equivalent combined asphalt layer.

<table>
<thead>
<tr>
<th>Speed (km/hr.)</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
</tr>
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<tbody>
<tr>
<td>Recommended Frequency Dynamic Modulus E*</td>
<td>Test (Hz)</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>2.0</td>
<td>2.2</td>
<td>2.5</td>
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The analysis showed that multi layers were sensitive to the chosen sub layer thicknesses and more work would be required on determining both the appropriate sub layering of multilayer asphalts and the effect of temperature profiles in the sub layering, before a multilayer approach should be recommended over the use of the equivalent asphalt layer approach.

The analysis has shown that there is a direct link between laboratory modulus and strain under a moving vehicle and dynamic modulus can be used in the structural design of LLAP's. The use of the master curves will enable the determination of either threshold strains or cumulative distribution of asphalt strain in LLAP structures as a function of the climatic conditions and the traffic distribution spectrum. This calculated strain will provide the means to rationally evaluate the compliance of candidate LLAP structures with the limiting threshold strain or cumulative strain distribution empirically derived from the evaluation of international LLAP's.

The results of the modulus inter-conversion study found that 3 of the 4 common Australian test methods time has a different physical meaning and a frequency conversion is needed to shift between the time and frequency domain. It was found that shift factors could be established from single time/temperature testing and that the time shift factors found form the single time and temperature testing and are valid across the whole time frequency domain.

For the purposes of standardisation, the modulus results need to be converted to a reference stiffness value. Comparable stiffness is obtained, between any of the three test methods by using a constant definition of time (inter-conversion) as shown following.

<table>
<thead>
<tr>
<th>Conversion From</th>
<th>Conversion To</th>
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<tr>
<td>IT-CY(time)</td>
<td>IT-CY(time)</td>
</tr>
<tr>
<td>4PB-PR(frequency)</td>
<td>4PB-PR(frequency)</td>
</tr>
<tr>
<td>DC-CY(frequency)</td>
<td>DC-CY(frequency)</td>
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It is evident from this study that frequency in the common Australian tests can have different physical meaning and the reporting of modulus at a frequency, without reference to the loading type is confusing. To remove this confusion, it is recommended that standard practice be established for the reporting of modulus values from different test methods and is referenced back to an equivalent design modulus. The study underlines the considerable challenges in comparing the modulus results from various test methods. It would be highly recommendable to harmonise testing across Australia and implement a standard reporting method.

For both the low strain 4PB-PR results and the 2PB-TR only limited data has been used and the results should be confirmed over a wider range of mixes.

The examination of extensive overseas research showed that there is a clear and strong relationship between mix stiffness and the FEL of asphalt mix. The international research has found that as stiffness of the mix increases the FEL decreases asymptotically and most likely reaches a limiting value. The research work showed that the basic material property, stiffness, is directly related to the FEL of a mix and can be used to allow for changes in binder, temperature and healing.

The examination of the LLAP sites from the NCAT Phase II study confirmed, in field, the variable nature of the FEL relationship and that LLAP can withstand strains significantly higher than previously recommended (when the asphalt has low stiffness) without undergoing damage. The examination of the two LLAP on the NCAT test track showed a direct relationship exists between the infield stiffness-strain curve of the two undamaged sections and the stiffness-FEL developed by from Australian and US mixes. This finding was then extended to actual LLAP in Australia, which confirmed the same finding as on the NCAT test track that a direct relationship exists between the infield stiffness-strain curve of LLAP and the laboratory determined stiffness-FEL curve. This direct relationship allows the development of a phenomenological stiffness-FEL relationship which can be used for the purpose of pavement design.

Based upon both the calibrated relationship from the NCAT data and validated Australian and UK LLAP, the use of a stiffness-FEL relationship is recommended for the design of long-life asphalt pavements in Australia. The relationship was validated to Australian and UK data and only required a slight modification to the relationship developed from the NCAT, as shown following.

\[ \text{Where;} \]

\[
\text{4PB-PR (frequency)} & \text{(frequency)} & \text{DC-CY (frequency)} \\
\hline
\text{2PB-TR (frequency)} & \text{2PB-TR (frequency)} & \text{2PB-TR (frequency)} \\
\hline
\]

\[
\text{Based upon both the calibrated relationship from the NCAT data and validated Australian and UK LLAP, the use of a stiffness-FEL relationship is recommended for the design of long-life asphalt pavements in Australia. The relationship was validated to Australian and UK data and only required a slight modification to the relationship developed from the NCAT, as shown following.}
\]

\[
\text{Where;} 
\]
FEL is the Fatigue Endurance Limit

$S_{mix}$ is the stiffness of the mix, and

$k_1$ is the adjustment constant for differences in rest periods, or confidence levels

$k_2$ is the mix adjustment factor

It was found that confidence limits could be incorporated into the modelling approach by the use of the $k_1$ adjustment factor and confidence limits could be determined from the Australian validation sites as shown following.

<table>
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<tr>
<th>Confidence Level</th>
<th>80&lt;sup&gt;th&lt;/sup&gt;</th>
<th>85&lt;sup&gt;th&lt;/sup&gt;</th>
<th>90&lt;sup&gt;th&lt;/sup&gt;</th>
<th>95&lt;sup&gt;th&lt;/sup&gt;</th>
<th>97.5&lt;sup&gt;th&lt;/sup&gt;</th>
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<tbody>
<tr>
<td>$k_1$</td>
<td>1</td>
<td>0.97</td>
<td>0.925</td>
<td>0.86</td>
<td>0.8</td>
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The analysis of the results showed that the design of LLAP can be supported by a fundamental test using the Ratio of Dissipated Energy Change (RDEC) and the constant Plateau Value ($PV_L$). However, a laboratory to field conversion is needed to use the test which is equivalent to the confidence values shown previously.

It is recommended the stiffness-FEL design approach be incorporated into a multi-season design approach. The current design recommendation for Australia to use:

- 9.0t design axle (8.2t may be used with a shift in the equation)
- Model using 3 seasons; Summer, Mid temperature (WMAPT), Winter.
- Mid layer asphalt temperatures be taken from the ARRB modified Bells equation

By using this approach the limiting thickness of asphalt pavements can be obtained with confidence for the full spectrum of circumstances encountered on Australian projects.
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1 Introduction and Objectives

1.1 Background

Fatigue cracking of an asphalt pavement, along with roughness and rutting, is one of the most common forms of pavement distress. Fatigue cracking in asphalt pavements manifests itself as a series of interconnected cracks resembling the skin of a crocodile and as such, is often referred to as crocodile cracking. This cracking is caused by a fatigue failure of the asphalt mix from repeated bending of the asphalt under traffic loading. The repeated bending of the asphalt layer(s) in the pavement results in elongation (tensile strains) at the base of the asphalt layer and the established theory states that each cycle of this bending results in un-recoverable damage to the asphalt mix. The sum of this repeated damage eventually results in the loss of structural integrity of the asphalt, and at a critical point in the damage process, the formation of a crack. Once the crack is formed the continual action of traffic propagates it through the asphalt resulting in fatigue cracking of the asphalt mix. However, recent studies show that not all loading to asphalt pavements may induce damage and if strains are kept low enough this cracking may not occur. Even after decades of research the modelling of fatigue performance of asphalt mixes is far from an exact science and a better understanding would enable significant advances in pavement design and construction.

To accurately predict the response of a pavement to loading and therefore the subsequent performance, the fundamental material characterisation property, modulus or stiffness, must be representative and able to characterise an asphalt mix across the full range of temperatures and loading speeds experienced by the pavement. Internationally, the Dynamic Modulus (DC-CY) test has gained widespread acceptance for the characterisation of asphalt mixes across the full temperature frequency range. One of the principal advantages of the dynamic modulus test is the broad database which has been and is continually being developed to link laboratory results to actual field performance (modulus and strain), particularly at the NCAT test track. As with all modulus tests there are a number of issues with the conversion of laboratory results to field response, namely; time inter-conversion, stress susceptibility, effective temperature, and loading pulse width, none of which had been fully validated.

The current Austroads pavement design procedure (AGPT02 (2012)) (and most mechanistic procedures throughout the world) uses conventional theory that assumes every cycle of loading to a pavement does damage to the asphalt layer(s) and therefore uses a proportion of the asphalt’s life. However, as previously mentioned, recent studies have shown that asphalts have a Fatigue Endurance Limit (FEL) and that if strains remain below this limit, the healing potential of the asphalt will, at some time, become greater than the damage sustained within the loading cycle. If the strains within the asphalt layers are kept below this FEL the asphalt can withstand an extremely large number of loading cycles, often referred to as infinite, without experiencing fatigue failure.

The concept of a FEL was introduced in metals by Wohler in the 1870’s. Wohler defined the FEL as the stress level below which fatigue failure did not occur. In pavement design the concept is not
new with the design of concrete pavements incorporating a FEL, expressed as a minimum strength to stress ratio. In asphalt pavements the concept was introduced by Monismith and McLean (1972) who found that there appeared to be a strain level below which damage to an asphalt mix does not appear to occur.

Field verification of the FEL and the resulting Long Life Asphalt Pavement (LLAP) was found to exist in the United Kingdom by Nunn (2001) who observed the performance of a number of sites across UK between 1984 to 1997, and found that pavements which should have been weakening with traffic loading (as per the conventional theory), showed no signs of structural deterioration. Contrary to the conventional theory, Nunn found that "the great majority of the thick pavements examined have maintained their strength or become stronger over time." Subsequent studies by US researchers such as, Mahoney (2001) and Powell (2010) also found LLAP indeed existed most likely due to the presence of the FEL.

Over the past decades a number of test protocols have been developed to determine the fatigue behaviour of asphalt mixes. The accuracy of these test procedures in being able to predict the fatigue performance of an asphalt mix in the field depends on how accurate the test procedure replicates the field conditions of support, stress state, environment, loading conditions and frequency. In Australia the most common fatigue test is the 4 Point Bending (4PB-PR) beam fatigue test. Even though this test has been in existence for 20 years there are still significant difficulties in simulating actual field conditions with this test, some of which were identified by Denneman (2013). For this reason a shift is required between laboratory results and field performance. However, as found by Harvey (1997) the shift factor varies according to a number of conditions and Harvey reported shift factors ranging between 10 and 100. Research by Thompson et al. (2006) found that the wide variety of shift factors was due to the constrained model form used in the conventional fatigue analysis and concluded that a global shift could not exist. While a shift factor is known to exist, the current AGPT02 (2012) offers no recommendations on calculating or applying the shift factor for the purpose of pavement design, most likely due to the complexity of recommending one. Clearly, the prediction and modelling of the fatigue performance of an asphalt mix is difficult and a simpler design approach would be to simply design fatigue out of asphalt pavements by using the FEL concept.

The realisation that LLAP exist and there may indeed be an FEL for asphalt mixes, has led researchers to examine and develop mechanistically based models and test methods for the prediction and incorporation of the FEL into pavement design. Thompson and Carpenter (2006) showed that the performance of a number of asphalt mixes at low strains was distinctly different from that at larger strain levels and that at small strain levels, any small decrease in the strain level resulted in a very large increase in fatigue life. Based on this testing Thompson and Carpenter developed a method for the prediction of the fatigue performance and the FEL of an asphalt mix based on laboratory testing using the Ratio of Dissipated Energy Change.
Further research undertaken as part of the NCHRP 9-44 study (2013) by Witczak et al. confirmed the existence of the FEL and showed that the FEL of all mixes varied with both temperature and length of the rest period, with the FEL increasing with increasing rest periods. It was hypothesized that this increase in the FEL was directly related to the healing potential of the asphalt mix. The 9-44 study recommended an alternative approach for the determination of the FEL based on the work done by Schapery's (1978) elastic-viscoelastic correspondence principle and the pseudo stiffness and Healing Index (HI) approach.

In addition to simplifying the design approach, if the FEL could be incorporated into pavement design it will become an important parameter to determine the limiting thickness of the asphalt layer, beyond which any increase in the thickness of the asphalt layers results in no increase in the structural capacity of the pavement. To incorporate the FEL into LLAP design, most researchers have recommended design approaches based on the use of a single FEL, as originally recommended by Monismith (1972). However, the use of a single FEL is contrary to the most recent research undertaken as part of the NCHRP 9-38 (2012) and 9-44 (2013) projects, both of which found the FEL is not fixed and changes with changes in both mix properties and temperature. These researchers found that stiffness could be used as a surrogate for the effect of temperature and mix properties on the FEL and that the FEL was directly related to the stiffness of the mix, with the 9-44 study recommending the use of stiffness-FEL relationships for LLAP design.

Understanding the significant limitations in applying a single FEL for the purpose of design, the NCAT researchers developed an alternative LLAP design approach, the Cumulative Distribution of Strain (CDS) concept. The CDS approach was unique in that it allowed for variability in FEL in practice, with the approach in reality controlling the strains so only a limited number of loads exceed the FEL of the mix.

Clearly, significant sustainability advantages can be achieved in Australia by designing LLAP. Realising this, AAPA undertook the Asphalt Pavements Solution for Life (APS-fL) project and developed a detailed plan to explain and validate a mechanistic design procedure for the incorporation of the FEL into pavement design. In particular, the procedure needed to consider the Australian environmental conditions and incorporate the effects of the increased healing potential of Australian mixes due to the higher pavement temperatures experienced in Australia relative to much of the rest of the developed world.

1.2 Need for the Development of a Long Life Asphalt Pavement Design Procedure

The value of an asphalt pavement is not the total thickness of the asphalt layers, nor the lowest strain or deflection in the pavement. It is the serviceability the customer realises from the pavement throughout its life, relative to what they pay for the pavement. A pavement is not quality because it is thick, contains polymers or costs a lot of money. If the current Australian design procedure produces asphalt pavements which contain asphalt which offers little value to the customer, there is a loss of value, sustainability and waste of resources in the solution.
This potential loss of value was recognised by clients, industry and consultants with the 2011 AAPA Master Class raising concerns that the current Australian design procedure produces overly conservative asphalt pavement designs, particularly at high temperatures and high loadings, leading to suboptimal value in the pavement solution.

At that 2011 Master Class, a number of issues were raised which were believed to possibly contribute to the overly conservative designs being produced in Australia. The issues identified were:

1. limited data on material characterisation of Australian mixes resulting the use of overly conservative material properties,
2. lack of recognition of the effect of confining stress when characterising mixes at low speeds and high temperatures,
3. the lack of a calibrated and validated shift factor between laboratory observations and field performance,
4. the lack of incorporation of the healing mechanism in asphalt mixes
5. discrepancies in the modelling of the fatigue potential of asphalt mixes at higher temperatures, and,
6. the lack of recognition of a `threshold` strain or FEL below which no damage occurs.

If some or all of these concepts could be incorporated into the current pavement design procedure, more efficient design could be developed. In particular, it was felt that if the FEL could be incorporated into a pavement design procedure, Australia could design LLAP, meaning the limiting thickness of asphalt could be determined, beyond which any increase in design thickness will result in no increase in the structural capacity of the pavement.

1.3 Research Objectives

This research was undertaken as part of the APS-FL project and the main objective of this phase of the APS-FL project was the development of a LLAP design procedure for Australia. To achieve this goal, an extensive experimental plan was developed. This phase of the study consisted of 4 main tasks and a number of subtasks for each as shown following.

 Task 1 Development of Dynamic Modulus Database. Task 1 consisted of a material characterisation study, undertaken to provide real data on the performance characteristics of actual standard Australian production mixes. To be consistent with the direction being taken internationally, the experimental design of the project was developed with an ultimate goal of developing a set of dynamic modulus master curves for real Australian production mixes. The primary advantage of the development of master curves is that the curves can be used for the determination of modulus and visco-elastic properties of Australian asphalt mixes across the full spectrum of temperature and load speeds relevant to Australian field conditions. To accomplish this the following subtasks were undertaken:
o A comprehensive laboratory testing program to characterise typical Australian Asphalt mixes using the Dynamic Modulus test of a full range of Australian production asphalt mixes.

o Development of a computerised database summary of these test responses and mix characteristics.

o A comparison of the material properties of Australian mixes against US and European mixes to confirm similar performance and therefore the transferability of the results from the US and Europe to Australian conditions.

o Recommendation of a modeling method for the prediction of dynamic modulus based on primary material properties (binder type, aggregate size) for the purpose of pavement design.

Task 2 Laboratory to Field Modulus Inter Conversion. At the time of undertaking the study, there was no recommended method found in the literature for the conversion of laboratory determined dynamic modulus to strains under a moving vehicle which had been validated against actual field measurements. Without an accurate prediction of strain, the calibration of any FEL model would be difficult, if not impossible. Fortunately, the NCAT track has a significant amount of data which was used to determine if an inter-conversion was possible between laboratory dynamic modulus and field stiffness and subsequent strain. To establish this inter-conversion the following sub-tasks were undertaken:

- Establish if any valid and accurate correlation/relationship exists between laboratory modulus and field stiffness using laboratory results and field measurements from FWD testing at full scale test tracks. (NCAT, MnRoads and Westrack)

- Determine the effect of stress susceptibility on the measurement of modulus and recommendation of how to consider stress susceptibility in the modeling of field response.

- Development of a quantitative method that can be used to inter-convert between laboratory modulus and field stiffness for the prediction of strain under a moving vehicle, using the relationship developed between laboratory modulus and field stiffness and strain/stress pulse relationship.

Task 3, Relationship between Dynamic Modulus and Current Australian Test Methods. Australia has a number of characterisation tests for the determination of asphalt modulus which are different from the dynamic modulus test used in this study and produce different modulus results. These differences have the potential to create confusion and errors in design. In order to not to lose existing experience and to add to the robustness of the project, an additional task was added to the study, to inter-convert the dynamic modulus test with the current Austroads test methods, namely:

- Development of an inter-conversion between the dynamic modulus test and the resilient modulus test conducted at 25°C and with a rise time of 0.04sec.
Development of an inter-converted between the dynamic modulus test and the flexural modulus test undertaken at 20°C and 10Hz.

And as a consequence the interconversion between the resilient and flexural modulus test.

Task 4 Recommendation of Method for Incorporation of FEL into Pavement Design.
Once an accurate prediction of pavement response had been established, it was then possible to calibrate mechanistic models for the prediction of structural damage, (or lack of in the APS-FL project), through calibration and validation of a FEL model against field performance. In the case of the APS-FL project, the procedure developed ensures the pavement response remains below FEL to ensure little to no damage occurs to the pavement. The objective of the research was to develop a procedure based off laboratory investigations and calibrated using full scale test track data and finally, validated using real in-service asphalt pavements. The resulting procedure developed is then able to model the effects of changes in asphalt mix properties (stiffness) and most importantly for Australian conditions, the increase in the FEL which is known empirically and has been subsequently confirmed in laboratory testing to increase with increasing pavement temperature. To accomplish this a number of sub sub-tasks were developed and undertaken, namely:

- Development of a modeling procedure for the incorporation of a FEL in pavement design based off the examination of both laboratory analysis and field performance of LLAP at the NCAT test track.
- Calibration of the modeling procedure developed by the examination of the NCAT test track data based on the performance of real LLAP sections on the NCAT test track.
- Validation of the modeling procedures based on in service Australian LLAP and time series data obtained from real United Kingdom LLAP pavement sections.

1.4 Report Organisation

The content of this report is divided into 9 chapters. The description of these chapters is as follows.

1) Chapter 1, Introduction and Research Objectives. As already observed Chapter 1 is intended to outline the research background, objective and scope of the research.

2) Chapter 2, provides a literature review and the theoretical background of:
   - Conventional fatigue analysis
   - Visco Elastic Theory
   - Endurance Limits and healing characterisation
   - Laboratory estimation of FEL
   - LLAP asphalt pavements
   - LLAP material selection
   - Field observation of LLAP
   - Design of LLAP
3) Chapter 3, documents the major scope of the comprehensive laboratory experimental testing program undertaken on production mixes from actual projects across Australia. The summary of the master computerised database of the dynamic modulus testing is shown in Appendix A.

4) Chapter 4, provides analysis of the dynamic modulus results, the production of master curves and recommendations on the use of the dynamic modulus results for pavement design.

5) Chapter 5, develops an inter-conversion between laboratory modulus results and field stiffness based off FWD testing at the NCAT, MnRoads and Westrack test tracks. These results are then used to develop a method for modeling the effect of vehicle speed on frequency to predict strain under a moving vehicle.

6) Chapter 6, recommends a method for the conversion of dynamic modulus results to the current Australian test methods, flexural and resilient modulus.

7) Chapter 7, documents the development of FEL design approach based on laboratory data and full scale test track data. This chapter examines the use of the single FEL, cumulative distribution of strain and the use of modulus based FEL for the purpose of pavement design.

8) Chapter 8 uses the recommendation of Chapter 7 and reviews and validates the recommendations based upon the results of actual field performance of pavements in both Australian and UK.

9) Chapter 9, brings the previous research findings together and documents a proposed supplement to the Austroads Pavement Design Guide.

10) The concluding chapter, Chapter 10, provides a summary of the major findings of the research study and provides recommendations for use of the research findings as well as recommendations for future research.
2 Literature Review and Theoretical Background

This chapter is divided into five major sections. In the first section, an introduction to LLAP is provided, followed by a discussion of conventional fatigue analysis of asphalt mixes. In the third part the phenomenological Linear Visco-Elastic (LVE) theory is presented. Next, the healing and the endurance limit concepts are presented followed by a discussion of laboratory methods for establishing the FEL. This is followed by a review of LLAP performance, material selection and field observations. Finally, the design procedures currently available for LLAP design are presented.

2.1 Introduction to Long Life Asphalt Pavements

The concept of Long-Life Asphalt Pavements (LLAP) (long lasting, extended life, or perpetual) was developed in both the United Kingdom and the United States in the early 90’s and now is well recognised internationally. In Australia, this concept is recognised by owners, industry and consultants alike, with the 2011 AAPA Master Class agreeing that there was a FEL and the FEL was an important parameter needed for future pavement design in Australia. Throughout the world there are two primary definitions for LLAP pavements, principally originating from the UK and US definitions respectively;

- a pavement which gets stronger with time (i.e. deflections reduce)
- or, a pavement which does not structurally crack.

Whether designs are produced by either definition is a pure theoretical debate as both definitions produce the same effect, which is a long-life pavement.

Sharp (2001) postulated the existence of LLAP in Australian which was subsequently confirmed by Rickards et al. (2010) by the examination of the LTPP sites across Australia, Rickards concluded, The empirical evidence from Australia, the US and UK, suggest that the continual application of our current design models for thick asphalt pavements, results in overly conservative and wasteful designs and some designs appear long-life.

The performance of LLAP pavements can be explained by the presence of either, or both, a Fatigue Endurance Limit (FEL) and thermal healing and in most cases both are combined to a single FEL (with fixed healing for a rest period). Regardless, of the explanation of the mechanism, the design objective is the same, to design an asphalt pavement structures where at some point in the life of the pavement, the rate of healing becomes greater than the rate of damage and therefore macro cracking will never occur and as a consequence no structural maintenance will be required and maintenance will be limited to periodic resurfacing. The mechanism for both thermal healing and FEL are described following:

- Thermal healing is the ability of a material to self-recover its mechanical properties (stiffness or strength) to some extent upon resting due to the closure of micro cracks.
Threshold strain or FEL is the strain level below which result in no damage occurring to the asphalt and the asphalt mix tends to have an extraordinary long fatigue life.

2.2 Conventional Fatigue Characterisation

2.2.1 Fatigue Cracking of Asphalt Pavements

Fatigue cracking of an asphalt pavement is one of the most common forms of pavement distress along with roughness and rutting. The appearance of fatigue cracking is a series of interconnected cracks resembling the skin of a crocodile and as such, is often referred to as crocodile cracking. This cracking is caused by the fatigue failure of the asphalt mix from repeated bending of the asphalt under traffic loading.

The repeated bending of the asphalt layer(s) in the pavement results in elongation or tensile strain at the base of the asphalt layer. Conventional theory states that with each bending cycle some of the elongation is continually un-recoverable and damage occurs to the asphalt mix. The sum of this un-recoverable damage eventually results in the loss of structural integrity of the asphalt and at a critical point, the formation of a macro-crack. Once this crack is formed the continual action of traffic propagates the crack through the asphalt material resulting in fatigue cracking of the asphalt mix, as shown in Figure 1 following.

![Figure 1 Typical Fatigue (Crocodile) Cracking](image)

Even after decades of research the modelling of fatigue performance of asphalt mixes is far from an exact science and any better understanding will enable significant advances in both pavement design and construction. Over the past decades a number of test protocols have been developed for determining the fatigue behaviour of asphalt mixes. The accuracy of these test procedures in being able to predict the resistance of an asphalt mix in the field to fatigue depends on how accurate
the test procedure replicates the field conditions of, support, stress state, environment, loading conditions and frequency. In Australia the most popular fatigue test is the beam fatigue test. Even though this test has been in existence for over 20 years, as described by Molenaar (2013) and others, significant gaps between the laboratory test method and actual field conditions and performance have been identified. To account for these gaps a “shift” factor is often recommended between laboratory testing and field performance. However, as identified by Harvey (1997) the shift factor varies according to a number of conditions and shift factors of between 10 and 100 have been reported. The same finding has been reported by numerous other researchers such as Roque (2006). Roque also found trend reversals between laboratory performance and field performance (i.e. good performance in the laboratory and worse performance in the field), and aged materials giving better performance than un-aged. While the shift factor is recognised in AGPT002 (2012), there are no recommendations on calculating or applying the shift factor for the purpose of pavement design.

2.2.2 Conventional Fatigue Mechanistic Analysis

In conventional Mechanistic-Empirical pavement design, the fatigue of asphalt mixes is based on the principle that the repeated cyclic strain (at the underside of the asphalt layers as a result of bending) ultimately results in cracking and fatigue failure of the pavement. To predict the life of the pavement before this cracking occurs, most mechanistic design methods make use of a theoretical damage relationship to ensure the magnitude of the strains are controlled below a critical level.

For the structural design of asphalt pavements the conventional model for the prediction of the fatigue life of an asphalt layer(s), is a straight line (log scale) relationship between the tensile strain and the number of allowable number of load repetitions to failure (Nf) (strain-Nf). In Australia (as in the majority of the word), failure is commonly taken as a 50% reduction in initial modulus and is shown in Equation 1, following:

\[ \varepsilon = k N_f^n \]

Where;

\[ N_f \] is the number of load cycles to failure,
\[ \varepsilon \] is the tensile strain, in micro-strain at the outer fibre of the asphalt mix,
\[ k \text{ and } n \] are regression constants from the laboratory testing.

Because of the phenomenological nature of this relationship (relationship built on observations), most researchers propose that an adjustment factor be applied to obtain a “better fit” with observed results. The most notable adjustment addition is the addition of a modulus term, as shown in Equation 2 following, which is usually applied when multiple mixes or temperatures are being included, as in the AGPT002 (2012).
Equation 2

\[ E_0 \]

Where;

\[ E_0 \]

\[ C \]

\text{is} the initial modulus of the mix,

\text{is} a regression constant from the laboratory testing.

Thompson et al. (2006) found that the fundamental nature of the strain relationship accounted for 90% of the prediction in the data and the addition of the modulus term accounted for the remainder, and for the inadequacies of a non-fundamental relationship. Thompson et al. identified the main failing of the current model form was the inclusion of mix variables primarily in the \( k \) term and the use of a constant \( n \) value, regardless of the asphalt materials or test temperature. Thompson stated that the use of this model form ignores the vast majority of test data which has shown that \( k \) and \( n \) are integrally related in a consistent manner. This consistent relationship was found by Thompson et al. (2006) and the same relationship is observed in Australian data, as shown in Figure 2 following. This figure illustrates the relationship between \( n \) and \( k \) based off the results of over 100 different mixes from Fulton Hogan database undertaken over the past 20 years.

In the data set air voids varied from 4 to 7 percent. Nominal maximum aggregate size has varied from 10mm to 28mm. Asphalt binders varied from C170 to C600, with both neat and polymer modified (SBS, SBR, EVA) materials being used.
As found by Thompson et al. (2006) on US mixes, all of the Australian mixes establish a consistent relationship between $k$ and $n$. Given this strong relationship, any model that fixes $n$ will be incorrect. Thus the outcome of the use of the $k$ and $n$ relationship (Equation 1) will be overshadowed by the use of a constant $n$ relationship, so any attempt at national calibration would not be expected to be indicative of the performance found in varying regions of Australia. It is not surprising therefore, that the variable shift factors are consistently obtained in laboratory to field calibrations (Harvey (1997)).

The other main failing of the classical model is the assumption that every cycle does damage. This assumption ignores the presence of relaxation, healing and the FEL concept. If these parameters are included in the classical model, bottom-up fatigue cracking, which the classical models predict, can be avoided, not just designed for.

### 2.3 Viscoelastic Theory

Any study on the characterisation of asphalt mixes including stiffness, healing and the FEL, needs to be undertaken with a background understanding of viscoelasticity. Whereas, pavement structures are commonly assumed to be linear elastic materials in design, the effect of time and temperature, the amount of damage, relaxation and healing of the asphalt mixes are all related to its viscoelastic nature.

Fundamentally, viscoelastic materials are those where the relationship between stress and strain depends on time. Due to this time dependent behavior, some of the properties of viscoelastic material are:

- If the stress is held constant, the strain increases with time, (creep).
- If the strain is held constant, the stress decreases with time, (relaxation).
- The effective stiffness or modulus depends on the rate of application of the load.
- If cyclic loading is applied, a phase lag (hysteresis) occurs, leading to a dissipation of mechanical energy.

Unlike purely elastic materials, a viscoelastic material has both an elastic component (spring) and a viscous component (dampener). It is the viscous component of a viscoelastic material that results in the strain response being dependent on time and is why they are commonly referred to as time dependent materials. The time dependent behaviour of viscoelastic materials is commonly explained by constitutive equations, which include time as a variable in addition to stress and strain.

#### 2.3.1 Dissipated Energy Due to Load

When a load or stress is applied to a material, the material will deflect under that load (strain), the resulting area under the stress-strain curve is the energy applied to the sample. For a purely elastic
material (such as most metals in the elastic region) when the load is removed from the material, the deformation is fully recovered. If the loading and unloading curves overlap, all of the energy put into the material is recovered and the sample returns to its original state, as is the case for elastic materials.

However, for viscoelastic materials the two curves do not coincide, as shown conceptually in Figure 3 following, and energy is lost in the material. This energy can be lost through mechanical work, heat generation, or damage. Because energy is lost, the material will not fully return to its original shape and some permanent deformation is observed. This energy difference between the loading and unloading curve is the dissipated energy of each loading cycle, Ghuzlan (2001).

![Figure 3 Typical Stress Strain Curve for Visco-elastic Solid](image)

In damage analysis of asphalt mixes it is a common assumption that all of the dissipated energy went into damaging the material. However, due to the viscoelastic nature of the asphalt materials, a hysteresis loop will always be created, even when there is no damage to the asphalt, and in the loading and unloading cycle only part of the total dissipated energy will go into damaging the material, with the remainder due to viscoelasticity, heating and other factors.
Manfredi (2001) demonstrated that not all dissipated energy goes into damaging the sample. Manfredi showed experimentally that energy dissipated during plastic cycles did not contribute to damage and should be excluded from the total energy. Hilton et al. (1992) found that part of the dissipated energy was converted to thermal energy through viscoelastic damping. Additionally, any source of release energy (which is still part of the overall dissipated energy) which does not go into crack formation and propagation should be eliminated from the total dissipated energy calculation for predicting damage.

For all linear viscoelastic material, such as asphalt, the equation for calculating dissipated energy per cycle in cyclic flexural fatigue test is given by Equation 3 following, (Tayebali et al., 1994):

\[ \Delta \varepsilon = \frac{\Delta \sigma}{E} \]

Where:
- \( \Delta \varepsilon \) is the dissipated energy at load cycle \( i \)
- \( \Delta \sigma \) is the stress at the load cycle \( i \),
- \( E \) is the strain at the load cycle \( i \), and
- \( j \) is the phase angle between stress and strain at load cycle \( i \).

2.3.2 Dissipated Energy and Fatigue

Various representations and applications have been proposed to relate fatigue life of asphalt mixes to energy. The initial work on energy used the total dissipated energy as either initial dissipated energy or used a cumulative dissipated energy approach. While these methods provide sound mechanistic relationships between stress, strain, energy, and fatigue life, and can be applied under a wide variety of environmental factors, reliable prediction of fatigue life cannot be predicted without extensive fatigue testing.

To determine the fatigue life from dissipated energy, fatigue tests need to be conducted where the phase angle, mixture modulus, and dissipated energy are measured throughout the test. Several mechanistic parameters are then calculated and used to relate fatigue life to dissipated energy by the Equation 4 following:

\[ W = A \cdot Z \cdot \Delta \varepsilon \]

Where:
- \( W \) is the Total dissipated energy (sum of dissipated energy found from Equation 3)
- \( A, Z \) are Mixture characterisation constants
The cumulative dissipated energy approach was researched by Van Dijk and Visser (1977) who found that the fatigue behaviour of asphalt mixes from different tests, loading frequencies and temperature conditions could be described by a single mix specific relationship. Van Dijk and Visser found that a single relationship existed between the number of cycles to fatigue failure (Nf) and the cumulative amount of energy dissipated during the fatigue test. It was found that all the variables, including rest period, mode of loading, temperature, and frequency, did not significantly influence this dissipated energy relationship. The researchers found that the slopes of the fatigue lines for different mixes were nearly all the same and similar to the 0.67 slope suggested by Chomton and Valayer (1972). However contrary to this finding, other researchers have found that the relationship was mix dependent (Van Dijk et al. (1972) and S HRP A-404, (1994), with the University of California (Berkley) study under S HRP-A-404, (1994) finding that all energy-Nf fatigue lines were not parallel and have different slopes, similar to the conventional model form.

In an attempt to account for the reality that not all dissipated energy goes into damaging the asphalt, various alternatives to the initial dissipated energy and cumulative dissipated energy approach have been developed such as; work ratio approach, dissipated energy approach and ratio of dissipated energy change. Of these, the dissipated energy change approach has been found to provide promising results in the prediction of both the fatigue and FEL of an asphalt mix and will be discussed in depth in section 2.5.2.

2.3.3 Relaxation and Creep

Relaxation modulus is generally considered the fundamental material property that determines the strain (or stress) development in flexible pavements and is needed not only for the characterisation of the viscoelastic behaviour of asphalt mixes but also the characterisation of material damage where it exhibits non-linear behaviour. However, the measurement of relaxation modulus is difficult and other tests are commonly used such as, creep, stress relaxation and dynamic modulus testing.

Jaeseung et al. (2008) found that creep compliance or complex modulus tests alone were not capable of providing complete information over the typical time or frequency range used in single-temperature tests. It was found that in general, the dynamic modulus test provides accurate creep compliance at short loading time, while the creep compliance test provides accurate creep compliance at longer loading time.

Relaxation modulus can be determined from either a creep compliance test using static loading or a complex modulus test using cyclic loading. Since the nature of each test is different, creep compliance determined from the complex modulus test can be different from that determined from the creep compliance test.

The relaxation modulus, $E(t)$, is defined as the stress response of a viscoelastic material due to a unit step of strain input. The relaxation modulus can be calculated as the time-dependent stress divided by the applied strain level. For a linear material, as with the creep compliance, relaxation
curves obtained at different strain levels can be superimposed by defining the relaxation modulus as shown in Equation 5 following:

\[
E(t) = \frac{s(t)}{e_0}
\]  
Equation 5

Where;

- \( E(t) \) is the relaxation modulus
- \( s(t) \) is the time-dependent stress, and
- \( e_0 \) is the constant applied strain.

At short loading times, the modulus is at a high plateau level (glassy or elastic modulus) and falls to the equilibrium (rubbery) modulus at longer times.

2.3.4 Dynamic Modulus Testing

While creep and relaxation tests are suitable to characterise material responses over long times, they are however less accurate for shorter times. It has been found by numerous researchers such as Katicha (2007), that the dynamic tests are more suitable to describe the short-term response. When a viscoelastic material is subjected to a cyclic stress, a steady state is reached in which the resulting strain is also cyclic, with the same frequency but lagging behind the stress, this is the phase lag (\( \phi \)).

Dynamic modulus is such a test which is applicable to characterise short-term response of an asphalt mix due to its inherent viscoelastic nature. The dynamic modulus test is undertaken by applying a vibratory load (stress) and measuring the resulting displacement (strain). Unlike purely elastic materials, where the stress and strain occur in phase to each other, due to the viscous component in asphalt there is a lag between stress and strain or a phase lag (\( \phi \)). Under a continuous, sinusoidal loading (vibratory) the response of the asphalt can be defined by a complex number, called 'complex modulus' (\( E^* \)). The absolute value of this complex modulus is defined as the Dynamic Modulus (\( |E^*| \)).

Mathematically, the response of the asphalt to the vibratory stress can be represented using the following equations:

Strain :
\[
\text{Equation 6}
\]
Stress :
\[
\text{Equation 7}
\]

Where,

- \( w = 2\pi f \) and \( f \) is frequency of stress oscillation,
- \( t \) is time,
- \( d \) is phase lag between stress and strain.
There are two components to Dynamic Modulus: The storage and loss modulus, the stored energy, representing the elastic portion, and the energy dissipated as heat, representing the viscous portion. The storage and loss moduli are defined as follows:

\[
\text{Storage modulus: } \frac{\partial \varepsilon}{\partial \sigma} = \text{Equation 8}
\]

\[
\text{Loss modulus: } \frac{\partial \sigma}{\partial \varepsilon} = \text{Equation 9}
\]

Complex variables can then be used to express the Complex Modulus \( E^* \) as follows:

\[
\frac{\partial^2 \varepsilon}{\partial \sigma} = \text{Equation 10}
\]

Where;

\( i \) is the imaginary unit

And Dynamic Modulus as, the absolute value of stress and strain:

\[
\frac{\partial^2 \varepsilon}{\partial \sigma} = \text{Equation 11}
\]

2.4 Fatigue Endurance Limits in Asphalt

2.4.1 Existence of Endurance Limits

The concept of the asphalt FEL was first postulated in the early 1970's by Prof. Carl Monismith and published by Monismith and McLean (1972). These researchers found that there appeared to be a strain level below which damage to an asphalt mix does not appear to occur. Since that time considerable research has continued and while there is widespread acceptance of the laboratory FEL concept its relationship to mix variables and temperature is still not fully understood and its incorporation into pavement design is not common place.

The FEL concept was further investigated by Carpenter at the University of Illinois from the early 2000's, Carpenter et al. (2000), and more recently by researchers at NCAT and under the NCHRP 9-38 (2012) and 9-44 (2013) projects. In the early studies by Carpenter (2000), and Thompson (2006), the FEL was determined using a combination of standard fatigue tests at high strain levels and by conducting extremely lengthy fatigue tests at low strain levels. The resulting strain-Nf relationships obtained by Carpenter et al (2006) are shown in Figure 4 following. The researchers found a consistent relationship for all mixes, being the traditional straight line strain-Nf relationship followed by a near horizontal slope and for all the mixes tested in the study, none deviate from this typical result.

Theses FEL studies found that there was a definite point for each mix, where the response deviated from the conventional straight line (log scale) strain-Nf relationship. After this point the slope of the line for all practical purposes becomes flat. The flat slope indicated that at lower strains asphalt
mixes can produce extremely long fatigue lives, often referred to as “infinite.” It was defined by Thompson that this transition point was the FEL of the mix. At a strain level below the FEL the mix will begin to show an extraordinarily long fatigue life, significantly higher than those predicted by the traditional straight line fatigue model.

Examination of the results from by both Thompson et al. (2006) and the testing undertaken as part of NCHRP 9-38 (2012) found that none of the mixes tested had strains lower than 70mε to achieve the transition to the virtual flat slope. Both Thompson et al. and the 9-38 study found that depending on the binder type, this extended transition point was achieved at significantly different strain levels.

As can be seen on Figure 4 following, Thompson found that there was not a single FEL for asphalts mixes and there was a range of FEL, which varied according to the mix tested. Thompson found the difficulty in differentiating the mix variables and their impact on the FEL derives from the use of the phenomenological relationship for strain and loads to failure. Because this relationship is not fundamental it cannot adequately describe mix performance under varying inputs.

Following on from the work of Thompson an extensive research study was carried out under the NCHRP project 9-38 in 2010 to investigate, the existence of FEL in asphalt mixes, the effect of asphalt mix characteristics on the FEL, and the potential for the FEL to be incorporated into structural design of flexible pavements. The results of the study were published as the NCHRP report 646(2012) and the main findings of this study are summarized following:

- The endurance limit for a long-life pavement was defined as a pavement being able to withstand 50 million design load repetitions in a 40-year design period.
The data supported the existence of an FEL for all the studied mixes. The estimated endurance limit for each of the six studied mixes in this research varied from 75 to 200 and was the one-sided, 95% confidence lower prediction limit that produces a fatigue life of 50 million cycles.

Both beam fatigue and uniaxial tension testing were conducted to determine fatigue life of the mixes. It was indicated that the uniaxial tension testing method provided a promising technique and could be used to more rapidly determine the endurance limit of a mix as it would not require the production of an asphalt beam and instead would use a sample more closely related to those being contemplated for Simple Performance Tests (SPT) for dynamic modulus. However, there were reported discrepancies in the predicted fatigue life trends defined by beam fatigue and uniaxial tension testing which necessitated further evaluation of the uniaxial tension testing results.

The role of the quality of construction along with pavement thickness design and materials selection to achieve a durable LLAP structure was emphasised.

Shift factors ranging from 4.2 to 75.8 based on fatigue transfer functions were introduced to correlate the laboratory and field fatigue performance.

Using the FEL determined from laboratory beam fatigue tests conducted as part of the study and a typical principal arterial traffic stream, the LLAP thickness determined with PerRoad was approximately the same as the 20 and 40 year conventional (no endurance limit) MEPDG or 1993 AASHTO Pavement Design Guide pavement thicknesses. However, the MEPDG perpetual thickness was approximately 50% thicker and the overall conclusions at the end of a 20 or 40 year period were significantly different. With the conventional designs, the pavements would have failed in bottom-up fatigue with cracking over 20% of the lane area at 90% reliability while no cracking would be expected if the endurance limit was considered.

2.4.2 Healing a Source of Endurance Limits

The effect of rest periods and healing potential of asphalt mixes has been investigated by researchers for many years starting with researchers at Shell in the 1960's (Roque 2006). Healing is commonly considered the capability of a material to self-recover its mechanical properties (stiffness or strength) to some extent upon resting due to the closure of cracks. Healing is not confined to asphalt mixes and in fact, a number of materials have been found to possess the ability to undergo healing, including metals, plastics and rubbers.

Current asphalt thickness design procedures ignore healing potential. As a consequence, the conventional analysis suggests the traffic applied at high pavement temperature (in theory) does a disproportionately greater amount of fatigue damage because of the high strain levels, resulting from the low asphalt stiffness. The NCHRP 9-44 (2013) proposed the use of a method for the incorporation of healing into pavement design based on the use of stiffness-FEL relationships for
different rest periods. The application of this model found that traffic at the high end of the temperature spectrum causes relatively less damage than mid-range due to healing effects. It was also apparent that at low temperature the high asphalt stiffness reduced strain more than offsets the reduced healing.

While a significant amount of research has been undertaken on the effect of rest periods on the fatigue life of asphalt mixes, little research has been focused on the actual mechanism of healing. Phillips (1998) proposed that the healing of asphalt mixes is a three-step process consisting of:

1) The closure of micro-cracks due to wetting (adhesion of two crack surfaces together driven by surface energy);
2) The closure of macro-cracks due to consolidating stresses and binder flow; and
3) The complete recovery of mechanical properties due to diffusion of asphaltene structures.

The first process was believed to be the fastest, resulting only in the recovery of stiffness, while the second and third processes are believed to occur at a much slower rate and improve not only stiffness, but also the strength of asphalt mix with properties returned to levels similar to that of the original material.

Jacobs (1995) also found that the introduction of rest periods has a beneficial effect on the fatigue resistance of the mixes. He proposed that this "healing" in the rest periods occurred by diffusion of the low molecular weight component of bitumen, maltenes, through the micro-cracks and re-establishing the bonds in the cracked area. Jacobs believed that the maltenes were involved because they are the most mobile components of the bitumen, although higher molecular weight molecules (asphaltenes) could also diffuse during longer rest periods, and as found by Phillips (1998) could result in completely restored material properties.

Lytton (2005) used the "dissipated pseudo strain energy concept" to explain the fracture and healing process in asphalt mixes. Lytton concluded that the fracture or healing of an asphalt mix was related to two mechanisms:

1. The surface energy storage or the surface energy release, which was related to polar or non-polar characteristic of the binder.
2. The energy stored on or near the newly created crack faces which governs the energy available to make the crack grow. It was found that this surface energy depended mainly on the chemical composition of the binder.

Lytton (2005) concluded that the micro-fracture and healing of the asphalt aggregate mix was controlled by the energy balance per unit of crack area between the "dissipated pseudo-strain energy" released and the energy that is stored on the surface of the crack.

Even when considering healing, debate exists as to when healing occurs; during rest periods, during all the loading and unloading periods, or just under certain conditions such as certain
temperature and material damage levels. It appears that these different conclusions are mainly based on the laboratory test setup used and the research approach adopted. Most researchers now believe that healing occurs in all conditions, just at varying rates (NCHRP 9-44 (2013)) at stiffnesses below the glass transition point.

While the healing concept is only in its infancy in pavement engineering, it is well understood in polymer engineering and a considerable amount of work has been completed on the study of the healing phenomenon of polymeric materials. Prager and Tirrell (1981) described the healing phenomenon:

"When two pieces of the same amorphous polymeric material are brought into contact at a temperature above the glass transition, the junction surface gradually develops increasing mechanical strength until, at long enough contact times; the full fracture strength of the virgin material is reached. At this point the junction surface has in all respects become indistinguishable from any other surface that might be located within the bulk material: we say the junction has healed."

2.4.3 Effect of Rest Periods on Healing

Some of the early work to develop a mechanical approach to quantify the healing potential of an asphalt mix was developed by Kim and Little (1990). In developing this approach they performed cyclic loading tests with varying rest periods on notched beam specimens of sand asphalt and found that the rest period has a significant effect on the enhancement of the fatigue life through healing and relaxation mechanisms.

To explain healing they proposed to use a concept called the Healing Index (HI), which was found to be highly sensitive to binder properties. The researchers then applied Schapery's elastic-viscoelastic correspondence principle (1978) to separate out the viscoelastic relaxation from chemical healing. After separating the relaxation from the healing, the magnitudes of pseudo energy density before and after rest periods were used to calculate the HI. The NCHRP 9-44 (2013) project expanded on the work undertaken by Kim and Little (1990) and used the pseudo energy and HI approach to determine the FEL for a range of mixes. The testing conducted under the 9-44 project consisted of both uniaxial fatigue testing and 4 point bending. The NCHRP project found that the effect of rest periods was a significant factor and could increase the FEL by a factor of 3 at reset periods of greater than 10 seconds relative to a 1 second rest periods. The NCHRP project also found that rest periods of greater than 10 seconds resulted in little increase in healing potential.

2.5 Laboratory Estimation of Fatigue Endurance Limits

For routine determination of the FEL of an asphalt mix, it is impractical to undertake extended laboratory fatigue testing as originally undertaken by Monismith (1972). Consequently, for practical implementation of testing for FEL a method needed to be established where limited cycle fatigue testing could be undertaken and the data extrapolated to establish the fatigue life of the mix. Ideally,
any method of extrapolation should be able to be used to extrapolate the results of any fatigue test whether the test is conducted below, at, or above the FEL of the mix.

2.5.1 Stiffness Ratio Methods

The NCHRP study 9-38 (2010) examined a number of different model forms for the extrapolation of fatigue curves (stiffness vs. N), namely;

- exponential (AASHTO T321),
- logarithmic,
- power, and
- Weibull function.

The study found that the exponential model consistently underestimates the stiffness at 50 million cycles and was slow to converge to the measured stiffness. This means that testing would need to be conducted to a high number of cycles to even approach the measured stiffness.

It was found that both the logarithmic and power models would converge to a reasonable predicted stiffness within 10 million cycles. However, the use of both the logarithmic and power function required a high degree of user input in establishing where to start the curve fitting. The reason for this is that when all of the loading cycles are used (including the primary stage with initial heating phase), the resulting fit would overestimate the stiffness at 50 million cycles and, consequently, would overestimate the fatigue life. To obtain a reasonable fit the primary stage would have to be ignored.

The study found that the single-stage Weibull function converges quickly and provided the most accurate results for one of the samples tested. It was however, found to do a relatively poor job at estimating stiffness for all other samples examined. For long-life fatigue tests conducted at strain levels slightly above the FEL, it was found that, the single-stage Weibull function provided the most accurate extrapolation of fatigue life. However, the three-stage Weibull function was found to provide the best fit to the stiffness versus loading cycle data and was most probably the best method for estimating the fatigue life at strain levels below that of the FEL. The conclusion of the study was that the Weibull functions were the best methods for extrapolating fatigue tests that did not fail within 50 million cycles.

The Weibull function first recommended by Tsai (2002) is shown in Equation 12 following.

\[
\ln\left(\frac{n}{\delta}\right) = \ln\left(\frac{x}{\delta}\right) + \alpha \ln\left(\frac{x}{\delta}\right)
\]

Equation 12

Where;

- \(n\) \(\Rightarrow\) Stiffness ratio or stiffness at cycle \(n\) divided by the initial stiffness
- \(\delta\) \(\Rightarrow\) Number of cycles
To improve the accuracy of the single stage Weibull function Tsai et al. (2002) developed a methodology for fitting a three stage Weibull curve (as used in the NCHRP9-38 study). It was theorised that three stage model fitted the three stages of the loading cycles vs. stiffness stages; initial heating and temperature equilibrium, crack initiation and crack propagation. In the case of testing below the FEL the third stage does not occur and represents a stage of decreasing damage with time.

Sullivan (2015) examined the use of the of the Weibull function for the prediction of both the stiffness curve and Nf of Australian mixes. The results are shown Figure 5 and Figure 6 following, and shows the accuracy of both the one stage and three stage Weibull function for an EME mix close to that of the FEL, while Figure 6 shows the accuracy of the prediction of the number of cycles to failure of a number of mixes using both the one stage and three stage Weibull function over a range of fatigue lives.
It was found that for all results close to and below the FEL the three-stage Weibull function captures the shape of the stiffness curve to a high degree of accuracy and can be used to accurately predict the extended fatigue life of mixes. Instead of using the approach recommended by Tsai et al. (2002) for the calculation of the three stages, Sullivan used the Solver function in MS Excel to solve for the best fit of the three stage model. The results showed that while the one-stage model gives an accurate prediction of the number of cycles to failure for standard test conditions, at the lower temperatures (mixes tested at 10°C) the model consistently over predicted the number of cycles to failure, as the model did not capture the third stage brittle failure. For the three stage model it was found that the model accurately predicts the number of cycles to failure over all strain levels.

Based on the testing and analysis Sullivan et al. recommended the use of three stage Weibull model to estimate the failure points.

2.5.2 Ratio of Dissipated Energy Change

Ghuzlan and Carpenter (2004) used the dissipated energy change approach to predict both fatigue failure and the FEL of an asphalt mix. Dissipated energy is the measure of the energy that is lost through mechanical work, heat generated or damage to the sample in a testing cycle. Some researchers have used the total cumulative dissipated energy to define the damage within a sample by assuming that all dissipated energy is responsible for damage. As dissipated energy may be a function of heat generation or work done within the sample Ghuzlan and Carpenter (2004) postulate that only a proportion of dissipated energy is responsible for actual damage.
The Dissipated Energy Change is different from pure dissipated energy approaches, as it uses the ratio of the amount of dissipated energy change between different loading cycles to represent the damage propagation. The basic premise of this approach is that the change in dissipated energy per cycle of loading is related to the growth of damage that occurs in an asphalt mix.

Shen (2005) improved the dissipated energy approach initially developed by Tayebali et al. (1992) and renamed it, based on the changes made to the approach to the Ratio of Dissipated Energy Change (RDEC). In the recommended approach the RDEC is defined as the average change in dissipated energy between two cycles divided by the dissipated energy from the first of the two cycles. This ratio illustrates the percentage of input dissipated energy which goes into damage for a cycle. The representation of damage produces a U shaped curve as shown following in Figure 7, following.

The RDEC was defined as the average change in the dissipated energy between two cycles relative to the initial two cycles. Shown mathematically by Equation 13 following:

\[
\text{RDEC}_a = \frac{\text{DE}_a - \text{DE}_b}{\text{DE}_a} \times \text{DE}_b
\]

Equation 13

Where:

- \( \text{RDEC}_a \) = Ratio of dissipated energy change for cycle a
- \( \text{DE}_a \) = Dissipated energy for cycle a and
- \( \text{DE}_b \) = Dissipated energy for cycle b.

The basic assumption of the approach is that the change in dissipated energy is directly related to the growth of damage in that cycle.

It was hypothesised by Thompson et al. (2006) that the RDEC approach provides a unique relationship between damage and load cycles to failure (Figure 8) and that there was one unique Plateau Value \((6.79 \times 10^{-9})\) of the RDEC where the behaviour of an asphalt changes from traditional accumulation of damage to a point where damage does not occur or healing potential is greater than damage. This is the balancing point (FEL) between healing and damage in the asphalt mix. It should be noted, that the balancing point in the RDEC approach is for the case of the test conditions undertaken in the testing, which was, 10Hz haversine loading (i.e. no rest periods). Different balancing points should be expected with different rest periods and loading frequencies. It was proposed by Shen (2005) that this point (Plateau Value) be defined as the FEL for all mixes and is denoted as Plateau Value, PV, on the following figures.
As can be seen in the figures along with a unique PV_L, there is a unique relationship between the PV and the cycles to failure. Given the proposed constant PV_L and the unique relationship indicates a constant number of cycles (1.1x10^7) to the transition point in the straight line strain-Nf curve and that the FEL can be estimated from the traditional straight line strain-Nf plot at a constant number of cycles (1.1x10^7), Sullivan et al. (2015) showed that the unique PV_L found for US mixes was applicable and could be used to estimate the FEL of Australian mixes.
2.5.3 Healing Index

Under the NCHRP 9-44 study (2013) fatigue and healing potential (and subsequent FEL) was analysed by using two fatigue tests (either uniaxial tension-compression or flexural bending tests). The first test was conducted under continuous loading condition with no rest periods, the second introduced rest periods between loading cycles. The inclusion of the rest periods decreases the stiffness deterioration through partial healing of fatigue damage. The result is that the stiffness deteriorates at a slower rate compared to the test without rest period, as can be seen in Figure 9 following.

The difference between the tests conducted with a rest period and the test without was defined as the Healing Index (HI), with the concept shown graphically in Figure 9 following, and numerically in Equation 14 following.

$$HI = \left( \frac{SR_{w/RP}}{SR_{w/o RP}} \right)_{at \ Nf_{w/o RP}}$$

Equation 14

Where:

- $SR_{w/RP}$ = Stiffness ratio with rest period
- $SR_{w/o RP}$ = Stiffness ratio without rest period

It was hypothesised that FEL occurred when no damage (no modulus reduction) occurred in a test with rest period. Given that failure in a beam fatigue test is defined as a 50% reduction in modulus,
the definition implies that the FEL could be estimated at a HI of 0.5; which means S R w/o RP = 0.5 (i.e. failed sample) and S R w/ RP = 1.0 (no damage or loss of modulus).

To estimate the FEL, the 9-44 project used both the number of repetitions to failure (Nf) without a rest period and the Nf on a fatigue test with a rest period to develop a Stiffness Ratio model based on fitting regression fitting of the experimental data for both tests, as shown conceptually in Equation 15 following. The S R was determined for both tests with and without rest period and all data points were used to establish the general S R model. With Equation 15 following showing the general form of the S R model based on the six factors, used in Phase 1 of the NCHRP 9-44 project:

\[
S R = a_1 + a_2 AC + a_3 Va + a_4 (BT) + a_5 (RP) + a_6 (T) + a_7 Nf w/o RP + 2-factor + 3-factor interactions
\]

Where:

- S R = Stiffness Ratio
- \(a_1, a_2 \ldots a_n\) = Regression coefficients
- AC = Percent asphalt content
- Va = Percent air voids
- BT = Binder type
- RP = Rest period (sec)
- T = Temperature (F)
- Nf w/o RP = Number of cycles to failure (test without rest period)

Once the S R model was developed, the HI for any test combination could be computed as shown previously in Equation 15. The next step is to correlate the computed healing index to the FEL. To do this all HI data points were plotted versus the strain levels that were used for each test at each temperature separately, since it was expected that different temperatures would have different FEL. This concept is shown conceptually in Figure 10 following, which illustrates a schematic relationship between healing index and strain at each temperature estimated from the S R model, this approach was used to extend the results to any rest period to get at S R and subsequent HI. Then, the FEL limit could then be estimated for any temperature and binder content by assuming a HI of 0.5, again shown conceptually in Figure 10 following.
Further research under the 9-44 project found that the binder terms (BT, AC) and the temperature (T) in Equation 15 could be directly replaced by using the initial stiffness of the mix, with no loss in model accuracy. The use of the stiffness enabled estimation of the FEL for any stiffness and any rest period and the production of stiffness-FEL relationships, which were recommended for pavement design.

2.6 Long Life Asphalt Pavements

In the US, the Asphalt Pavement Alliance (APA), under Newcomb et al. (2010) introduced the concept of LLAP as ‘an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement’. As with the studies of Nunn, the APA found that contrary to conventional theory, many of the full-depth or deep-strength pavements which had been in service for decades were only requiring surface renewals (mills and re-sheets) of the upper surface (Asphalt Pavement Alliance (2002)) and had no signs of structural failure.

Based on these findings the Newcomb et al. found that the use of LLAP could offer significant benefits to the US economy and with the current budget shortage for expansion and rehabilitation of the network and the increasing demand on the road network, the LLAP concept could play an important role in providing a life-time solution to maintain and construct asphalt pavements.

In the US two main schemes have been utilised for LLAP design, full-depth and deep strength pavements, each of which can lead to design of thinner overall pavement structures in comparison to thick granular base pavements. Full-depth pavements consist of asphalt layers placed directly on subgrade material (modified or unmodified) while deep-strength pavements are where the
asphalt layers are placed on top of a thin granular base. It was found by APA that these pavements can exceed their design life with minimal rehabilitation if their cracking potential (surface initiated) could be confined to the upper removable wearing layers Asphalt Pavement Alliance (2002).

In traditional pavements, fatigue cracking and rutting are the two major failure mechanisms of pavements which can occur at or before the end of design life Mahoney (2001). Both deep-strength and full-depth pavements can be designed to be structurally resistant to these distresses. The concept of LLAP evolved from recent attempts of pavement engineers to design pavements which are resistant against bottom-up fatigue cracking and rutting, Newcomb, Willis & Timm (2010). However, according to both the current and previous, mechanistic-empirical and empirical design approaches, the thickness of the pavement increases with the increase in traffic level to resist these modes of failure. Whereas according to the LLAP concept, any thickness of the pavement beyond a certain level may lead to unnecessary added expenses Newcomb, Willis & Timm (2010) and new design approaches are required. As with thickness, there is also no point and a lack of sustainability, in the excessive usage of non-renewable natural resources as a result of over-designing pavements according to the current pavement design procedures. For example, Huber et al. (2009) reported on an over-designed pavement in India by 40-115 mm, which was designed in accordance with the 1993 AASHO pavement design led to waste of 900-3000 tons of material per lane kilometre.

The recent research which has shown that there is a level of stress or strain below which no structural damage will occur to the asphalt pavement, has resulted in suggestions that a pavement can be designed to last indefinitely without any major structural damage, if the pavement is designed and constructed so that no damage occurs under the cyclic traffic load. While no structural damage will occur, surface maintenance and overlays will still be required to keep the pavement in a serviceable condition Newcomb, Buncher & Huddleston (2001), Powell et al. (2010). Ferne (2006) suggested a broader definition for long-lasting pavements: 'long-life pavement is a well-designed and constructed pavement that could last indefinitely without deterioration in the structural elements provided it is not overlooked and the appropriate maintenance is carried out.' Various other definitions of perpetual pavement can be found in the literature all of which indicate almost the same concept; a summary of perpetual pavement definitions are listed following:

- **Transportation Research Laboratory (TRL):**

  Well-constructed fully-flexible pavements designed for 40 years and for traffic in excess of $80 \times 10^6$ LLAP pavements Nunn, Brown & Weston (1997).

- **Asphalt Pavement Alliance (APA):**

  A Perpetual Pavement is defined as an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement, Asphalt Pavement Alliance (2002).
The European Long-Life Pavement Group (ELLPG):

A long-life pavement is a type of pavement where no significant deterioration will develop in the foundations or the road base layers provided that correct surface maintenance is carried out, FEHRL (2004).

This concise definition can be elaborated with the following clarification.

Deterioration: This includes whatever the network manager considers important e.g. significant cracking or (progressive) deformation in the structural layers of a fully flexible pavement; for other types of pavement “deterioration” could be quite different.

Later in 2009 the definition was revised as: A long-life pavement is a well-designed and well-constructed pavement where the structural elements last indefinitely provided that the designed maximum individual load and environmental conditions are not exceeded and that appropriate and timely surface maintenance is carried out, FEHRL (2009).

The World Road Association (PIARC):

A pavement is considered as a “success story” when it has proved to behave better than expected when it was designed. Such a pavement must be clear of structural maintenance and still be in good shape, despite the fact that it has sustained a cumulated traffic higher than the one contemplated at its design, PIARC Technical Committee 4.3 Road Pavements (2009).

National Cooperative Highway Research Program (NCHRP)

A long-life or perpetual pavement is one able to withstand 50 million axle repetitions in a 40-year period without failing or a mix that provided 50 million cycles or more of fatigue life in the laboratory, Prowell et al. (2010).

Rickards and Armstrong (2010) believed that the TRL findings provided a rational measure of the achievement of the perpetual pavement status in which, over time, the deflections in a full depth perpetual pavement remained constant or indeed reduced despite heavy traffic loading (according to the UK network study); this was later confirmed in extensive German research.

As stated by Newcomb et al. (2010) LLAP performance is more than a function of design traffic, climate, subgrade and pavement parameters (such as modulus), pavement materials, construction, and maintenance levels, all contribute to how a pavement will perform over the course of its life. Most researchers and pavement engineers (Merrill, Van Dommelen & Gáspár (2006) and Walubita et al. (2008)) now believe that alongside the construction issues LLAP should be designed in a way that their structures remain intact during their life time and distresses such as fatigue cracking and permanent deformation won’t occur, the should also be durable enough to withstand damage from traffic and environment.
Although the initial construction costs of perpetual pavements may be higher than conventional pavements designed to a lower life (< 30 years), Timm & Newcomb (2006) have shown that perpetual pavements have the following benefits over conventional designs:

- They eliminate reconstruction costs at the end of a pavement’s structural capacity.
- They lower rehabilitation-induced user delay costs.
- They reduce use of non-renewable resources like aggregates and asphalt.
- They diminish energy costs while the pavement is in service.

All of these result in a lower life-cycle cost of the pavement network.

The concept of LLAP’s has now been adopted by many countries (USA, UK, Germany, Canada, and China) and is being implemented on some of their most heavily-trafficked highways Asphalt Pavement Alliance (2002). In US, states such as California, Washington and Ohio have designed and developed LLAP (Hornyak et al. 2007; Monismith 1992; Ursich 2005). In Texas, Scullion (2006) reported on eight completed LLAP completed in 2005. In addition, Texas published design guidelines (Texas Transportation Institute), based on the findings of a study done by Walubita et al. (2009). The design guide contained recommendations for structural thickness design, design software, response criteria, mix design, and layer moduli values for Texas LLAP structures were presented (Walubita & Scullion (2010)). National Centre for Asphalt Technology (NCAT) also conducted studies on and developed methods for LLAP design in 2000, 2003 and 2006, the details of which are discussed in detail in Chapter 7 (Willis & Timm 2007).

Mahoney (2001), in a study for Washington Department of Transportation (WSDOT) specified goals which would define a pavement as perpetual pavement (LLAP). These goals, along with basic LLAP concepts, give engineers, the design qualifications for perpetual pavements.

1. Perpetual pavements should have a wearing course life of 20 years
2. Perpetual pavements should have a structural design life of 40 to 50 years
3. Perpetual pavements use a mill and fill (re-sheet) as their primary surface rehabilitation
4. Perpetual pavements contain their distresses to the top few centimetres of the surface

2.7 LLAP Material Selection

As found by Newcomb, Willis & Timm (2010) achieving a LLP is not just a matter of pavement thickness. The composition and the structure of a perpetual pavement also play an important role in achieving a LLAP. In a Washington State’s study by Mahoney (2001) of long-lasting pavements, it was found that many pavements with shorter life-cycles were actually thicker than pavements with superior life-cycles. Additional research has shown that while increasing the pavement thickness can help with decreasing the tensile strain at the bottom of the asphalt layer, the magnitude of strain reduction is highly mix dependent Romanoschi et al. (2008).
According to Newcomb et al. (2010) material properties and mix designs are the two other important factors along with pavements thickness to achieve extended-life distress resistant LLAP. Newcomb et al. (2001) recommended that the LLAP structure should include a rut and wear resistant impermeable upper asphalt layer. In many cases, a stone matrix asphalt (SMA), an open grade friction course (OGFC), or a dense graded asphalt (DGA) design can be used in this location. Below the wearing course there is a rut resistant and durable intermediate layer and below that there is a fatigue resistant and durable base layer, which can be achieved by the use of a low air void mix (usually achieved by higher bitumen content). This theme is consistently found in the literature with, (Gierhart (2007), Harm (2001), Newcomb, Buncher & Huddleston (2001)) all recommending a common structure, consisting of four parts to reduce the initiation and propagation of fatigue cracking and rutting in LLAP:

1) A solid foundation and/or working platform
2) A flexible, fatigue-resistant base asphalt layer
3) A durable, rut-resistant intermediate asphalt layer
4) A rut-resistant, renewable surface layer.

Bushmeyer (2002) reported on the first purposely designed LLAP in the US, the I-710 in California, which used a similar structure to the recommended LLAP concept. It consisted of 25mm open graded friction course (OGFC), 75mm of high temperature rutting resistant polymer modified asphalt as the surface layer, 150mm of standard asphalt as the intermediate layer, 75mm rich-bottom fatigue resistant (with bitumen content above the optimum) as the base layer.

(Newcomb, Willis & Timm 2010 reported on the LLAP concept recommended by Texas which proposed a very similar full depth asphalt pavement structure, which from top to bottom comprises of 25-40mm optional porous friction course (PFC), 50-75mm Stone Mastic Asphalt (SMA) course, 50-75mm transitional asphalt layer, 200mm plus rut resistant asphalt layer, 50-100mm fatigue resistant asphalt layer, all placed on a 150mm lime stabilised granular base layer. All pavement layers are supported by a natural well compacted subgrade). It is believed that this composition would undergo 30million equivalent single axle loads (ESALs) during the pavements life cycle (Walubita et al. (2008), Walubita, Scullion & Scullion (2007)). More technical details of the four of the required layers in the perpetual pavement structure can be found summarised in Yousefdoost (2015).

2.8 Field Observation of LLAP

2.8.1 US Studies

The first US study which identified the possible occurrence of LLAP in the field was undertaken by Schmorak and Van Dommelen (1996) who investigated 176 pavement sections contained in the SHRPNL database for evidence of traditional bottom up fatigue cracking. Their observations found that for pavement sections with asphalt thicknesses greater than 160mm traditional bottom up
fatigue cracking did not occur and cracking was confined to the surface and had a maximum depth of 100mm. From this, they concluded that that traditional bottom up fatigue cracking was unlikely to occur in thick asphalt pavements. These observations did not however confirm the existence of LLAP. The observations only confirmed that surface initiated cracking would occur first.

Von Quintus (2006) conducted a review and analysis of field investigation data from Long Term Pavement Performance (LTPP) asphalt test sections across North America in 1995 and again in 2006, the purpose of which was to confirm the presence of LLAP and the FEL. While not formally documented, the 1995 study utilised randomly selected LTPP sites and field observations of cracking, to conduct survivability and probability of failure analysis for full depth asphalt pavements more than 10 years old. The survivability analysis was completed to estimate a field based FEL value based on fatigue cracking observations, rather than just use values estimated from limited laboratory test programs available at the time.

The definition of failure used by Von Quintus for confirming the FEL was nominally no fatigue cracking, which in practice was taken as 2% fatigue cracking to account for recording errors. For each site the EVERSTRESS linear elastic model was used to calculate the maximum tensile strain at the bottom of the asphalt layer. A typical survival curve, based on amount of fatigue cracking and probability of occurrence, is shown in Figure 11, following.

Based on the 1995 LTPP analysis, Von Quintus suggested the existence of an endurance limit of 65m at a 95 percent confidence level for an 80kN single axle load at the equivalent annual temperature for the specific site. Conversely, the survival analysis completed with the updated LTPP performance data in 2006, did not support the concept of FEL. Von Quintus noted that the volume of heavy vehicles used in the sections in the updated study was lower than would be
considered heavy truck traffic and that much higher level of heavy vehicle traffic were needed to validate the endurance limit design premise with field observations and data. Additionally the analysis did not examine the source of the cracking, which may have been either surface or base initiated cracking.

Although Von Quinns (2006) believed that the endurance limit is a valid design premise and an asphalt mix property, he noted that no definite conclusion could be reached from the field performance data collected without a forensic investigation, to identify the source of the fatigue cracking.

2.8.2 Designed LLAP Sections

In the US and Canada there are a number of non-instrumented test sites where the performance of LLAP designs are being observed. Rosenberger et al. (2006) documented a LLAP design undertaken using Per-Road on a by-pass around Bradford, Pennsylvania that consisted of 330mm of asphalt over 315mm of granular base. Lane et al. (2006) documented three pavements in Canada which were constructed in Ontario on Highway 402, near Sarnia. These sections included a high binder asphalt layer, asphalt with a Superpave mix as the base, and a conventionally designed pavement section. These LLAP sections, again designed by PerRoad, were 325mm of asphalt over 500mm of granular base, and the conventional section, designed according to the empirical 1993 AASHO design guide, had 240mm of asphalt over 550mm of granular material.

2.8.3 UK TRL Studies

In the United Kingdom (UK) Nunn (1997) performed experimental studies on field pavements and proposed concepts for LLAP for which classical bottom-up fatigue cracking would not occur. Nunn defined LLAP as those that last at least 40 years without structural strengthening. He drew together information from full-scale experimental pavements, studies of deterioration mechanisms on the road network, long-term deflection monitoring of motorways and condition assessments with the aim of producing a design method for LLAP. The UK's pavement design system was based on experimental roads that had carried up to 20 million standard axles. When the study by Nunn et al. was conducted, these relationships were being extrapolated to more than 200million standard axles. Nunn evaluated the most heavily travelled pavements in the UK, most of which had carried in excess of 100million standard axles to evaluate the current design system. Nunn concluded the following:

- Pavements with less than about 180mm of asphalt deformed at a high rate but thicker pavements deform at a rate about two orders of magnitude less; the sudden transition suggesting a threshold effect.
- No correlation existed between the rate of rutting and pavement thickness for thick (180+ mm) asphalt pavements.
The level of traffic loading was not the major factor affecting the residual fatigue life of the thick asphalt pavements.

Deterioration of thick, well-constructed, fully-flexible pavements was not structural, and that deterioration generally occurred at the surface in the form of cracking and rutting.

Any evidence of fatigue cracking or damage in the main structural layers of the thicker, more heavily trafficked pavements was unable to be detected.

Deterioration, either cracking or deformation, was far more likely to be found in the surfacing than deep in the pavement structure.

The great majority of the thick pavements studied became stronger over time, rather than gradually weakening with trafficking as assumed.

In addition to the visual assessment, Nunn et al. undertook laboratory investigation of a number of sections of four motorways representing a range of ages (11 to 23 years) and traffic loadings (22 to 71 million standard axles). All the pavements examined, had carried more traffic than they were originally designed to carry. Cores were cut to enable the structural properties of materials that had been subjected to heavy commercial traffic in the wheel path of Lane 1 to be compared to the lightly trafficked material of the same age and nominal composition from between the wheel-paths in Lane 3. The laboratory measured residual fatigue life was then calculated. Analysis of the data showed no consistent difference between the measured residual fatigue lives with most of the difference being accounted for by variations in binder hardness and binder content between the samples extracted from the two lanes. Nunn & Ferne (2001) found that when these factors were taken into account, none of the differences were statistically significant.

The testing undertaken by Nunn et al. (2001) found that all the material tested in their experiment had a residual fatigue life lower than that of new material and traffic loading could not account for that reduction. The authors attributed this lower laboratory residual fatigue life to aging of the material. However, the relationships developed as part of the study showed that the increase in elastic stiffness with age resulted in a reduction in the traffic-induced, tensile strains, responsible for bottom up fatigue, which more than compensated for any reduction in the laboratory fatigue life. The net effect was that the predicted fatigue life increased with age.

Nunn and Ferne also examined the deflection histories of 10 heavily trafficked sections of motorway to investigate whether the stiffness of thick, fully flexible pavements reduced with time and traffic. As expected, the deflections of these sites showed considerable fluctuations which was attributed partly to the difficulty of applying accurate temperature corrections, seasonal variations in the subgrade stiffness, and variation in alignment of successive surveys. Further confirmation of these deflection trends were also provided by Falling Weight Deflectometer (FWD) measurements on the same sites. The results from FWD surveys showed that all sections had a trend of steady or decreasing deflection with age and traffic, with one exception that showed no decisive trend either way. The authors concluded that the traffic-induced stresses and strains in the road base and the
subgrade, which were considered to be responsible for structural deterioration, contrary to conventional theory were decreasing due to stiffening of the pavement over time.

As a result of the work by Nunn et al. the UK procedure for design of asphalt pavements was revised in 1997 to include a maximum asphalt thickness corresponding to minimum threshold pavement strength for the most common asphalt mixes, beyond which the pavement should have a very long but indeterminate structural life.

2.8.4 Australian Field Investigations on LLAP

Ross reviewed the design and construction of deep strength asphalt pavements, including 14 sites in Victoria which were constructed between 1971 and 1995. For this investigation, the current traffic volume data was collected, deflections were measured using the FWD and actual pavement layer thicknesses and asphalt resilient modulus for the top, middle and bottom sections of the pavement structure were determined by taking cores. Ross concluded that the majority of the investigated sites would exceed or had already exceeded the design life predicted by mechanistic analysis without further structural improvement. He also noted that the performance of the majority of the deep strength asphalt pavements studied was much better than predicted.

Based on the results, of the laboratory testing of core samples, Ross reported that the top layers of asphalt had higher modulus than the bottom layers for 11 out of total 14 investigated sites. This difference was quite significant on some of the older pavements, albeit the lower layers were 20mm mixes compared with 14 mm or 10 mm in the top layers. Ross suggested the age hardening of the top layer could be the most probable reason for the stiffness discrepancies.

Tsoumbanos (2006) examined the performance of four pavement sections aged from 20 to nearly 30 years, against roughness, rutting, cracking, strength (deflection) and stiffness (curvature). The thickness and resilient modulus of the top, middle and lower asphalt layers were determined through coring. It was found that:

- Roughness at all locations was increasing; however, it was below VicRoads intervention criteria.
- Rutting was generally between 3 and 10 mm, with two sites exhibiting constant average rutting depth and two sites exhibiting an increase in average rutting depth of approximately 1 mm per year.
- There was an increasing proportion of cracked pavement for all sites, though the rate of increase varied.
- Very low average deflections and curvature were obtained utilising VicRoads deflectograph, in all lanes and both wheel-paths.
- Higher modulus values, from resilient modulus testing of core samples, were obtained for the bottom layers compared with the top for two sites, consistent with observations by Ross but inconclusive overall.
Tsoumbanos concluded that for three of the four sites investigated, typically higher thicknesses than 210 mm, and the sites generally showed expected performance of LLAP in that:

- cracking was mostly confined to the top 40 to 60 mm of the surface layer,
- and, collected deflection data suggested a very strong pavement structure and no structural maintenance was required to date.

Carteret & Jameson (2009) identified that Tsoumbanos did not undertake coring between wheel-paths and therefore definitive conclusions on whether micro-cracking due to fatigue occurred below the surface could not be determined. However, this may not be significant as identified by Thompson et al. (2006) LLAP can be achieved with some damage to the asphalt layer, as long as the balancing point is achieved, and the lack of macro cracking is what is important.

The mechanistic analysis undertaken by Tsoumbanos of the four studied pavement sections, found that the tensile strain threshold of 70με, proposed by Monismith (1972), was exceeded for all four sites and yet no visual sign of fatigue cracking of the pavement sections which had been in-service for 20 to 30 years, was observed.

2.8.5 The Asphalt Pavement Alliance (APA) Perpetual Pavement Awards

The APA developed the Perpetual Pavement award for owners of LLAP pavements that:

- Are at least 35 years old
- Never have had a structural failure,
- Had average intervals between resurfacing of no less than 13 years
- And, the road must demonstrate the qualities expected from long-life asphalt pavements: excellence in design, quality in construction, and value to the traveling public.

Nominations, for the Perpetual Pavement Award are assessed by a panel of industry experts at the National Centre for Asphalt Technology (NCAT).

Table 1 following summarises a number of the perpetual pavement award winners, which can aid in expansion of the knowledge of LLAP. While the awards also include numerous awards for lower traffic pavement sections with less than 150mm of total asphalt thickness and while these sections show that the LLAP concept can be allied to lower volume roads, it is outside the scope of this study and has not been included in the Table 1.

<table>
<thead>
<tr>
<th>Asphalt Thickness (mm)</th>
<th>Year of Construction</th>
<th>Location Base Conditions</th>
<th>Traffic (AADT)</th>
<th>Heavy Vehicles (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>1952</td>
<td>I-287 New Jersey Turnpike</td>
<td>Clay/Marsh</td>
<td>175,000</td>
</tr>
<tr>
<td>No.</td>
<td>Year</td>
<td>Route</td>
<td>Location</td>
<td>Details</td>
</tr>
<tr>
<td>-----</td>
<td>------</td>
<td>-------</td>
<td>----------</td>
<td>---------</td>
</tr>
<tr>
<td>230-355</td>
<td>1964</td>
<td>I-90, Washington State</td>
<td></td>
<td></td>
</tr>
<tr>
<td>306</td>
<td>1967</td>
<td>I-65, Marshall County, Tennessee</td>
<td>200mm crushed rock</td>
<td></td>
</tr>
<tr>
<td>305</td>
<td>1962</td>
<td>I-40, Oklahoma City</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>1966</td>
<td>I-35, Pine County, Minnesota</td>
<td>300mm granular base, upper 75 Bitumen treated</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>1962</td>
<td>I-80, Johnson County, Iowa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>1963</td>
<td>I-17, Don Valley Parkway, City of Toronto</td>
<td></td>
<td></td>
</tr>
<tr>
<td>210</td>
<td>1963</td>
<td>Missouri DOT, US 63, Texas County</td>
<td>170 stone base</td>
<td></td>
</tr>
<tr>
<td>190</td>
<td>1955</td>
<td>Garden State Parkway, New Jersey Turnpike</td>
<td>200mm gravel base</td>
<td></td>
</tr>
<tr>
<td>320</td>
<td>1969</td>
<td>I-181, Johnson City, Tennessee</td>
<td>200mm Crushed limestone</td>
<td></td>
</tr>
<tr>
<td>250</td>
<td>1969</td>
<td>I-26, Spartanburg County, South Carolina</td>
<td>90mm sub base, 180mm base</td>
<td></td>
</tr>
<tr>
<td>260</td>
<td>1937</td>
<td>Minnesota Department of Transportation, Trunk Highway 10, Mileposts 224 to 227</td>
<td>overlays</td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>1969</td>
<td>Kentucky Transportation Cabinet, Julian Carroll-Jackson Purchase Parkway</td>
<td></td>
<td></td>
</tr>
<tr>
<td>406</td>
<td>1969</td>
<td>Montgomery County, Kentucky</td>
<td>Unstable Marsh</td>
<td></td>
</tr>
<tr>
<td>320</td>
<td>1966</td>
<td>Mississippi Department of Transportation, Interstate 59, Lauderdale County</td>
<td>150mm lime on sand clay base</td>
<td></td>
</tr>
<tr>
<td>270</td>
<td>1966</td>
<td>Tennessee DOT, I-24, Coffee County</td>
<td>200 crushed rock</td>
<td></td>
</tr>
<tr>
<td>240</td>
<td>1965</td>
<td>Virginia DOT, I-81, Mile Posts 318.4 to 324.9</td>
<td>150mm base, 300mm sub base</td>
<td></td>
</tr>
</tbody>
</table>
These awards show that LLAP can be achieved in pavements ranging from 180mm to over 300mm depending on the level of support given to the asphalt layers by the subgrade and sub-base. The results also show that the limiting thickness is close to that as found by Nunn (2001) that '270mm of asphalt would be sufficient for any traffic loading' and 340mm will give high confidence level.

### 2.8.6 LLAP Terminal Thickness Requirements

Since the concept of designing long-life pavements began in 2000, the required thickness for LLAP has been much studied and investigated. Nunn et al. (2001) first proposed the idea of thickness limit for LLAP and proposed both an upper and lower limit for pavement thicknesses. The researchers found that any asphalt in excess of 390mm (N.B. this thickness included safety factors of 100mm for top down cracking and 20mm for increased load limits) determined from the existing UK design procedure, would be of no benefit in prolonging the fatigue life of the pavement, as such pavements would perform perpetually. On the other hand, they stated that pavements with thicknesses less than 180mm would not last 40 years and were considered substandard for heavily traffic roads. Nunn et al. (2001) found that structural deformation, rapidly increasing surface cracking and eventually premature pavement failure were found to occur in thinner pavement sections.

The APA (2002) believed that there would be no additional life-cycle benefit to the pavement structure when its thickness is more than a certain limit. Gierhart (2007) found that in the US, a thickness of 500mm is proposed by some states while others have experienced surface confined distresses in pavements as thin as 160mm. Walubita, Scullion & Scullion (2007) found that the thickness of a typical pavement in Texas designed by Asphalt Institute (AI) Mechanistic-Empirical (M-E) method was greater than 500mm, however, by using perpetual pavement design concepts, the pavement thickness could be reduced by 180mm to 340mm.

In Europe, Merril et al. (2006) continued research on pavement thickness and its effects on propagation of cracks in pavement structure in Netherlands and found evidence of full-depth cracking in thin pavements with thicknesses less than 80mm while only 28% of the studied pavements with thicknesses of more than 290mm showed signs of cracking. Moreover, all cracking in these thick asphalt sections was found to be confined to the top layers, suggesting LLAP. They found that there was a distinct change in the predominant form of cracking that occurred at a
thickness in the region between 170 and 200mm. Merrill found that at this point, the formation of cracks changed from full-depth cracking to top down surface cracking).

In a study on pavement thickness and its effect on pavement structure, Rolt (2001) found results which corroborated findings of Merrill et al. Rolt reported that well-designed and constructed asphalt pavements with 270mm of thickness could provide a fatigue resistant pavement structure while little deformation would accumulate in 180mm thick pavements over time. Rolt suggested a very conservative thickness of 370mm assuming the propagation of surface cracks up to 100mm into the pavement structure and excluding the cracked section of the pavement from contributing to load spreading. However, Rolt believed that a well-built pavement of 300mm thickness was likely to be structurally resistant with some minor surface deterioration.

Al-Qadi et al. (2008) found that the tensile strain at the bottom of asphalt and the maximum shear strain in the pavement are strongly influenced by the asphalt thickness. They reported a significant strain reduction when the thickness of the pavement was increased beyond 250mm. Also, the results from their study showed that the strains in pavements 350mm thick were smaller than the recommended endurance limit to prevent bottom-up fatigue cracking by Carpenter et al (2006).

Wu & Hossain (2002) found that some in service roads in the US, despite having pavement structures thinner than 330mm and being in service for over 40 years, such as the Kansas Turnpike, have had little or no sign of fatigue cracking. While Mahoney (2001) found in studies on long-lasting asphalt pavements in Washington State that pavements with thicknesses greater than 160mm, cracks (if any) were initiated at the surface and generally didn't propagate through the full depth of the pavement structure.

Fee (2001), by reviewing the strategies for designing and maintaining long-life pavements in the UK, France, Netherland and several states in the US, concluded that to achieve an extended life asphalt pavement, typically a thickness of 300-350mm was required and a minimum of 200 mm was required.

2.9 Design Procedures for LLAP

Notwithstanding the wide acceptance of the fatigue endurance limit concept its introduction into LLAP thickness design is problematic. Its introduction into pavement design has been subject to two major US studies under NCHRP project 9-38 and 9-44. One of the key issues under research is the healing of micro-cracks and restriction of fatigue crack propagation from the bottom of the asphalt base.

In order to successfully design a pavement section, the designer needs to consider the appropriate thickness for the pavement layers taking into account the heaviest anticipated traffic loads while avoiding overdesigning the thickness. Newcomb, Willis & Timm (2010) showed that by limiting the mechanistically defined stresses, strains and displacements of the pavements layers, the initiation
of deep cracking or rutting in the pavement structure can be avoided. These thresholds are often referred to as limiting pavement responses.

2.9.1 Pavement Structural Distresses

For the design of LLAP defining critical pavement responses (stress, strain or displacement) below which no accumulation of structural damage to the pavement occurs is required. In the LLAP design concept it is assumed that if the load-induced pavement responses remain below such a level, then the design can be considered as a LLAP or perpetual pavement. Currently, as identified by, Newcomb, Willis & Timm (2010) most LLAP design approaches deal with pavement responses associated with structural rutting and bottom-up fatigue cracking.

2.9.2 Structural Rutting

When the overall strength of the pavement structure is incapable of resist the traffic loading structural rutting may occur from induced deformations either in the granular base or subgrade. As identified by Nunn (2001) and by Newcomb, Willis & Timm (2010), although structural rutting is rarely seen in modern thick asphalt structures, it necessitates major rehabilitations and reconstructions. Rolt (2001) and Brown et al. (2002) based on the findings from the National Centre for Asphalt Technology (NCAT), found that no structural rutting takes place in thick asphalt pavements and rutting is was confined to the upper few centimetres of these pavements, which can be easily repaired by mill and resheet treatments. To limit structural rutting Harvey et al. (2004) and Walubita et al. (2008) recommended the use of a vertical compressive strain of 200mε at the top of subgrade layer as the limiting design parameter. They found that rutting (plastic deformation) in the lower layers would not take place if the compressive strain at the top of the subgrade was kept below 200mε. Newcomb, Willis & Timm (2010) identified that this limiting response can be achieved by either increasing the overall pavement structural thickness or increasing the stiffness of one or more of the pavement layers.

The researchers at the University of Illinois used a different approach, being the ratio of subgrade stress to the unconfined compressive strength of the soil, Subgrade Strength Ratio (SSR), as the limiting response characteristic. Bejarano & Thompson (2001) and Bejarano, Thompson & Garg (1999) recommended for the clay soils studied in their research, that the critical value for SSR was found to be in the range of 0.5 to 0.6. They suggested an SSR of 0.42 for design purposes.

2.9.3 Fatigue Cracking

The other major mode of distress in flexible pavements is bottom-up fatigue cracking. After formation, this cracking propagates to the surface through the layers of the asphalt. This causes the infiltration of water into the pavement layers and subsequently changes of unbound material properties and can result in accelerated surface deterioration, pumping and rutting. Huang (1993) identified that conventional theory states that high repeated load induced strains at the bottom of
The asphalt layer is the cause of this mode of distress. Conventional design procedures control this fatigue cracking by controlling the horizontal strains at the bottom of the asphalt layer. The main approach used to decrease the probability of bottom-up fatigue cracking is to increase the thickness of the pavement structure.

Willis (2008) reported on a 2006 survey of Accelerated Pavement Testing (APT) facilities in the US that to predict and study fatigue potential a large majority of the responding facilities measured horizontal strain at the base of the asphalt layer. LLAP projects such as the I-5 in Oregon (Estes (2005), Scholz et al. (2006)) have incorporated measuring strain, Hornyak et al. (2007) identified that the Marquette Interchange in Wisconsin have incorporated measuring strain at the base of the asphalt layer into their research.

While the predominate research is to measure strain at the underside of the asphalt layers to study fatigue, it has been shown by numerous researchers that cracking in thick pavements is generally confined to the pavement surface due to reduction in intensity of strains at the bottom of asphalt pavement (Al-Qadi et al. (2008), Asphalt Pavement Alliance (2002), Martin et al. (2001), Merrill, Van Dommelen & Gaspár (2006), Newcomb, Buncher & Huddleston (2001)).

As identified by Mahoney (2001) and Rolt (2001), for thick asphalt pavements the critical location of the strains in pavements may change from the bottom of the asphalt layer to the surface of the structure as the strains at the bottom of the asphalt layer are reduced. Ferne (2006) showed that by controlling the strains at the bottom of the asphalt layer and hence confining the distresses to the surface layer, deep structural rehabilitations and reconstructions can be avoided and maintenance would be limited to functional maintenance such as skid resistance and ride quality. Mahoney (2001) identified that surface cracking in these pavements can be maintained by, a "mill and fill" (mill and resheet) maintenance plan for extending the pavement's life.

Beside the advantages of thick asphalt pavements to control fatigue cracking, studies on thick pavements at the National Centre for Asphalt Technology (NCAT) Pavement Test Track Brown et al. (2002) and research done by Rolt (2001) have shown that rutting is also limited to the surface layer in thick pavement structures.

Al-Qadi et al. (2008) has shown that the longitudinal strain at the bottom of asphalt layer has proven to be critical in thinner pavements and in a fully-bonded pavement, it is always the location of highest tensile strain. Walubita et al. (2008) recommended that a typical threshold limit for strain (FEL) at the bottom of asphalt layer to prevent bottom-up fatigue cracking should be 70m.e. However, some other research has shown different threshold values.

Based on laboratory testing, Romanoschi et al.(2008) proposed that the strain threshold of 60 to 100m.e be applied everywhere in the pavement structure, while according to an experimental pavement project in China, Yang et al.(2005) reported a value of 125m.e.
However, as identified by Willis & Timm (2009b), it should be noted that pavement responses and
FEL of flexible pavements rely on the type of mixes and the conditions of the testing undertaken in
each reviewed study and that results based on tests on different mixes with different material and
testing conditions may not be globally representative.

2.9.4 Empirical Approaches

Von Quintus (2001b) developed one of the earlier approaches to LLAP design for the State of
Michigan. Von Quintus chose to use a mechanistic approach employing the ELSYM5 computer
program to calculate stresses and strains in the pavement structure. This approach applied the
concept of cumulative damage to determine the appropriate section for a design period of up to 40
years. Von Quintus used this methodology, in the absence of other approaches, as a way of
determining a reasonable range of pavement thicknesses for LLAP.

In the definition of LLAP Von Quintus used low levels of predicted distresses for the design criteria
rather than limiting strains. As part of the procedure Von Quintus recommended rehabilitation
strategies to enable the pavement to last for a period of 40 years. In line with the current approach
to LLAP the rehabilitation strategies were mill and resheet operations at years 15 and 30, except
for the lowest level of traffic where they were scheduled for years 32 and 40. The result of this
analysis was that Von Quintus developed a catalogue of LLAP design for Michigan.

2.9.5 Terminal Thickness or Traffic

As a result of the work done by Nunn et al. the UK procedure for the design of asphalt pavements
was revised in 1997 to include a limiting thickness for the most common asphalt thicknesses.
Beyond this thickness the pavement would have an indeterminate structural life.

The structural section for the LLAP in the United Kingdom includes the use of granular base and
sub-base layers below a thick asphalt pavement. The thickness of the asphalt is such that traditional
bottom-up fatigue cracking and structural rutting are avoided. Nunn and his associates found that
pavements having a total asphalt thickness of less than 180mm are prone to structural rutting, while
the rutting in thicker pavements is confined to the top of the structure. Rutting occurs mainly in the
top 100mm of thick asphalt roads in the United Kingdom. The TRL approach allows for an
adjustment in asphalt thickness according to the type of mix and stiffness of the binder. The
standard dense bitumen macadam base uses a 100-penetration asphalt binder and has a limiting
thickness of 390mm. For the DBM50 which is similar to Australian C320 and 450, mixes a limiting
thickness of 340mm is recommended.

Using increasingly stiff binders allows for the design of thinner sections according to the British
approach. However, the British researchers placed an upper limit on asphalt thickness based upon
observed distresses. Studies of the performance of British roads show that additional pavement
thickness, beyond that required for 80 million ESAL, would not provide additional benefit as shown
in Figure 12. Nunn and his associates state that the fourth power law, traditionally used for describing the relationship between pavement damage and axle loads, is not appropriate for thick asphalt pavements.

![Figure 12 TRL Design Chart, after Nunn (2001)](image)

2.9.6 Mechanistic Empirical

The Asphalt Pavement Alliance worked with Auburn University Timm (2008), to develop PerRoad, a computer analysis program to design LLAP using Mechanistic Empirical principles. The program couples layered elastic analysis with a statistical analysis procedure (Monte Carlo simulation) to estimate stresses and strains within a pavement Timm and Newcomb, (2006). In order to predict the strains which would prove detrimental for fatigue cracking or structural rutting, PerRoad requires the following inputs:

- Seasonal pavement moduli and annual coefficient of variation (COV)
- Seasonal resilient moduli of unbound materials and annual COV
- Thickness of bound materials and COV
- Thickness of unbound materials
- Load spectrum for traffic
- Location for pavement response analysis
- Magnitude of limiting pavement responses
Transfer functions for pavement responses exceeding the user-specified level

For accumulating damage PerRoad follows a linear sum of damage approach. The Monte Carlo simulation is simply a way of incorporating variability into the analysis to more realistically characterise the pavement performance. The output for PerRoad consists of an evaluation of the percentage of load repetitions lower than the limiting pavement responses specified in the input, an estimate of the amount of damage incurred per single axle load, and a projected time to when the accumulated damage is equal to 0.1. On high volume pavements, the critical parameter for LLAP design is the percentage of load repetitions below the limiting strains. It is generally recommended that the designer ensure that 90 percent or more loads are less than the critical threshold value.

Thompson and Carpenter (2004) presented LLAP design concepts in the context of laboratory work undertaken at the University of Illinois. In this case the model employed to represent the pavement was a finite element program called ILLI–PAVE in which 8kN and 8.5kN axle loads served as the loading condition. These researchers reasoned that this would be the extreme case in hot weather as these loads would represent the worst condition with very few loads being greater than this. Their work showed that up to 30 percent of the fatigue life of the pavement could be consumed, yet if the remaining strains were below the FEL, there would be no fatigue cracking. They went on to verify these results with field deflection measurements. From these, they were able to conclude that many existing pavements could be classified as LLAP.

The US AASHTO Mechanistic-Empirical Pavement Design Guide AASHTO, (2008) can be used for LLAP design by incorporating a Fatigue Endurance Limit (FEL). The design procedure is currently being calibrated and adopted by a number of states across the U.S. It predicts the accumulation of a variety of pavement distresses over a user-prescribed analysis period. Based on information from NCHRP Project 9-38 Prowell et al. (2006), Witczak et al. (2006) incorporated an optional FEL ranging between 75 and 250m.

Researchers have begun to investigate the use of the MEPDG in conjunction with the FEL to optimize pavement designs (Behbahais et al., 2009; Tarefoler et al., 2009). In fact, Willis and Timm (2009) found good agreement between PerRoad and the MEPDG in terms of thickness requirements when the FEL was employed.

2.9.7 NCAT Test Track

Willis et al. (2009) and researchers at NCAT examined the performance of three cycles of the test track (2000, 2003 and 2006) to develop strain criteria for prevention of fatigue cracking in LLAP. They estimated the strain distributions for the 2000 test track experiment using the mechanistic pavement modelling program PerRoad and for the 2003 and 2006 cycles, direct strain measurements from the base of the asphalt layers were used to develop strain profiles. In 2000 test track, six test sections that had experienced at least 20 million ESA’s (Equivalent Single Axle load) without showing signs of fatigue cracking, were selected and studied for theoretical strain.
analysis. The stiffness of the asphalt and the resilient modulus of the soil were characterised using FWD (AASHTO two-layer back-calculation methodology). Relationships were developed between the asphalt modulus and the midpoint temperature in the asphalt for each test section during the time of testing.

Using the developed relationships, cumulative distributions of the stiffness were developed by calculating stiffness based on the average hourly temperature under trafficking.

In the 2003 test track, embedded asphalt strain gauges were used to measure the strain at the base of the asphalt layer(s) of the pavement structure in eight sections. Detailed trucking databases allowed precise loading configurations to be analysed and weekly measured pavement responses were used to develop continuous strain distributions for the structural sections. These two design components were linked to the observed pavement performance of the section to make correlations between pavement response and performance in test sections. Previously developed relationships between pavement response and temperature were used to develop cumulative distribution of strain for the life of the pavement. Similar to 2003 cycle, in the 2006 test track the actual strains were measured at the base of the asphalt and the same procedure was used to create the cumulative strain distributions. In the 2006 test track, relationships between longitudinal strain and temperature by axle for each section instead of by truck were also developed.

Willis et al. derived distinction between the cumulative strain distributions of the sections that failed in fatigue to those that did not. The comparison of the field performance and cumulative strain distribution of each test section suggested the existence of a limiting cumulative distribution of strain to avoid asphalt fatigue cracking. Three criteria were considered to help develop a new strain-criterion for flexible perpetual pavement design:

1) The section could not be overdesigned;
2) The section could not have exhibited any fatigue cracking;
3) The section had to have experienced at least 20 million ESALs. Analyses for N3 and N4 test sections by axle and by truck met the criteria.

Allowing ±15% confidence boundary, Willis et al. 2009 determined that the average of the strain distributions for these test sections were an appropriate field-based strain threshold for designing LLAP. Rickards & Armstrong (2010) believed that the concept of LLAP thickness design based on compliance with limiting cumulative distribution of asphalt strain has considerable merit. It has the potential to avoid the acknowledged uncertainty in theoretical fatigue modelling and constrain management of asphalt fatigue and durability to satisfying compliance hurdles.
3 Experimental Plan

3.1 Overall Design

The overall design of the study was divided into five (5) main tasks, as shown following and described in depth in the following chapter.

1. Material classification experiment
2. Laboratory field modulus interconversion study
3. Modulus inter-conversion study
4. Development and calibration of a LLAP design procedure
5. Validation of the LLAP design procedure using Australian and UK LLAP data.

3.2 Material Classification Experimental Design

The objective of the material characterisation component of the APS-fL project was to provide real data on the performance characteristics of actual standard Australian production mixes. Given the combination of binders, aggregate sources, producers across Australia, obviously not all production mixes could be included in the study. Therefore in order to keep the size of the characterisation study to a manageable level, the design of the experiment was rationalised to 30 mixes by the Project Steering Committee. The 30 mixes were selected to cover the majority of the combinations of aggregate sources and binder types used across Australia, without duplication of relatively similar mixes.

The performance characteristics of asphalt mixes are primarily influenced by the following key factors;

- mix size and gradation,
- bitumen type and content,
- aggregate type and proportions,
- and, air void content.

In order to capture the effects of these parameters on the standard asphalt materials, the study primarily focused on the asphalt mixes produced by Australia’s major asphalt producers, for State Road Authorities. It was believed that these mixes would most likely be used in major projects where the LLAP concept is most likely to be implemented.

The design of the experiment was limited to 30 asphalt mixes, nominally 15 each of 14mm and 20mm mixes. As the overall objective of the project is the development of LLAP design procedure, the emphasis was placed on the harder binder grades, which are believed to offer greater structural benefit. Likewise, as the main structural layers in the pavement will be the larger stone mixes, focus was placed on the 20mm mixes.
As the goal of the project is to model actual field performance as close as possible, the experiment was designed using plant produced asphalt mixes, as it was believed that these materials more closely matched the reality of asphalt produced and placed in the field than that produced in the laboratory. One item noted in using plant produced mixes is that the current Austroads mix design and pavement design characterisation is based on laboratory mixes. However, this is not always the case in practice with State Road Agencies such as the NSW Roads and Maritime Services, Queensland Department of Main Roads and DPTI, moving to plant validation of asphalt mixes over laboratory values. Additionally, nearly all designs in Australia to date are based on indicative values with little validation of the actual modulus values used across the spectrum.

3.2.1 Mix Selection

Currently, there are three major suppliers who operate in all states of Australia. If all suppliers, in all states, were to contribute to the program, the size of the experiment would become be very large and result in the duplication of essentially identical mixes. It was therefore considered necessary to firstly go through a rationalisation process to constrain the experiment to a manageable size without lessening the value of the output. This was achieved by limiting the duplication of mixes, comprising the same bitumen and aggregate components, as it is accepted that these components have the greatest influence on the characterisation of the asphalt mix. An asphalt mix with the same components from different suppliers should have similar performance. (There are obviously other factors that will impact on the performance results e.g. mix gradation and in particular fine fractions and fillers).

To accomplish this as a first step, suppliers were requested to fill out, for each mix submitted, the 'Asphalt Material Component and Gradation Details', results of which are shown in Appendix A. The anonymous summary was then reviewed by the Project Steering Committee and materials for the subsequent testing program selected and the supplier advised accordingly and samples requested.

3.2.2 Supply of Asphalt Mixes

In order to ensure an adequate supply and reserve of the selected materials, producers were requested to supply a minimum 300kg of a current production plant manufactured mix.

Prior to packaging each of the samples was subjected to standard compliance testing (maximum density, bulk density binder content and gradation) and identified only by the AAPA mix identification code. This code was provided to the testing laboratory to enable the compaction of cylinders and beams at the Australian standard 5% target air void level. Additionally, for all samples, the supplier was to supply the: aggregate composition, the gradation, their design method, volumetric, quantity of Recycled Asphalt Product (RAP) and binder type, the results of which are shown in Appendix A.
3.2.3 Supplied Asphalt Mix Materials

The results of the initial rationalisation identified 30 mixes for use in the study. These mixes were identified to cover the spectrum of aggregate types, design methods and non-modified Australian bitumen (Classes 320; 450; 600 and Multigrade) used in Australia. In states where multiple suppliers use common aggregate types, only a single supplier provided the mix.

Given the objective to model actual field performance as close as possible, the experiment was designed using plant produced asphalt mixes. It was believed that these materials more closely matched the reality of asphalt produced and placed in the field than that produced in the laboratory.

Of the 30 mixes identified for inclusion in the study, 28 were in production over the experimental period and therefore included in the study. Table 2 following summarises the resultant 28 mixes used with their volumetric properties and mix design method.

<table>
<thead>
<tr>
<th>Nominal Size (mm)</th>
<th>Binder Type</th>
<th>Mix Design Method</th>
<th>Design Voids (%)</th>
<th>Design VMA (%)</th>
<th>Design VFB (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>A15E</td>
<td>Marshall 50 blow</td>
<td>4</td>
<td>14.7</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 120 cycles</td>
<td>5.4</td>
<td>15.5</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>Gyratory 120 cycles</td>
<td>4.2</td>
<td>15.5</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 120 cycles</td>
<td>5</td>
<td>15</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>C320</td>
<td>Gyratory 120 cycles</td>
<td>4.3</td>
<td>15.5</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 120 cycles</td>
<td>5.4</td>
<td>15.5</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall 75 blow</td>
<td>5.01</td>
<td>15.17</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall</td>
<td>4.9</td>
<td>16</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall 75 blow</td>
<td>5.5</td>
<td>16.4</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall 50 blow</td>
<td>5.2</td>
<td>15.6</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 80 cycles</td>
<td>4.5</td>
<td>14.5</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 80 cycles</td>
<td>4.5</td>
<td>14.6</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall 75 blow</td>
<td>3.8</td>
<td>14.2</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td>Multi Grade</td>
<td>Marshall</td>
<td>5.7</td>
<td>15.7</td>
<td>64</td>
</tr>
<tr>
<td>20</td>
<td>AR450</td>
<td>Gyratory 120 cycles</td>
<td>4.8</td>
<td>15.5</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 120 cycles</td>
<td>4.9</td>
<td>14</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gyratory 120 cycles</td>
<td>5.2</td>
<td>15.6</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td>C320</td>
<td>Marshall</td>
<td>5</td>
<td>15.2</td>
<td>67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Marshall 75 blow</td>
<td>3.8</td>
<td>14.2</td>
<td>73</td>
</tr>
</tbody>
</table>
3.3 Asphalt Mix Properties

For each of the 28 supplied production mixes, the volumetric properties and aggregate gradations were supplied by the producer. A comparison of the supplied information was undertaken for each nominal aggregate size in order to obtain an indication of the variability of standard production mixes across Australia, the results of which are discussed in the following section.

3.3.1 Gradation

The gradation of the supplied nominal 14mm and nominal 20mm mixes can be seen graphically in Figure 13 following, on a 0.45 power gradation curve.

<table>
<thead>
<tr>
<th></th>
<th>Marshall 50 blow</th>
<th>Gyratory 80 cycles</th>
<th>Marshall 75 blow</th>
<th>Gyratory 120 cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.3</td>
<td>4.5</td>
<td>3.8</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td>15.2</td>
<td>13.5</td>
<td>14.2</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>69</td>
<td>66</td>
<td>73</td>
<td>72</td>
</tr>
<tr>
<td></td>
<td>Marshall 50 blow</td>
<td>5</td>
<td>4.8</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>15.2</td>
<td>14.8</td>
<td>13.9</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>68</td>
<td>68</td>
<td>67</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td>Marshall 50 blow</td>
<td>4.5</td>
<td>14.5</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>69</td>
<td>69</td>
<td>69</td>
<td>69</td>
</tr>
</tbody>
</table>

Figure 13 Gradation plots "Nominal" 14mm mixes
The plots show that for both the 14mm and 20mm nominal mixes, nearly all gradations closely follow the maximum density line, with, nearly all slightly coarse graded. The results show that fine and gap graded mixes do not appear to be commonly used in production mixes for major projects throughout Australia. What is evident in the gradations, particularly in the 20mm mixes, is that the definition of ‘nominal’ does vary across Australia and some mixes defined as 20mm nominal mixes would be classified differently in other states.

These results may indicate it may be difficult to distinguish between Australian asphalt based on aggregate gradation, due to the limited variation.

3.3.2 Volumetric Properties

The volumetric properties of the supplied mixes are shown graphically on a four axis volumetric plots, for the 14mm nominal mixes and the 20mm nominal mixes, in Figure 15 and Figure 16 following.
Figure 15  Volumetric Plots (a) 14mm Mixes

Figure 16  Volumetric Plots 20mm Mixes
What is noteworthy, given the variability in the design methods (Marshall, gyratory and Superpave) and the differing compaction efforts, is that all mixes fit into a very small volumetric window.

- For the 14mm nominal mixes all mixes had VMA between 14 and 16%, VFB between 65 and 75% and volume of effective binder between 10 and typically 11%.
- For the 20mm mixes there was a slightly higher but still surprisingly small variation, with all mixes having VMA between 13.5 and 15.5%, VFB between typically 60 and 70% and volume of effective binder typically between 9 and 10.5%.
- All mixes had design voids between 4 and 6%.

As with the results of the gradation analysis these results indicate it may be difficult to distinguish between Australian asphalt based on volumetric properties due to the limited variation in mixes across Australia.

3.4 Sample Preparation

For the 28 production mixes 300kg of representative asphalt samples were taken from plant production mixes from actual projects, cooled and delivered to Fulton Hogan’s National Technical Laboratory in sealed nominal 20kg containers.

3.4.1 Reheating of Mixes

Reheating was standardised across all mixes to minimise any effect aging may have on the measured material characterisation. To accomplish this, the following reheating process was undertaken.

1. Material was warmed to 70°C and broken down by hand on a quartering tray and quartered to give representative 28-30kg for each shear-box block.
2. The mix was placed into two separate shear-box feeding trays.
3. Thermo couples were inserted into the centre of the loose mix for each tray.
4. The shear-box trays were covered and placed in preheated ovens at 150°C for conventional binder and 165°C for the polymer modified binder.
5. Temperature was monitored via the thermal couples inserted in the sample until a constant temperature of the mix was achieved.

3.4.2 Compaction of Mixes

The compaction method chosen for the production of laboratory samples was the shear box compactor. It was recognised the Shear-box compactor is not used in AASHTO and Europe standards for material characterisation (although there is a current draft ASTM method) and the method has not been adopted by Austroads. However, the shear-box compactor was selected as:

1. The current Austroads practice (gyratory compactor) would not produce specimens of the correct size and air void distribution for use in dynamic modulus test.
2. The US standard compaction method, the Superpave Gyratory Compactor, was not readily available in Australia, produces samples with a higher degree of variability between samples than the shear box, and requires more laboratory time and material than the shear box.

3. The rolling wheel compactor was impractical and as stated by Harman (2002) "rolling wheel compaction proved to be somewhat impractical as a sole means of laboratory compaction, the equipment proposed was large and required very large batches of mix."

4. The shear box is highly repeatable, as confirmed by Qiu et al. (2009) who found "the shear box compactor provides a reliable means of sample preparation, making it very suitable for producing specimens with constant volumetric properties."

While the Shear-box was selected as the preferred method for compaction, most work undertaken in the US uses a different compaction method, the Superpave Gyratory Compactor. Therefore it was noted parallel testing needed to be undertaken to ensure the result of shear box compacted mixes are directly comparable to Superpave Gyratory Compactor compacted mixes.

3.4.3 Shear-Box Compactor

The concept of the Shear-box compactor is shown in Figure 17, following. The concept of the shear-box is that a sample, at a controlled temperature and of prescribed size and prismatic shape is subjected simultaneously to constant static vertical pressure and to a lateral (shearing) stress that alternates in direction. The vertical pressure, shearing rate (no. of cycles/minute) and shear angle are controlled during compaction and compaction is continued until a predetermined height is achieved.
Because the shear box compactor produces large rectangular prismatic blocks, multiple specimens can be obtained from the same block, therefore reducing the variability between samples.

For the AAPA study the dimension of each shear box compactor prism was 450 x 150 x 180 to 200mm (L x W x D). From this block 3 AMPT samples (i.e. 3 by 100 x 150mm cylinders); were obtained by coring of manufactured block.

1. The process followed to compact the shear-box prisms is as follows.
2. The top and bottom platen of the shear box is preheated to the mix temperature
3. A mix feeder is placed on top of mould, the feeder is designed to minimise segregation and provide particle orientation in the direction of compaction and achieve uniform density
4. The sample was fed into the mould using two feeder trays designed one from each side of the mould
5. The surface of the sample is manually levelled using minimum mixing to avoid surface segregation
6. The top platen is placed on surface of the sample
7. A 750kPa constant vertical pressure is applied to the sample
8. Shearing is applied to the sample at a rate of 3 cycles per minute, at an shear angle of 2°
9. Shearing is continued until the required density of the specimen is achieved.
10. The number of cycles and shear force required to compact each prism to the target void were recorded
11. Bulk density of each block was recorded by water displacement method AS 2008
12. Any remnant of each of the un-compacted materials was retained in their original drums for possible future examination.

Repeats were sometimes necessary to quantify the effect of boundary conditions on sample density.

Figure 18 Shear Box Compactor

3.4.4 Coring and Trimming

Each asphalt sample prism from the shear box was cored to produce 3 cylinders using a 100mm diameter diamond tipped coring apparatus. Each sample was then trimmed to 150mm height, with approximately 15mm being taken from each end of the sample, using a diamond tipped automated saw, to produce test specimens with a diameter of 100mm and height of 150mm.
The density of the trimmed samples was calculated and tested for consistency and conformity with the target voids (5%) voids and two of the three cylindrical samples tested in the AMPT. The third sample was kept as a reference and stored for any future/replicate testing.

### 3.5 The Dynamic Modulus Test

To obtain real information on the characterisation of standard production mixes used throughout Australia, the APS-fL project undertook dynamic modulus test on all of the supplied 28 actual production mixes. As at the time of testing it was unknown what state of stress would be required to accurately model the response of the pavement to cover all possible stress states, the AAPA study conducted dynamic modulus testing in both unconfined and confined state. For the confined state three levels of confinement were used: 50, 100 and 200kPa.

For each of the 28 supplied mixes the dynamic modulus test was performed according to AASHTO TP62-07 using an IPC AMPT. To minimise potential damage to the specimen, testing was undertaken in the following order, before the next sequential test; the reason for this approach is asphalts are stronger at lower temperatures and higher frequencies.

- For each test temperature, $E^*$ tests were conducted on each specimen at a full sweep of loading frequencies (25, 10, 5, 1, 0.5 and 0.1Hz).
- Testing was conducted at 200, 100, 50 and 0kPa confining pressures.
- Test temperatures were used from coolest to highest 5, 20, 35 & 50°C.
- Two replicates samples were tested for each factor combination for both confined and unconfined testing.

#### 3.5.1 Testing System Set Up

An Industrial Process Controls (IPC) AMPT (Asphalt Materials Property Tester), formally SPT, was used to conduct the dynamic modulus test. The AMPT is a closed-loop servo-hydraulic testing system manufactured by IPC in Australia. The machine is capable of applying load over a wide range of frequencies (from 0.1 to 25 Hz). The servo hydraulic system is controlled by an IPC controller. The temperature control system of the AMPT is refrigeration-based and is able to control temperatures in the range of 5 to 55°C, for extended periods.

The measurement system is computer controlled and capable of measuring and recording a minimum of 8 channels, simultaneously. For the dynamic modulus test 7 channels are used; three channels for on-sample vertical deformation measurements, two channels for the load cell and the actuator Linear Variable Differential Transducer (LVDT), one channel for temperature, and one channel for the confinement pressure measurement.

Loads are measured using electronic load cells capable of measuring loads with an accuracy of ± 0.1 %. Vertical deformations are measured using three LVDTs. Figure 19 following, shows the AMPT testing system.
3.6 Linking AAPA database with Overseas Research

In the United States the Superpave Gyratory Compactor (SGC) is generally used for sample compaction and preparation for dynamic modulus testing. This compaction method is different from the shear-box compaction used in the AAPA study. It was unknown whether the AMPT results on samples manufactured using SGC compaction will vary relative to the shear-box compacted samples. Hence, to investigate whether this different method of compaction has any influence on the material characterisation results, a small subset of the production mixes was sent to NCAT and compacted using the Superpave Gyratory Compactor.

These mixes were tested at the NCAT laboratories for Dynamic Modulus using the same equipment, an IPC Global Asphalt Mix Performance Tester (AMPT). NCAT uses a both a different test procedure, AASHTO TP 79-12 'Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mix Performance Tester (AMPT)', and used different testing frequencies and temperatures as shown in Table 3 following. The NCAT testing used one confining pressure only, 200kPa.

<table>
<thead>
<tr>
<th>Test Temperature (°C)</th>
<th>Loading Frequencies (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>25,10,5,1,0,5,0.1</td>
</tr>
<tr>
<td>20.0</td>
<td>25,10,5,1,0,5,0.1</td>
</tr>
<tr>
<td>35.0</td>
<td>25,10,5,1,0,5,0.1, 0.01</td>
</tr>
<tr>
<td>50.0</td>
<td>25,10,5,1,0,5,0.1</td>
</tr>
</tbody>
</table>

Figure 19 Dynamic Modulus Test Setup
In order to allow direct comparison of results obtained by NCAT and results obtained in the AAPA study, four shear-box compacted samples were sent to NCAT for Dynamic Modulus testing. The mixes sent to NCAT were chosen to cover the range of typical mixes used in the study and range of modulus results, namely: A15E, C320, AR450 and C600. In addition to the four samples compacted using shear box compactor, two loose samples were sent to NCAT for sample preparation using the SGC to determine if the compaction method has any effect on the material characterisation results.

3.7 Calibration Using NCAT Test Track Sites

Currently there are no long term instrumented pavement research studies in operation in Australia, with accompanying dynamic modulus characterisation. While instrumented studies are being developed by both Curtin University and University of the Sunshine Coast; data will not be available from these studies for a number of years. Therefore, the study had to rely on test track results from US research to determine:

1. The conversion between laboratory and field modulus.
2. The relationship between ambient and pavement temperature regimes.
3. The effects of a moving load on asphalt modulus and calculated strain using linear elastic analysis.
4. The determination/calibration of the threshold modelling method for LLAP pavement design, whether that is, the cumulative distribution of strain, threshold strain criteria or a threshold traffic loading.

It was found that the NCAT instrumentation test sections would provide all the data which can be used to meet these objectives. Additionally, it is noted the climate in Alabama is not dissimilar to the lower east coast of Australia, although it is recognised that validation against Australian data and sites of the proposed models developed from the NCAT test track will be required.

3.7.1 Selected Test sites

The APS-fL project examined the NCAT test track and concluded that the 2003 test cycle would provide the most valuable information for the calibration of a LLAP design procedure. The 2003 NCAT Test Track cycle included eight structural sections. The eight structural sections were selected to evaluate pavement sections designed for varying levels of traffic, polymer modified and standard binders, stone mastic asphalt (SMA), and high binder layer. For the experiment, all eight sections were placed on 150mm (6 in.) granular base, the test section layout is shown in Figure 20 following.
The pavement sections were constructed using both a standard PG 67-22 binder (similar to an Australian C450 binder) and a styrene-butadiene-styrene (SBS) modified PG 76-22 (similar to an Australian A10E binder). Loading on the 2003 track, was provided by tri-trailer trucks and one five-axle single trailer. For the tri-trailer, the typical weight of each set of dual wheels was just over 9 tonnes. For the five-axle single trailer, the typical dual wheel load was just less than 8 tonnes. Tyre inflation pressures were typically 700kPa (105 psi).

During the test cycle six of the eight structural sections developed some degree of fatigue cracking, the exception being sections N3 and N4 which were the 230mm thick asphalt sections. (These two sections have not failed in subsequent studies and have now had in excess of $6 \times 10^7$ cycles). Of those sections which cracked, three of the sections failed, (N1, N2, and N8) with failure defined as fatigue cracking exceeding 20%. The remaining sections N5, N6 and N7 failed in subsequent studies. Detailed data on the performance of each section is reported by Timm and Priest (2009).

During the test cycle, the performance of the rich bottom section N8 was unexpected, particularly when compared with the performance of Section N7. The performance of this section was investigated by Willis et al. (2007) who conducted a forensic evaluation of the N8 section and found that slippage between the rich bottom layer and the overlying base layer had occurred and that cracking began in the overlying base layer. Due to potential bias that this may introduce it was decided to exclude section N8 from the analysis in this study.

### 3.7.2 Accelerated Loading at NCAT

The research being undertaken at NCAT is what is commonly referred to as accelerated pavement testing (APT). Accelerated pavement testing compresses a full design life of truck damage (up to 20 years) into 2 years. At NCAT, loading of the facility generally begins in the US autumn after the completion of construction, and the collection of baseline data. The loading of the pavement operates on two shifts, an am shift which runs from 5am until 2pm, and a pm shift, which runs from 2pm until 11pm. Under this operation each truck in the 5 truck fleet runs approximately 1000km (680 miles/day) to load the experimental pavements. Because all sections are subjected to identical
and precisely monitored levels of traffic, it is possible to complete meaningful field performance comparisons between asphalt material types and designs. The NCAT test track has been through a number of full test cycles.

3.7.3 Monitoring Field Performance (NCAT)

The performance of each of the pavement sections on the NCAT test track is monitored on a weekly basis. The following performance monitoring is undertaken:

1. Rutting, roughness and surface are measured on the full track using an inertial profiler equipped with a full lane width dual scanning laser "rutbar".
2. Random locations selected from within each section are used to measure wheel path densities using non-destructive testing. The transverse profiles are measured along these same locations so that rutting may be verified using a contact method.
3. Falling Weight Deflectomer (FWD) testing is typically run weekly, which is also the case with high speed structural response data collection and surface crack mapping.

3.7.4 Instrumentation

From Phase II of the NCAT test track (2003 study), the researchers began to equip the test track with full instrumentation for the measurement of temperature profiles and high speed instrument arrays for measurement of pressure and strain in the pavement. The temperature probes for the measurement of temperature profiles were also paired with data from an onsite automated weather station, which can be used to characterize the performance environment. For the structural experimental sections, (the sections of most use to the APAA study), high speed instrumentation arrays consisting of strain gauges and pressure plates are installed at critical depths within the pavement. Measurement data generated by these devices can be used to quantify the response to passing loads at a given temperature in the pavement. It is this data which will be extremely useful in calibrating the proposed mechanistically based APS-FL LLAP pavement design methodology.

3.8 Other Calibration Sites

Apart from the NCAT test track a number of additional sources of information were available in the literature, which could be used to compare laboratory dynamic modulus to field stiffness. Additional data was available from MnRoads, Clyne et al. (2004), and the WesTrack test tracks, Ulditz et al. (2006) and Pellinen (2001). Both sites have documented results for field measured stiffness determined from FWD testing and laboratory characterisation of asphalt mixes undertaken using the dynamic modulus test.

3.9 Validation with Field Data

Because the NCAT data may not extend to true LLAP pavements (>50million cycles) the theoretical examination of pavement performance of both Australian and UK data (over a much longer period
of time) may prove beneficial to validate the results of the calibration undertaken from the NCAT results.

In the UK numerous sites have been identified that clearly achieve the LLAP status i.e. the pavement deflection is reducing with time. With the assistance of TRL and the members of the European LLAP Pavement Assessment Group (ELLPAG) details of some of the pavements were made available to the AAPA study.

Additionally, as part of the development of the development of the Structural Testing and Evaluation of Pavement (STEP) procedure for the assessment of pavement Remaining Life, the New South Wales Roads and Maritime Services developed an extensive database of sites throughout the state which can be interrogated to find LLAP.

With knowledge of the appropriate climatic data, the combination of these data sets can be used to validate the mechanistic models developed from the laboratory experiments, theoretical models and the calibrated NCAT data.

3.10 RMS STEP Database

The Roads and Maritime Services (RMS) of NSW developed an extensive database of pavement profiles, materials and condition of pavement across NSW as part of the development of the STEP modelling system. This database was made available to the AAPA APS-fl life project by RMS, to validate the presence of LLAP in Australia and aid in the development of criteria for the design of LLAP.

The RMS STEP database was developed with a number of surfaced flexible pavement types, namely:

- Sprayed seals over unbound granular base and sub-base layers
- Sprayed seals over granular layers including cemented layers
- Asphalt layers of variable thickness over unbound granular layers
- Asphalt layers over granular layers including stabilised layers

Of interest to the APS-fl project were the asphalt pavements over unbound granular base and sub-base layers and particularly those pavements with greater than 150mm of asphalt thickness.

3.10.1 Database Development

The development of the STEP database was a significant project for RMS and represents the most significant database of pavement structures, condition and materials in Australia. Although the STEP database only represent a single point in time, the distribution of pavement types, age and condition represent a significant tool for validating the presence and performance requirements of LLAP in Australia.
As part of the development of the database RMS determined the type, extent and severity of cracking on site by a visual assessment, the remaining life of the pavement by an experienced practitioner and the condition of the asphalt by an inspection of the asphalt obtained from the borehole investigation at each location.

a) Deflection Data: For each location FWD testing was undertaken. The FWD testing was undertaken at a target pressure of 700kPa, and measure the full deflection bowl and surface layer temperature.

b) Visual Assessment: The field information collected in the database included the surface cracking, if present, drainage type and estimates of the drainage effectiveness. The assessment of surface condition was undertaken by a visual inspection and noted the type of cracking (crocodile, longitudinal, block), if any, and the severity of the cracking (low, medium of high). Additionally an assessment was made of the remaining life of each location in terms of years by an experienced engineer.

c) Geotechnical Information: For each location a borehole investigation was undertaken, then borehole investigation document the material type, thickness and gave an assessment of the condition of each layer. i.e. for asphalt layers the investigation noted if the asphalt was sound, cracked, stripped etc.

d) Construction and Maintenance History: In preparation of the database RMS cross matched the chosen locations to the corporate RAMM asset database to obtain the construction year and maintenance history for each location. It was recognised by RMS that some of the maintenance history was lacking and not all maintenance activates would have been included.

e) Traffic Data: The traffic data in the database was in terms of AADT, traffic mix, ESA per commercial vehicle and growth rates. The traffic data was actual design traffic loading as opposed to `design traffic loading, i.e. without design safety factors.

For the RMS STEP database a location represents a single point on the road and not an area. In this way deflection data, materials, thickness can be exactly matched, eliminating risk associated with variability in the pavement.

For the validation study a subset of the STEP database was selected which included, only:

- Structural Asphalt pavements, with greater than 140mm of asphalt.
- Pavements which had not been overlaid or strengthened in their life.

The database of the subset of structural asphalts, with the assessed condition, deflection result, traffic loading, materials and maintenance history can be found in Appendix B of this report.

3.11 UK VALMON Site Data

The Highways Agency (HA) of the UK maintains a network of approximately 45 VALMON sites distributed across their trunk road network in England. The sites have been annually surveyed and
monitored for the past 10 years in order to build a database of pavement structural and functional data, which can be used to validate and monitor the Highways Agency pavement designs and assessment methods. The AAPA project made use of the deflection history data (FWD) for ten of these VALMON test sites on the most heavily-trafficked UK motorways. These sections were used as they were the same sections were examined as part of the TRL report 250 undertaken by Nunn et al. (1997) to establish the UK LLAP design procedure.

The data provided to AAPA consisted of a total of 33 sites monitored during the VALMON project which included 25 fully flexible and 8 flexible composite sites, for each site the location, construction and maintenance details, performance surveys (FWD and deflectograph), asphalt thicknesses, traffic data and construction specifications were provided by TRL. Details of the sections and deflection histories can be found in Yousefdoost (2015).
4 Dynamic Modulus Testing and Results

4.1 Reasons for Adopting the Dynamic Modulus Test

Given the ultimate goal of the material characterisation study was the development of modulus master curves to enable calculation of asphalt modulus at any temperature or vehicle speed, the dynamic modulus test was selected over other modulus tests such as the resilient modulus. This ability to calculate the modulus at any temperature or vehicle speed will offer a substantial improvement on the current Australian method which is based on a single standard laboratory test temperature and time of loading. In addition, the dynamic modulus test was selected as the primary material characterisation test for a number of reasons:

- Researchers such as Loulizi et al. (2006) have established that the dynamic modulus test provided a better characterisation of asphalt mixes than the resilient modulus test because of its full characterization of the mix over a range of temperatures and load frequencies.
- The dynamic modulus test and the resulting master curves are internationally accepted as being able to discriminate key asphalt performance properties. The NCHRP 9-19 project Witzcak (2002), concluded the dynamic modulus, and creep properties (flow number or flow time) had the best correlation with field performance, observed on major US field trials (WesTrack, MnRoad and the FHWA ALF).
- The dynamic modulus test has been used as a key material characterisation test at a number of international accelerated pavement test tracks, (FHWA ALF, NCAT, MnRoads and WesTrack). This enables the development of a quantitative process for the calibration of the performance of asphalt materials in the laboratory, against the performance of real pavements in the field.
- Because of the ability to model the asphalt mixes at any temperature and frequency, the results of the dynamic modulus test and subsequent master curves will enable the rational and quantitative assessment of asphalt materials used in the historical LLAP sections constructed in the Australia, the US and Europe at the specific temperatures and vehicle speeds encountered at those sites.

Given these benefits, the dynamic modulus test and the resulting master curves will facilitate the ultimate goal of the APS-FL project, which is, the structural analysis of the performing LLAP sections and the determination of the threshold design method. The finding of this threshold design method can then be transposed to different environmental conditions found in Australia, using the dynamic modulus master curves to form the basis of the development of LLAP design procedure.

4.2 Master Curve Development and Time Temperature Superposition

Because of the viscous component of asphalt mix, the material response is a function of both time of loading and temperature; the time-temperature dependency. For asphalt mixes it is common to represent this time temperature dependency by the construction of dynamic modulus master
curves. Master curves enable comparison of viscoelastic materials when tested using different loading times or frequencies and test temperatures. The construction of master curves is achieved by using the principle of time-temperature superposition and reducing all testing data to a common curve. The application of this principle typically involves the following steps:

- Experimentally determine the frequency dependent modulus curves at a number of temperatures.
- Calculation of a 'shift_factor to correlate the modulus over the temperature and frequency range, relative to a reference temperature
- Development of a master curve showing the effect of frequency for a wide range of frequencies.
- Use of the 'shift_factor to determine the temperature dependant moduli over the whole range of the master curve frequencies.
- The amount of shifting at each temperature described the temperature dependency of the mix; this is determined by the Time-Temperature superposition principal.
- Time-temperature superposition, is a well-established procedure which can be applied to asphalt mixes to either;
  - determine the temperature dependency of the asphalt
  - or, to expand frequency at a given temperature at which the material behaviour is being determined.

The time-temperature superposition principle can only be applied to 'Thermorheologically Simple_' materials, that is, to materials in which the shift factor is identical for all relaxation times. Fundamentally, the use of the time temperature superposition principle allows the prediction of long-term behaviour of asphalt from relatively short-term tests, as in the dynamic modulus test.

For Australia, it was agreed by the APS-fL Project Steering Committee, to use a reference temperature of 25°C, as opposed to the standard of 20°C used in the US. The 25°C temperature was selected to be consistent with current Australian characterisation methods. It was also agreed that dynamic modulus master curve should be modelled using a sigmoidal (S shaped) function, as recommended by Witczak (2002). However, due to the current debate over the definition of time in the dynamic modulus test, the sigmoidal function would be determined as a function of frequency, not time, as described by the following function:

\[
\log\left(\frac{f}{f_r}\right) = a + \frac{1}{1 + \left(\frac{f_r}{f}\right)^{1/2}}
\]

Equation 16

Where:

- \(f_r\) = reduced frequency at the reference temperature
- \(a\) = the minimum value of E*
\[ a + b = \text{the maximum value of } E^* \]

\[ g, d = \text{shape fitting parameters, determined through numerical optimisation of experimental data.} \]

In this process a shift factor, \( a_T \), is used to calculate the reduced frequency, \( f_r \), required to shift the dynamic modulus test results on the frequency scale to form a continuous curve at the 25°C reference temperature. The shift factor can be mathematically shown in Equation 17 following:

\[
\frac{f_r}{f} = \frac{f_{\text{ref}}}{f}
\]

Equation 17

Where:

\[ a_T = \text{shift factor} \]

\[ f = \text{frequency of loading at desired temperature} \]

\[ f_r = \text{reduced frequency of loading} \]

\[ T = \text{temperature} \]

While classical viscoelastic theory suggests a linear relationship between \( \log(a_T) \) and \( T \), Anderson et al. (1994) and research Pellinen (2001), has shown that a higher precision is achieved by the use of a second order polynomial relationship between the logarithm of the shift factor (\( \log(a_T) \)) and the temperature (\( T \)). The use of 2\textsuperscript{nd} order polynomial relationship can be further simplified by directly incorporating the reference temperature in the polynomial form. This polynomial shift factor approach was adopted by the Steering Committee as the method to be used for the AAPA study, as shown in Equation 18 following:

\[
\frac{f_{\text{ref}}}{f} = \frac{10^{a + b(T - T_{\text{ref}})}}{10^a + b(T - T_{\text{ref}})}
\]

Equation 18

Where:

\[ T = \text{temperature of interest} \]

\[ T_{\text{ref}} = 25^\circ C \]

\[ a, b = \text{coefficients of the polynomial} \]

This process of master curve development is shown graphically in Figure 21, which shows the measured dynamic modulus results of a typical mix as a function of frequency for four test temperatures.

Firstly, the results at the four individual test temperatures are shifted to the reference temperature (25°C) on the frequency scale to form a continuous curve as shown in Figure 22. Once the continuous curve is formed, the sigmoidal function is fitted to the measured data to construct a master curve. The curve is usually fitted by using a numerical optimisation procedure, such as the
Solver function in Excel, by minimising the sum of the squared errors between the measured and predicted values.

The amount of shifting required on the frequency axis to make the continuous curve is the shift factor. The amount of shifting for each temperature is then plotted against the temperature, as can be seen in Figure 22, to develop the temperature shift factor equation. The figure illustrates the higher precision of the polynomial shape of the shift factor relationship recommended by the Steering Committee. (All master curves in the AAPA study had $R^2$ values of greater than 0.98 and typically greater than 0.99)

4.2.1 Numerical Optimisation for Determination of Master Curves

When manually undertaken the process for accomplishing the horizontal shifting and sigmoidal function fitting for dynamic modulus, is a two phase process. However, in practice the two steps can be undertaken in one using a numerical optimisation process. In this process, initial trial values for the coefficients of the polynomial shift factor ($a$, $b$) and the sigmoidal function ($a$, $b$, $g$ and $d$) are assumed to calculate the dynamic modulus. This calculated modulus is then compared to the measured modulus and the squared error is obtained between calculated modulus and measured modulus as shown in Equation 19 following.

\[
\sum \left( \log \left( \frac{\mu_{\text{calc}}}{\mu_{\text{meas}}} \right) - \log \left( \frac{\mu_{\text{calc}}}{\mu_{\text{meas}}} \right) \right)^2
\]

Equation 19

The sum of squared errors is then set as the Objective Function, (OF) in the non-linear optimisation, with an objective of minimising OF by changing the coefficients of the polynomial shift factor ($a$, $b$) and the sigmoidal function ($a$, $b$, $g$ and $d$). In this study the fitting parameters were established using a coded procedure, using a polynomial error minimisation technique, due to the number of mixes. However, normally the objective function is minimised using the Solver Function in Excel.
Figure 21 Construction of Dynamic Modulus Master curve and Temperature Shift Factor Function

Figure 22 Temperature Shift Factor Function
4.3 Master Curves and Dynamic Modulus Test Results

4.3.1 Master Curve Fitting Parameters

The dynamic modulus results for the 28 mixes tested were used to generate master curves for each individual mix, which would cover the spectrum of the pavement temperature and vehicle speeds encountered under Australian conditions. The master curves were all created at a reference temperature (25°C) which allows the stiffness of the Australian mixes to be viewed without temperature as a variable. This method of analysis allows for relative comparisons to be made between multiple mixes.

In solving the master curves, all fitting parameters were free to be solved to obtain the best fit between the measured and predicted data (i.e. not constrained).

The fitted parameters for the sigmoidal and the polynomial shifting parameters for the 28 different mixes examined in the study are shown in Table 4 and Table 5 following.

The fitted parameters for the sigmoidal function and the temperature shifting factor for the 28 different mixes examined in the study can be seen in Table 4 and 5 following. While the master curves can be seen graphically in Figure 23 and Figure 24 following for the 14 and 20mm mixes respectively. In the figures the red curves are for C320 mixes, green are AR450, yellow are C600, blue are A15E and purple are Multigrade mixes.

Table 4 Master Curve Fitting Parameters

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<tr>
<td></td>
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<td>11119</td>
<td>4.4</td>
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<tr>
<td></td>
<td>Multi</td>
<td>12013</td>
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<td>20</td>
<td>AR450</td>
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<td>11123</td>
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<td></td>
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<td>C320</td>
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<td>30</td>
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</table>

**Table 5 Polynomial Shift Factors**
<p>| | | | | | |</p>
<table>
<thead>
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<td>0.0006</td>
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<tr>
<td>C600</td>
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<td>0.0003</td>
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</tr>
<tr>
<td>12087</td>
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<td>0</td>
<td>0.0004</td>
<td>-0.107</td>
<td></td>
</tr>
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<td>12111</td>
<td>4.3</td>
<td>0</td>
<td>0.0004</td>
<td>-0.108</td>
<td></td>
</tr>
<tr>
<td>Multi</td>
<td>12017</td>
<td>4.3</td>
<td>15</td>
<td>0.0004</td>
<td>-0.111</td>
</tr>
</tbody>
</table>

Figure 23 Master curves 14mm
The initial examination of the results shows that the minimum modulus values as expected increase with an increase in binder grade, with C320 mixes being lower than AR450, which are lower than C600 and Multigrade binders. As expected nominal 20mm mixes are also typically higher than the 14mm mixes. This is consistent with the latest version of the Witczak model, with Bari (2006) finding the minimum modulus value, \(a\), was affected by aggregate gradation, volume of air, volume of binder and binder stiffness. This finding suggests the minimum modulus value appears best at distinguishing between different binder grades. As there was little to no change in air void level of typical Australian mixes (typically 5%), the effect of air void level on the minimum modulus value could not be assessed.

Unexpectedly, no correlation was found between the minimum modulus and RAP content indicating that at current RAP contents, RAP content has little effect on the minimum modulus value and therefore overall modulus values (plant characteristics appear to be more important). While the minimum modulus value appears to be influenced by effective binder content, and the amount of filler, due to the relatively small change in the effective binder volume, the effects to changes in the volume of binder are small and cannot be easily assessed.

The \(b\) parameter or the maximum modulus value parameter appears not to be sensitive to changes in binder grades for conventional (neat) binders or maximum aggregate size, which is the same
finding as Bari who found that the \( b \) value was a function of volumetric properties of the mix and finer fraction of the gradation.

The examination of the relative shape of the master curves tends to indicate that for the current Australian mixes, the design method, aggregate source and the relatively small variance in volumetric properties appears to have little effect on the shape of the master curve, with results of the \( g \) and \( d \) factors showing little change within binder types.

For conventional binders, the shape factors (\( g \) and \( d \)) appear to be relatively consistent regardless of the grade of binder. However, this is not the case for multi-grade and A15E, which have a different shape from the conventional binders indicating lower time-temperature susceptibility. This is somewhat consistent with the findings of Bari (2006), who found \( g \) the shape factor was a function of binder properties only. However, for Australian binders, this change of shape is only evident when comparing conventional and modified binders. It is clear that the shape factor, \( g \), should not be constant across different binder classes.

On first examination, due to the consistency of these parameters within a binder grade and nominal aggregate size, it may be practical for design purposes to define the whole master curve using one point only and use this point to shift the master curve either up or down based on a limited number of test points. While this will not be as accurate as the measurement of the whole master curve and risk may be associated with its use, it may be a practical solution for level 2 analysis, with level 1 being typical modulus values and level 3 being the measurement of the whole master curve.

4.4 Grouping of Australian Mixes

The initial intent of the APS-FL project was to validate and calibrate one of the two well-known prediction models for dynamic modulus. However this may not be necessary. Unlike the US, Australia has limited grades of binder and these grades are controlled under an Australian Standard. Also, as shown in Section 3, the volumetric properties and gradations for typical Australian mixes do not vary to a great extent between suppliers and even between states, regardless of the design method used or specification. Given the consistency of; gradation, volumetric properties, and the consistency of the shape of the master curve for a given binder type and nominal mix size, it may not be necessary in Australia for practical implementation to develop complex master curve equations such as the Witczak or the Hirsch model for routine pavement designs. It may be more relevant to group or sub group mixes to have typical modulus values.

To investigate the applicability of this approach the dynamic modulus results were grouped by nominal aggregate size (14 and 20mm) and binder grade (C320, AR 450 and C600) to produce six subgroups within the study. The data from each of these subgroups was then used to create a typical master curve for all results with that subgroup.
For each of these subgroups master curves were generated by minimising the squared error between the measured and predicted data in the log space i.e. \((\log E_{\text{meas}} - \log E_{\text{pred}})^2\) using the previously described sigmoidal function and the polynomial temperature shift factor. For all master curves all parameters were free to be solved to obtain the best fit between the measured and predicted data.

Figure 27 following, shows a typical result of the subgroup of mixes, in this case mixes with a nominal aggregate size of 20mm and an AR450 binder. RAP contents in these mixes varied between 0 and 20%.

The validity of using these typical master curves for design will understandably depend on the accuracy of the typical master curves in the prediction of modulus and the degree in which confidence can be obtained around those results.

To investigate this validity, the relative accuracy of the proposed approach was compared against the accuracy of the two well-known models for the prediction of dynamic modulus, the Witczak and Hirsch models. The accuracy obtained from the prediction of modulus using the grouped results was compared against the published accuracy of both the Witczak and Hirsch models in terms of the coefficient of determination and the standard error \((R^2, S_e)\).

As shown by Bari (2006), the current versions of the Witczak and Hirsch models have a \(R^2\) of 0.9 and 0.92 respectively in the log space and 0.8 in the arithmetic space for the Witczak model. Given
the range of data used in the two published data sets (Witczak/Hirsch) and the AAPA database are very similar (500 to 25000MPa), comparison of the coefficient of determination alone will provide a good comparison of the relative accuracy of the two approaches.

The accuracy of the grouping approach can be seen in Figure 26 following, which shows the measured modulus against and the typical modulus master curve for the six grouped mixes, grouped by nominal aggregate size, 14 and 20mm mixes and the three primary binder classes used in Australia, C320, AR450 and C600. The variability of the measured modulus data for the grouped mixes can be seen in Table 6 following.

Figure 26 Accuracy of Grouping Approach
As can be seen in both the figure and table the grouping of results provided an "excellent fit with the coefficient of determination being 0.98. This accuracy is significantly better than both the Witczak and Hirsch models (0.9 and 0.92) in both the log and arithmetic space.

These findings are significantly beneficial to Australia, indicating that Australia can achieve a higher degree of confidence in the predicted modulus values by grouping common mixes together, than from the use of complex model forms, such as the Hirsch and Witczak models which require volumetric properties, binder shear modulus and aggregate gradation, all of which will not be typically available the consultant at the time of design to.

4.5 Typical Master Curves and Development of Confidence Intervals

As already established the modulus of the grouped mixes will vary throughout Australia in production due to use of RAP, binder source, and effective binder content amongst others factors. At least initially this variation will not be known to the pavement designer who will not know the binder source, aggregate gradation and percentage of RAP. The designer will generally only specify a grade of binder and a nominal aggregate size. While the results of the grouping approach showed that the variation prediction was small compared to published prediction models, it does present some risk in the design process. Therefore the designer should consider the risk, or the level of confidence required from the modulus, when assigning a design modulus for the purposes of pavement design.

One of the main benefits of the grouping approach is that this risk can be rationally assessed as confidence limits can be developed around the prediction of modulus. These rational confidence levels can be established because, like most engineering parameters the prediction of modulus should follow a normal distribution and by using this distribution and the variation or standard error

### Table 6 Grouping Accuracy

<table>
<thead>
<tr>
<th>Nominal Size (mm)</th>
<th>Binder Type</th>
<th>Arithmetic Space</th>
<th>Log Space</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Coefficient of Determination ($R^2$)</td>
<td>Standard Error (Se)</td>
</tr>
<tr>
<td>14</td>
<td>C320</td>
<td>0.97</td>
<td>1457</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>0.95</td>
<td>1551</td>
</tr>
<tr>
<td></td>
<td>A15E</td>
<td>0.93</td>
<td>1468</td>
</tr>
<tr>
<td></td>
<td>Multigrade</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>C320</td>
<td>0.97</td>
<td>1393</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>0.97</td>
<td>1315</td>
</tr>
<tr>
<td></td>
<td>C600</td>
<td>0.98</td>
<td>1139</td>
</tr>
<tr>
<td></td>
<td>Multigrade</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>
in the prediction, it is possible to assign confidence to the prediction of modulus values. This is a significant benefit over the typical median values developed by standard predictive models.

4.5.1 Distribution of Errors around Master Curve

Like most engineering parameters the prediction of dynamic modulus is assumed to follow a normal distribution. This assumption can be easily checked by plotting a histogram of the residuals or errors (difference between measured and predicted values). If the results are normally distributed a plot looking like a normal distribution centred on zero should be obtained.

For the prediction of dynamic modulus from the grouped data, it was found that the residuals followed a normal distribution where the residual was in the log space i.e. \( \log(E_{\text{measured}}) - \log(E_{\text{predicted}}) \). A typical plot of the residuals, in the log space, is shown in Figure 27 in this case for the AC14 C320 mixes.

![Figure 27 Typical Modulus Grouping Results and distribution of errors around the master curve](image)

As can be seen in the figure, the shape of the distribution closely follows that of a normal distribution with the standard deviation equal to that of the standard error. Therefore, for practical purposes the residuals can be assumed to follow a normal distribution in the log space. This finding is important in developing confidence in the prediction of results, as it shows that a normal distribution can be placed directly around the sigmoidal function in the log space.
4.6 Development of Confidence Based Master Curves

Given the residual errors around the master curve can be assumed to follow a normal distribution with the standard error equal to the standard deviation, it is possible to establish confidence limits, for each of the proposed sub-groups of Australian production mixes. The practical result of this will be to enable the designer to say they are x% confident that the adopted modulus value used in design will not exceed the design value.

As the residuals are normally distributed around the master curve, for practical purposes the master curve can be simply shifted up and down on the modulus axis to obtain any degree of confidence. This means that for design purposes confidence limits can be simply established by varying the a parameter to shift the curve up or down to cover a greater or lesser number of results or simply assigning the normal distribution to the minimum modulus value. Because of the limited sample size used in the grouping of mixes, it was decided that a student’s t distribution would give a better measure of confidence than that of the normal distribution. The student’s t distribution was used in preference to the normal distribution to account for the limited observations obtained from the normal distribution for estimating the confidence value of the a parameter.

The student’s t distribution and standard error were then used to determine the minimum modulus value, a, which would give 50, 75 and 95% confidence in prediction of modulus for all sub groups of Australian mixes. The full listing of confidence values and master curves for each subgroup can be found in Table 7 following while Figure 28 following shows one of the typical confidence interval plots, in this case AC 14 AR 450 mix, with the results for each subgroup being found in Appendix C.
Table 7 Master Curve Fitting Parameter

<table>
<thead>
<tr>
<th>Nominal Size (mm)</th>
<th>Binder Type</th>
<th>Master Curve Sigmoidal Fitting Parameters</th>
<th>Confidence Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>b</td>
<td>g</td>
</tr>
<tr>
<td>14</td>
<td>C320</td>
<td>3.184</td>
<td>-1.287</td>
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<tr>
<td></td>
<td>AR450</td>
<td>3.080</td>
<td>-1.384</td>
</tr>
<tr>
<td></td>
<td>A15E</td>
<td>3.446</td>
<td>-0.827</td>
</tr>
<tr>
<td></td>
<td>Multi</td>
<td>2.760</td>
<td>-1.040</td>
</tr>
<tr>
<td>20</td>
<td>C320</td>
<td>3.132</td>
<td>-1.276</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>3.075</td>
<td>-1.417</td>
</tr>
<tr>
<td></td>
<td>C600</td>
<td>3.068</td>
<td>-1.377</td>
</tr>
<tr>
<td></td>
<td>Multi</td>
<td>2.702</td>
<td>-1.206</td>
</tr>
</tbody>
</table>

Using this figure the designer can easily establish the modulus of a standard Australian mix at any temperature, frequency of loading and confidence level.

For example, and shown on Figure 28, consider if the designer wants to establish the modulus of the mix at a frequency of 12 Hz and 25°C. The first step would usually be to calculate the reduced frequency. However because the temperature required is the reference temperature, no shift is required on the frequency axis and the reduced frequency is the frequency required. The designer then simply selects 12 Hz on the horizontal scale and follows the value down till it meets the desired confidence master curve. The value is then read off the vertical axis, in this case 3800 MPa at 95% confidence or 4500 MPa at 50% confidence.

4.7 Correlation with NCAT

To establish if there was any bias or variability between the Shear-box compacted samples and samples compacted using the US Superpave Gyratory Compactor (SGC) and most importantly, whether Australia can use the results of NCAT testing to develop and calibrate performance models. Four Shear-box compacted samples were shipped to NCAT for comparison testing. In addition, two loose mixes were sent to NCAT for compaction in the SGC. This was undertaken to enable a direct comparison to be made between the Shear-box compacted samples and the SGC samples. It needs to be noted, that because of the time between compaction and testing the Shear-box compacted samples sent to the NCAT could have been up to 3 month old.

Table 8 following summarises the results of testing undertaken by NCAT on the prefabricated cylinders and the NCAT fabricated cylinders.
### Table 8 NCAT Modulus Results Australian Mixes

<table>
<thead>
<tr>
<th>Temp, °C</th>
<th>Freq, Hz</th>
<th>Dynamic Modulus (MPa)</th>
<th>Pre-Fabricated Cylinders</th>
<th>NCAT Fabricated Cylinders</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>BKR</td>
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<tr>
<td>4</td>
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<td>21858</td>
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<td>22319</td>
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<td>10</td>
<td>20413</td>
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<td>21331</td>
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<tr>
<td>4</td>
<td>5</td>
<td>19293</td>
<td>12231</td>
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<tr>
<td>4</td>
<td>1</td>
<td>16727</td>
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<td>7833</td>
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<td>10</td>
<td>6602</td>
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<td>5</td>
<td>5785</td>
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<td>4177</td>
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<td>3065</td>
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<td>1589</td>
<td>767</td>
<td>987</td>
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<td>0.5</td>
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<td>937</td>
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<tr>
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<td>1373</td>
<td>863</td>
<td>829</td>
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</table>

For the two mixes where a direct comparison of the NCAT testing could be undertaken (Shear-box and SGC), similar behaviours were observed in the dynamic modulus results at different frequencies and temperatures. However, for both mixes, the shear-box compacted specimens were slightly stiffer than the SGC fabricated specimens across the full range of temperatures and frequencies. For both mixes, there was an increase in stiffness of typically 10% between the Shear-box compacted samples in comparison to the SGC prepared samples. For each mix, the master
curves behaved in identical fashion over the full range of temperatures and frequencies but were separated by an offset. This separation in the curves could be caused by several variables. Initially, it was concluded that the most likely difference was due to the way the specimens were handled and compacted in the laboratory during the specimen fabrication process. However, the direct comparison between samples tested in Australia just after fabrication and by NCAT after fabrication in the SGC, shows this is not the case.

The direct comparison between the two test methods can be seen in Figure 29 following, which shows the dynamic modulus results obtained at NCAT against that of the dynamic modulus results obtained in the AAPA study of the shear-box compacted samples.

The direct comparison between the two approaches was undertaken by comparing the dynamic modulus results obtained at NCAT against that of the dynamic modulus results obtained in the AAPA study using the shear-box compacted samples. What was quickly noticed is that the trend was the same as the trend found in the direct comparison undertaken by NCAT, that the older samples (results obtained by NCAT) were about 10% higher than that recorded in the AAPA study. Given the trend was the same as found by NCAT, the results would indicate a slight ageing of the samples and that the stiffness has increased in the period between fabrication and testing.
This ageing was confirmed when the results of the NCAT prepared SGC prepared samples were compared directly to the initial results obtained in the AAPA study, as shown in and Figure 30 and Figure 31 following. In this case the figure shows that the results for all practical purposes, for the two master curves, are identical.

Figure 30 NCAT and AAPA Modulus Results mix 14-10
This would indicate that the differences seen in the comparison of the results obtained by NCAT between the SGC samples and Shear-box compacted samples, as well as the difference seen between the NCAT tested Shear-box samples and the AAPA results are primarily due to a slight ageing of the samples.

The comparison of the results on the two un-aged samples show that the difference in the compaction method and test method have no practical influence on the dynamic modulus results, with identical results obtained. Therefore, it can be concluded that the APS-fL project can utilise NCAT modulus results and performance data with confidence for those sites where dynamic modulus testing has been undertaken to:

- Correlate dynamic modulus estimates from back analysis of deflection data
- Validate measured strain and predicted strain using Linear Elastic Analysis
- And, develop threshold strain levels based on calibrated strain and field performance.

4.8 Conclusions and Recommendations

International research has established that the dynamic modulus test provides a better characterisation of an asphalt mix over the resilient and other modulus tests because of its ability to fully characterise a mix over a range of temperatures and load frequencies. Additionally, the dynamic modulus test is internationally accepted as being able to discriminate key asphalt performance properties. For these reasons and most importantly, the ability to link dynamic
modulus to a number field studies, AAPA selected the dynamic modulus test to undertake a full characterisation of standard Australian production mixes for the APS-fL project.

Examination of the gradation and volumetric properties of Australian SRA production mixes shows that despite the variability in the design methods (Marshall, gyratory and Superpave) and the differing compaction efforts, all mixes fit into a very small volumetric window. Additionally it was found the design gradation of all standard Australian production mixes closely follow the maximum density line, with nearly all mixes being slightly coarse graded, indicating that distinguishing between Australian mixes based on gradation and volumetric properties may be difficult.

The comparison of the results obtained by AAPA and NCAT on two un-aged samples showed that the difference in the compaction method and test method had no influence on the measured dynamic modulus results, with identical results obtained. Therefore, it can be concluded that the APS-fL project can utilise the results of NCAT testing for modulus and the performance data with confidence to:

- Correlate dynamic modulus estimates from back analysis of deflection data
- Validate measured strain and predicted strain using linear elastic analysis
- And, develop fatigue endurance limits.

Examination of the master curves of standard Australian production mixes suggests the minimum modulus value appears to be the best at distinguishing between binders and nominal aggregate size. As there is little difference in the volumetrics of Australian mixes, no significance could be found in air void levels or binder contents within sub mix types. Unexpectedly, no correlation was found between the minimum modulus and RAP content, indicating that at current RAP levels, RAP has little effect on the minimum modulus value and therefore overall modulus values. The results showed that most likely because of the small variance in aggregate gradation and volumetric properties there was no change in the shape of the master curve within grouped binder types and nominal aggregate size.

Because of the consistency of the master curve for a given binder type and nominal mix size, for practical implementation it was found that it was not necessary to develop complex master curve equations for routine pavement designs. The results of grouping of Australian mixes showed that Australia can achieve a higher degree of accuracy by grouping common mixes than from the use of complex models such as the Witczak or the Hirsch models.

It was found that because the residuals in the prediction of modulus followed a normal distribution with the standard deviation equal to that of the standard error, confidence could be established from the grouped data by simply varying the minimum modulus data to move the dynamic modulus curve down the modulus scale. By doing this it was shown that confidence level master curves could be established for the nominal 14 and 20mm mixes and the three primary binder classes used in Australia, C320, AR450 and C600.
5 Conversion between Laboratory and Field Response

5.1 Introduction
The AAPA APS-fL procedure proposes the use of the frequency-temperature dependent dynamic master curves as the method for determining the modulus of the asphalt under field loading conditions. Dynamic modulus helps to define the viscoelastic nature of asphalt mixes by quantifying the effects of temperature and frequency on stiffness under dynamic loading. This effect of frequency and temperature is necessary to accurately predict the pavement responses to varying load speeds and temperatures throughout the pavement’s cross-section.

Currently, limited research has been undertaken to compare the behaviour of asphalt mixes in the laboratory using the dynamic modulus test and the behaviour of asphalt mixes in the field using measured response. Presently, no information exists in Australia to enable the comparison of modulus determined in the field from FWD testing with that of dynamic modulus test results. While this information does not exist in Australia, Phase II and Phase IV of testing at the NCAT test track provides a valuable source of information for the comparison. As part of the APS-fL project a direct link has been established between NCAT dynamic modulus testing and the AAPA dynamic modulus database, showing there to be no difference between dynamic modulus results determined in the AAPA study and modulus determined by NCAT. Therefore the results obtained from a comparison of dynamic modulus and field modulus and ultimately field strain at NCAT should be directly transportable to Australia.

A main objective of the study was the determination of the stiffness of the asphalt mixes which can be used in multi-layer elastic model to accurately predict the pavement response under traffic load. There are two ways of accomplishing this objective.

- First, pavement responses can be predicted by multi-layer elastic model using different stiffness and then compared with measured responses.
- The second method is to back-calculate the layer moduli from measured pavement responses and compare the back-calculated moduli to laboratory determined moduli.

5.1.1 Data for Comparison
As part of the experimental plan of the Phase II and Phase IV test cycles at the NCAT test track, structural testing using FWD was undertaken on known pavement structures. This measured response was used to determine the effective modulus of the combined asphalt layers throughout each phase of the test cycles. Furthermore, for the structural sections of the test track of both Phase II and Phase IV a series of laboratory dynamic modulus tests were performed on each of the individual mixes used in the structural experiment sections.

In addition to the NCAT, there are a number of sources of valuable information in the literature which can be used to compare laboratory dynamic modulus to field modulus, with data also being
available from both MnRoads and the WesTrack test tracks. Both of these have documented results for field stiffness determined from FWD testing and laboratory characterisation of asphalt mixes undertaken using the dynamic modulus test.

5.2 Factors Effecting Conversion

5.2.1 Time Frequency Conversion

There is currently significant debate amongst researchers on how frequency is related to time in the dynamic modulus test. The two primary schools of thought are the angular frequency approach vs. the pulse frequency approach. Researchers such as Dongre et al. (2006) recommend the angular frequency approach, \( t = \frac{1}{w} \), while researchers such as Katicha et al. (2008) recommend the pulse frequency, \( t = \frac{1}{f} \) approach.

It appears that the earliest use of the angular frequency approach, \( t = \frac{1}{w} \) for asphalt mixes was from the work undertaken by Papazianin at the First International Conference on the Structural Design of Asphalt Pavements (1962). This approach was then adopted by Shell for their development of a ME pavement design procedure, subsequently adopted as the basis of the AGPT002 (2012) asphalt characterisation method. However, the angular frequency approach has not been universally adopted by all design procedures, with the US MEPDG following the \( t = \frac{1}{f} \) approach.

The reason the \( t = \frac{1}{w} \) approach is recommended by some researchers is based on the solution of the Inverse Fourier Transformation (IFT) which is required to convert from the frequency to time domain, to determine the relaxation modulus, \( E(t) \), from angular frequency testing, from the storage modulus \( E' \), as follows, Ferry (1980).

\[
\text{Equation 20}
\]

\[
H(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} F^{-1} \left( \frac{w}{2\pi} \right) e^{iwt} \, dw
\]

Where,

- \( w \) is angular frequency in rad/s
- \( f \) is the cyclic frequency in Hz
- \( t \) is loading time in seconds
- \( H(t) \) is the continuous spectrum of the relaxation time
- \( F^{-1} \) is the Inverse Fourier Transformation

Dongre (2006) found that the exact solution of the IFT to calculate relaxation modulus from the dynamic modulus test was \( t = \frac{1}{w} \). This is somewhat contrary to the early recommendations of Van der Poel (1954) who suggested the conversion was only approximate. Notwithstanding this, Dongre did not establish that the testing in dynamic modulus test was an angular frequency, which would require the above conversion. The issue is still not resolved amongst researchers.
5.2.2 Vehicle Speed and Load Frequency

The case of the FWD loading is relatively simple, being a relatively fixed load time with depth. This however is not the case for a moving load which varies with depth and vehicle speed. Therefore in order to compare dynamic modulus test data with the response of a pavement in the field, the equivalent loading pulse time of the design vehicle needs to be determined. Early research in mechanistic pavement design identified two approaches to model the pulse time within an asphalt layer; time as a function of the strain pulse or time as a function of stress pulse.

The approach to modelling time as a function of strain pulse was first hypothesised by Coffman (1967) who believed that to translate laboratory tests to field modulus the cycle length and the phase shift needed to be known. Coffman based his recommendation on the correlation of field testing and analytical work and determined that cycle length was a function of the measurement of field strains under a moving load. Coffman found that it was possible to fit a sine wave to the measured deflection and therefore determine the cycle length. Coffman found that 6 feet was a good choice for average cycle length and also that the use of higher frequencies for upper most pavement layer layers and lower frequencies for the lower pavement layers was not justified.

The approach of using the stress distribution appears to have been first postulated Brown (1973) based on work done by Barksdale (1971). Brown’s approach was to use the use the average loading time for both vertical and horizontal stress pulses throughout the asphalt layer. The equation developed by Brown using the average stress loading time is shown in Equation 21 following:

\[
\text{Equation 21}
\]

Where;

- \( t \) = loading time (sec)
- \( h \) = thickness (mm)
- \( V \) = vehicle velocity (km/hr.)

Brown’s method has subsequently gained widespread acceptance and forms the basis of various different design processes throughout the world, including the current Austroads design procedure.

Ullditz (2006) proposed an alternative approach to using stress distributions to determine loading pulse, using a simplified model incorporating tyre contact area with the load being distributed at 45° from the tyre radius. The model proposed by Ullditz is shown conceptually in Figure 32 and numerically in Equation 22 following:
Equation 22
\[
\frac{S}{h} = 200 \quad \text{Equation 22}
\]

Where;

\[ S = \text{the vehicle speed in (mm/s)} \]
\[ h = \text{the desired thickness} \]
\[ 200 = \text{is assumed to be the length of the tyre contact} \]

The US Mechanistic Empirical Design Guide (MEPDG) uses a similar approach to that proposed by Ullditz; however, it does not use a fixed stress path. The approach makes use of Odemarks approach, to transform asphalt layers into and equivalent thickness of subgrade material over the actual subgrade layer. These results in an increase in the asphalt layer thicknesses and in theory should result in significantly longer loading times than the approach adopted by Ullditz and Brown.

5.2.3 Empirical Relationships to Frequency

Yager (1974) used the early work by Coffman to relate vehicle speed to test frequency. Using the 6 foot load pulse, Yager determined frequencies of 1, 6 and 12Hz related to speeds of 10, 40 and 80km/hr, respectively. Jacobs et al. (1996) recommended a loading frequency of 8Hz to correspond to a vehicle speed of 60km/hr. In NCHRP report 465 Witzcak et al. (2002) recommended 10Hz be used as the frequency for highway speed and 0.1Hz for creep intersection traffic. The Asphalt Institute assumes a value of 10Hz regardless of the conditions Asphalt Institute (1999). These empirical recommendations would indicate that frequencies in the range of 0.1 to 12Hz are representative of vehicle speeds for the purpose of pavement design. However, none appear to be validated to field performance.

As previously mentioned, the US MEPDG uses the relationship of frequency as the reciprocal of pulse time (\(1/t\)) to convert between frequency and load pulse time in the field. Using this method,
frequencies in the range of 1 to 100Hz, would be expected if a stress pulse was used. Clearly this is a long way from the empirical recommended values.

The use of the angular frequency approach would give frequency ranges of 0.1 to 14Hz, which are in line with empirical recommendations. The belief that the frequencies used in the MEPDG are too high, is supported by a review of the accuracy of the MEPDG by NCHRP 2006, which found that the use of the cyclic frequency approach lead to unrealistically high modulus values.

5.2.4 Dynamic Modulus and Field Strain

It is not surprising, given the debate about;

- the conversion between time and frequency in the dynamic modulus test,
- the effect of depth on load pulses times and
- the effect of stress distributions within the pavement structure,

that currently little published work has been found to determine a direct correlation between laboratory modulus, field modulus and strain.

Some work relating strain to dynamic modulus has been undertaken by Bayat et al. (2005). In a series of field-controlled wheel load tests, Bayat found that longitudinal strain in asphalt followed an exponential relationship to pavement mid-depth temperature, when speed was constant. This finding is similar to the findings of Timm et al. (2006). However, Timm found the relationship was a power relationship at the NCAT test track. Bayat found that dynamic modulus was directly related to measured strain and found that dynamic modulus was inversely proportional to the field measured asphalt longitudinal strains and found `dynamic modulus test replicates pavement field response.. However neither Bayat nor Timm’s research recommended a final method for the conversion of laboratory measured modulus to field modulus.

5.3 Calibration Plan

There appears to be no easy solution to the debate about the exact conversion between frequency in the dynamic modulus test and time of loading in pavement structures, with the solution requiring complex mathematics which currently can only be numerically approximated, does not appear to be easily solved and are even then still open for debate. The mathematical solution of this problem was determined to be outside of the scope of this project. It was accordingly decided to disregard the solution of the mathematics and to develop a direct conversion between dynamic modulus and field measured modulus using FWD testing, and the results of dynamic modulus testing undertaken at NCAT, WesTrack and MnRoads. The relationship between dynamic modulus and field modulus would be used to develop and validate a direct conversion between dynamic modulus frequency and field pulse loading, skipping the solution of the mathematics.
In this way, conversion factors can be established which enable the conversion between laboratory and field modulus and pavement temperature, without resorting to solving the complex differential equations (where no exact solutions exist).

This conversion can then be used to validate strain results obtained at the NCAT test track to strains predicted by the use of layered elastic analysis and the converted modulus and determine calibration coefficients, if any.

To determine the conversion between dynamic modulus, back-calculated FWD results and strains under wheel loading, a series of stepwise numerical optimisation procedures was undertaken. Based on the results of this analysis, a series of conclusions and recommendations can then be drawn to determine strains under a moving vehicle load using the results of dynamic modulus testing.

5.4 Optimisation Approach

Due to:

- the complexity of the problem being solved
- the number of possible combinations of each variable, and
- most importantly, the requirement for the nonlinear optimisation to have seed values which need to be relatively correct to ensure the solver function converges on the correct optimal solution,

a three stage sequential optimisation approach was used in the analysis. The three stage sequential optimisation is as outlined in Table 9 following. The Solver function of Excel was used for the nonlinear optimisation, to assess the combination and contributions of each variable and therefore, optimising the calibration of modulus and the prediction of strain under a moving vehicle from dynamic modulus.

<table>
<thead>
<tr>
<th>Process</th>
<th>Data Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1- Calibrate laboratory field modulus interconversion</td>
<td>Calibrate Frequency Interconversion</td>
</tr>
<tr>
<td></td>
<td>Calibrate Temperature Profile</td>
</tr>
<tr>
<td></td>
<td>Validate findings with WesTrac and MnRoads</td>
</tr>
<tr>
<td>Stage 2- Calibrate frequency under moving load</td>
<td>Calibrate load pulse width</td>
</tr>
<tr>
<td></td>
<td>Calibrate effect of frequency with depth</td>
</tr>
</tbody>
</table>

Table 9 Optimisation Approach
The first stage was to use the results of the field modulus from the FWD to determine the frequency conversion between the laboratory modulus and field modulus and determine the temperature correction required to account for any difference between mid-depth measured on site and the effective temperature within the asphalt layers.

The second stage was to calibrate for any static/dynamic effects, temperature effects, determine the load pulse width and the corresponding stress path within the pavement using an equivalent single layer of asphalt.

The final stage was to optimise the model for use with multi-layer asphalt pavements.

5.4.1 Field Modulus Measurements

The first stage of the proposed optimisation process requires the use of field measured modulus values from FWD testing combined with the results of dynamic modulus testing. Three sources of information were identified in the literature to accomplish this; NCAT, MnRoads and WesTrack.

At the NCAT test track, in-service modulus was determined from the results of FWD testing undertaken using a Dynatest FWD. The field testing process undertaken at NCAT is described in depth by Timm et al. (2003). The back-calculation of modulus from the results of the FWD testing was accomplished using Evercalc. Timm (2005) described the several simulated cross sections that were attempted to determine the best grouping of the pavement layers for back-calculation to determine the optimal cross section. The optimal cross section was determined to have the aggregate base and fill material combined into a single layer.

The second source of field modulus was taken from the MnRoad test track. Since construction in August 1999 (Cells 33, and 34), FWD testing has been performed several times on the cells at MnRoad, Clyne (2004). Several locations have been tested in each cell, and load, deflection, and temperature data has been collected with each test. For MnRoads these results were then used to back-calculate the modulus of the asphalt pavement. The back-calculation method again utilised Evercalc.

The Westrack test track also utilised FWD testing for field validation of modulus. However WesTrack utilised EImod 5 for the purpose of back-calculation of layer moduli. Back-calculation of the asphalt layer moduli was done for all of the FWD test series, and for the test positions between the wheel paths as well as in the right wheel path.
5.4.2 Effective Layer Modulus

As mentioned back-calculation nearly always considers the entire depth of the asphalt layers, while the dynamic modulus test considers each mix separately. This was the case for the NCAT test track where different asphalt layers were used within the pavement structure.

As the initial optimisation process proposes the use of a single layer for the conversion of laboratory measured dynamic modulus to field measured modulus from FWD testing, the individual asphalt layers will need to be combined into a single equivalent asphalt layer. The individual asphalt layers were combined into an equivalent single asphalt layer using the concept of conservation of the moment of inertia (method of equivalent thickness) via Equation 23 following.

\[
\begin{align*}
E_{\text{eff}} &= \frac{\sum h_i E_i}{\sum h_i} \\
E_{\text{eff}} &= \text{effective modulus of the combined layers} \\
h_i &= \text{the thickness of layer } i \\
E_i &= \text{the modulus of layer } i \\
n &= \text{the number of asphalt layers}
\end{align*}
\]

Where:

In this process each moduli result from the dynamic modulus test, (from Timm (2003) and Vargas-Nordcbeck (2013)), at each frequency and test temperature were combined using the as constructed layer thickness to determine the effective modulus at each test frequency and test temperature. These effective modulus values were then used to construct an effective dynamic modulus master curve, using the sigmoidal function and polynomial shift factor as described in Section 4.3.

The master curve fitting parameters for the resulting master curve for the combined layers are shown in Table 10 following. The sites chosen for the analysis were 6 NCAT test cells where little damage had occurred. Additionally, shown in Table 10 are the master curve parameters for the two MnRoad test sites (cell 33 and 34) and the 6 Westrack test sites, with dynamic modulus results being obtained from Clyne (2004) and Pellian (2001).
Table 10 Modulus Calibration Master curve Fitting Parameters

<table>
<thead>
<tr>
<th>Test Cell</th>
<th>Temperature Shift Factors (T&lt;sub&gt;ref&lt;/sub&gt; = 20°C)</th>
<th>Sigmoidal Fitting Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>B</td>
</tr>
<tr>
<td>N3-Phase III</td>
<td>0.0003</td>
<td>-0.116</td>
</tr>
<tr>
<td>N5 Modified-Phase II</td>
<td>0.0004</td>
<td>-0.119</td>
</tr>
<tr>
<td>N7-SMA</td>
<td>0.0004</td>
<td>-0.122</td>
</tr>
<tr>
<td>S9-Control</td>
<td>0.0004</td>
<td>-0.115</td>
</tr>
<tr>
<td>S10 WMA-F</td>
<td>0.0004</td>
<td>-0.115</td>
</tr>
<tr>
<td>S11 WMA-A</td>
<td>0.0004</td>
<td>-0.115</td>
</tr>
<tr>
<td>MnRoad C34</td>
<td>0.0007</td>
<td>-0.118</td>
</tr>
<tr>
<td>MnRoad C33</td>
<td>0.0003</td>
<td>-0.109</td>
</tr>
<tr>
<td>WesTrac C2</td>
<td>0.0005</td>
<td>-0.135</td>
</tr>
<tr>
<td>WesTrac C5</td>
<td>0.0001</td>
<td>-0.148</td>
</tr>
<tr>
<td>WesTrac C6</td>
<td>0.0016</td>
<td>-0.155</td>
</tr>
<tr>
<td>WesTrac C7</td>
<td>0.0008</td>
<td>-0.136</td>
</tr>
<tr>
<td>WesTrac C23</td>
<td>0.0017</td>
<td>-0.156</td>
</tr>
<tr>
<td>WesTrac C24</td>
<td>0.0008</td>
<td>-0.135</td>
</tr>
</tbody>
</table>

5.4.3 Strain Calibration Validation

To better understand mechanistic design principles Phase II of NCAT test track included the installation of strain gauges at the underside of the asphalt layer, enabling direct measurement of the pavement response in terms of strain. Based on the findings of Phase II the use of strain gauges was extended into Phase III and IV. However, both Phase III and IV used a different definition for the recording of strain, producing strain values typically larger than the Phase II definition. The inclusion of two definitions in the calibration would add a degree of complexity to the analysis and could force the solution in the wrong direction. Furthermore the AAPA APS-fL project includes a sub-task for calibration of strain against performance, so any difference in definition can be handled in the performance calibration phase. It was therefore decided that only the standard definition used in Phase II of Test track would be used in the calibration.

For each test section twelve strain gauges were installed; therefore, one truck pass produced at most, six readings in both the longitudinal and transverse orientation. The maximum reading of each orientation (transverse and longitudinal) was considered the 'best hit' of a tire over a gauge strain, and was therefore, the value for that tyre pass. The strain gauges were installed at three
lateral orientations to help ensure that one of the three offsets would very closely register a direct hit of the tire over the gauge, thus producing if not the maximum, close to the maximum strain value.

For the initial calibration of strain determined from linear elastic analysis against measured strain, individual raw data, in terms of temperature at strain recordings was not available. However, Timm et al. (2006) determined the effect of temperature on strain in the Phase II NCAT structural test cells for both the triple trailer and a box trailer. As the box trailers have a fairly uniform load per tyre of 21,000lbs and lower potential for dynamic effects the box trailers only would be used in this analysis. Timm et al. found the effect of temperature on strain for a given site followed a power relationship as shown in Equation 24 following, with the fitting relationships for each cell shown in Table 11 following.

\[ b_1 T^{-b_2} \]

Equation 24

Where;

- \( T \) is temperature in \(^\circ F\)
- \( b \) and \( b_T \) are fitting parameters

<table>
<thead>
<tr>
<th>Section</th>
<th>( b_5 )</th>
<th>( b_T )</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>N2</td>
<td>3.922x10^{-5}</td>
<td>3.579</td>
<td>0.871</td>
</tr>
<tr>
<td>N3</td>
<td>5.501x10^{-3}</td>
<td>2.332</td>
<td>0.773</td>
</tr>
<tr>
<td>N4</td>
<td>1.304x10^{-3}</td>
<td>2.632</td>
<td>0.733</td>
</tr>
<tr>
<td>N6</td>
<td>1.852x10^{-2}</td>
<td>2.155</td>
<td>0.881</td>
</tr>
<tr>
<td>N7</td>
<td>8.310x10^{-4}</td>
<td>2.796</td>
<td>0.821</td>
</tr>
</tbody>
</table>

It should be noted that there was not enough collected strain data to develop a relationship for the box trailer in section N1 and section N8 was found to have a slippage between the high binder layer base and the layer above and was excluded from the calibration.

In order to calculate strain values using a linear elastic code the support to the subgrade needs to be known. To undertake this analysis the average support values for the combined base/fill and effective subgrade as published by Timm et al. (2006) was used in the calculations, with the values shown in Table 12 following.
Table 12 Subgrade Fill Modulus

<table>
<thead>
<tr>
<th>Section</th>
<th>Base/Fill Layers</th>
<th>Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thickness(mm)</td>
<td>Modulus(MPa)</td>
</tr>
<tr>
<td>N2</td>
<td>635</td>
<td>69</td>
</tr>
<tr>
<td>N3</td>
<td>533</td>
<td>90</td>
</tr>
<tr>
<td>N4</td>
<td>533</td>
<td>83</td>
</tr>
<tr>
<td>N5</td>
<td>584</td>
<td>48</td>
</tr>
<tr>
<td>N6</td>
<td>584</td>
<td>76</td>
</tr>
<tr>
<td>N7</td>
<td>584</td>
<td>79</td>
</tr>
</tbody>
</table>

N.B. the reasons for the differences in published modulus values between fill and subgrade can be found in Timm et al. (2006).

5.5 Conversion between FWD and Dynamic Modulus

To initially determine which of the two approaches debated in the literature most closely matches field performance, the results obtained from test section S9 from the Phase IV study of the NCAT test track was examined. This section was selected due to the low scatter of the results from the back calculation, limiting the chance of drawing the wrong conclusion due to variability in the results.

Because of the current debate on the accuracies of both approaches, it was decided for initial analysis to compare the findings from both approaches and from this initial assessment determine the approach which most accurately predicts field performance to use as the 'seed' value for the full calibration exercise.

To accomplish this, the frequency determined by both the angular frequency, \( t = \frac{1}{2\pi f} \), and pulse frequency, \( t = \frac{1}{f} \) approach was used to determine the modulus in the equivalent master curves at the mid-layer temperature. The time used for the FWD load pulse was a haversine load pulse with duration of 30ms for both comparisons between dynamic modulus and FWD modulus.

Therefore to establish dynamic modulus at 0.030 seconds from the master curves the two comparison frequencies were utilised:

\[ f = \frac{1}{0.03} \text{ or } 33\text{Hz} \]
\[ \omega = \frac{1}{\sqrt{33}} \text{ rad s}^{-1} \text{ or } 5.3\text{Hz} \]

Utilising the equivalent master curves for NCAT test section 9 and the two frequencies currently being debated by researchers, the measured modulus was compared against the predicted modulus from the equivalent master curves. The results of the comparison are shown in Figure 33, for both the angular frequency approach and the cyclic pulse frequency approach.
Figure 33 S9 Control Mix Frequency Conversion

The results of the comparison of the Phase IV control section S9, suggest that the angular frequency approach appears to be significantly more accurate. The results show that the cyclic frequency approach is out of phase with the test results and predicts higher modulus than determined under the pulse load of the FWD. The results show, a phase correction of 2p applied to the dynamic modulus results to convert from angular frequency to cyclic frequency closely matches the measured data. When this correction is applied the dynamic modulus test can be accurately used to predict the response of an asphalt layer under FWD pulse loading.

While highly correlated, the results do show that dynamic modulus test has determined slightly higher modulus than measured in the field. This is most likely due to temperature variations within the asphalt layers, as in the day the average temperature of the asphalt layer will be marginally higher than temperature recorded at mid-depth in the pavement.

5.5.1 Combined NCAT Sections

Based on the initial findings obtained from the control section S9 at the NCAT test track, the study was extended to a number of sites used at both the 2003 and 2009 test track, where dynamic modulus test results were available and field testing of modulus testing against temperature was undertaken.

In order to undertake the first part of the proposed multistep optimisation, the optimisation was set up to determine the optimal conversion factor, k, between the dynamic modulus test and field measured modulus, and the effect, if any, of the temperature gradient within the pavement structure on the conversion.
\[ T_{ave} = \text{at}_{\text{mid}} + b \]  

Equation 25

Equation 26

Where \( f_{fm} \) is the frequency in the dynamic modulus test, \( a, b \) and \( k \) are optimisation constant. The seed values used in the analysis were taken from the results of the analysis of section S9, being 1, 1, and 2\( \pi \) for \( a, b \) and \( k \) respectively.

The seed and trail calibration coefficients were then used with the effective dynamic modulus master curves to predict the laboratory modulus at the equivalent reduced frequency applicable and effective temperature. The optimisation was then run fully unconstrained with \( k, a \) and \( b \) being free to minimise the sum of the difference between predicted and measured modulus.

The results of the optimisation can be found in Appendix D of this report and found that the frequency time conversion constant, \( k \), for all practical purposes was equal to the value recommended by Dongre (2006) of 2\( \pi \). Given the recommendation by Dongre and the results of the optimisation it was decided that all future calculations, the frequency-time conversion factor, \( k \), should be assumed to be 2\( \pi \).

It was found through the optimisation that the temperature equivalency multiplication factor, \( a \), approached that of 1 and the addition factor, \( b \), approached 2. For simplicity, the value of \( a \) and \( b \) was set at 1 and 2 respectively. The following figures shows the results of the comparison with the constants of 2\( \pi \), 1 and 2 applied with Figure 34 showing the FWD modulus results against that of the Dynamic modulus results, and with Figure 35 showing the same results only this time plotted against average layer temperature.
From both figures it is clear that there is a very strong correlation between dynamic modulus and modulus determined from FWD testing. The nonlinear optimisation of frequency and average temperature support the recommendation of Dongre (2006) in comparing dynamic modulus results against resilient modulus, that dynamic modulus frequency should be considered as an angular
frequency and a frequency conversion is required to convert from radians/sec to time. The optimisation found to convert between laboratory dynamic modulus and modulus measured in the field under a 0.03 seconds FWD load pulse a frequency of 5.3Hz should be used in the dynamic modulus test.

The multi-variable optimisation also found that if mid-depth temperature is used to determine the pavement response, a correction of +2°C degrees should be used for daytime analysis to compensate for the average temperature results being slightly higher than the mid-layer temperature.

5.5.2 Validation against MnRoads and WesTrack
To validate the findings obtained from the NCAT test sites and determine whether the results were transferable to other mixes and climate, the dynamic modulus results and field modulus reported for MnRoads and WesTrack test tracks were analysed using the same calibration factors.

Figure 36 following shows the results of the comparison undertaken on MnRoads and Figure 37 for WesTrack test tracks. For MnRoads field modulus data was available across a range of temperatures for Cell 33 and 34. For WesTrack, Figure 37, only results were published for Phase 1 testing at a single temperature for a number of sites.
While the scatter of results is higher than found at NCAT, from both Figures 36 and 37, it is clear that at these two additional locations with a number of different mixes, there is again a very strong correlation shown between dynamic modulus and modulus determined from FWD testing and no bias is observed in the comparison.

Additionally, it is worth noting that at the higher temperatures on cell 34 the predicted modulus from the unconfined modulus test began to deviate from that of the measured modulus, with the results beginning to asymptote around a value of 1000MPa. This is consistent with the findings of the effect of stress susceptibility (as discussed in Section 5.10), that at higher temperature and low frequencies modulus is stress susceptible. The results show that the effect of stress susceptibility in the pavement has a positive effect on modulus and is best represented by applying confinement to the sample in the laboratory. While more work will need to be undertaken to quantify this effect the results tend to indicate that a confinement of approximately 200kPa is required to model asphalt pavements where unconfined modulus falls below a value of approximately 1000MPa. This is not surprising considering the confining stress in an asphalt pavement for 150 to 200mm asphalt layers averages between 200 and 70kPa. The results also indicate that laboratory tests which apply tensile strains to the specimen may be ineffective in characterising mixes at higher temperatures and lower rates of loading.

Based on these findings, it is clear that Dynamic modulus in the laboratory can be directly related to modulus determined under a pulse loading in the field, by equating time and frequency as follows:
Where:

- \( f_{dm} \) is the frequency in the Dynamic modulus test (Hz)
- \( t_p \) is the time of the pulse loading (sec)

This equation should be used to determine the equivalent dynamic modulus frequency to any pulse loading in the field.

The results have shown that dynamic modulus is highly correlated to modulus measured by the FWD when a frequency conversion is applied to the dynamic modulus results, based upon studying the results of the NCAT test track, MnRoads and WesTrack. It was found that for a median depth temperature a temperature correction was required for day time testing to compensate for the temperature gradient within the pavement. Therefore if calculations of modulus are going to be undertaken at times of day other than typically mid-day, more work will be required on modelling the full temperature with depth profile in the pavement structure to determine the effective temperature of the asphalt layers.

5.6 Calibration of Strain
5.6.1 Validation of Stress Based Approaches

Using the developed relationship between dynamic modulus and pulse loading established from the NCAT testing and validated against the results of both MnRoads and WesTrack, the validity of the two existing frequency with depth models used in literature, the Brown model and the CalMe model was assessed.

Both these models calculate an equivalent loading time with depth. This loading time was then used with the relationship found between field loading time and frequency in the dynamic modulus test to validate the accuracy of both approaches. For this analysis, the calculation the pulse width with depth as proposed by Brown (1977) was for the whole pavement layer (i.e. effectively the mid depth of the layer) while for the CalMe (2008) the depth was taken as both the mid layer depth and the depth at the bottom of the asphalt layer.

To undertake this analysis and to calculate strain, the dynamic modulus was used with the subgrade and fill support layers modulus values for each of the selected Phase II test cells, as shown by Table 10 and taken from Timm et al. (2003), to calculate the strain at the underside of the asphalt layer using Linear Elastic Analysis. These strains were then compared against the typical strain, from Timm (2005), for mid-depth temperatures ranging from 5°C to 45°C for five Phase II test sites. The test sites used in the analysis consisted of those sites from the Phase II study where asphalt thicknesses where greater than 150mm (Constant stress), as the LLAP pavement solution method will only be concerned with the thicker asphalt pavement sections.
The results of the analysis can be found in Appendix D of the report and are summarised in Figure 38 following for case (a) using the Brown model and case (b) the CalMe approach, this case with load time taken at the bottom of the asphalt layer.

Figure 38: Validation of Pulse with Depth Approaches Brown
As can be seen both of the proposed pulse time approaches, (using of a stress pulse), result in an under prediction of strain values. This is due to an underestimate of the time of loading which in turn results in an overestimate of modulus under the moving vehicle. While initially there appears to be a lower bias in the CalMe model than the Brown model, this is solely due to the depth in the CalMe model being taken at the bottom of the asphalt layer in the figure, while the Brown model uses the average time within the asphalt layer.

The results also show for both models that the two thicknesses are grouped together, with the 9” pavements being closer to the line of equality than the 7” layers. As in the models the time of load increases with decreasing depth, the results imply that the thickness of the asphalt layer may not be as important in determining the response of the pavement under a moving load as both the Brown and CalMe model imply. Based on this finding it was concluded that a pure calibration of either the CalMe model or the Brown Model was not warranted, as there appears to be an incorrect assumption in how both models determine the effect of depth on loading time.

5.6.2 Numerically Optimised Approach

Given both the Brown and CalMe models did not appear to effectively model both the time of load and the effect of frequency with depth, a fully unconstrained optimisation approach was run to determine whether better agreement could be achieved between the measured and predicted strain. In the unconstrained optimisation the slope of the load pulse, the dynamic/static ratio and the surface contact length were all allowed to be unconstrained to determine the optimal solution.
That is, the time of loading was allowed to be optimised as a function of depth, by the following relationship:

\[
\frac{t_{l}}{t_{p}} = a + b \cdot h
\]

Equation 28

Where \(a\) is the calibration coefficient for the effective length of the load pulse and \(b\) is the effect of depth on the load pulse, \(t_{p}\) and \(v\) are as before the time of loading and the velocity of the design vehicle.

The results of the fully unconstrained optimisation can be seen in Figure 40 following for the equivalent layer approach and Figure 41 for the multi-layer pavement.

Figure 40 Unconstrained Optimisation Equivalent Layer
The results of the equivalent layer analysis show that an extremely high correlation was achieved with the unconstrained model, with no bias observed between the measured and predicted values.

Surprisingly, the optimisation found that thickness of the asphalt layer played little to no part in determining the effective frequency, with the slope value (b) approaching 0. The results also found that the wave length of the pulse on the surface of the pavement was significantly larger than would be predicted by any stress pulse model, at 1.79m. Vehicle dynamic effects were determined to have little to no effect on measured response, with the optimisation determining a constant of 1.

At first, these results were surprising as they are contrary to current design procedures. However, if the results are compared against the findings and recommendation of Coffman back in 1967, remarkably similar results have been obtained. Remember that Coffman found that a vehicle acts as a cyclic load with a wave length of 6 feet and that using higher frequencies in the upper most layer and lower frequencies for the lower layers does not appear justified. His findings are identical to the findings of the optimisation of 1.79m and depth has no effect on determining the effective frequency.

The primary difference in the approach of researchers such as Brown against that of Coffman was that Coffman used deflection (strain) to determine the load pulse, while the approach of Brown and others used stress. The results show that when using loading time in a pavement to evaluate the
results against laboratory determined modulus, the time of loading should be considered as the
time of loading of the strain pulse, not the commonly used stress pulse.

From this it is recommended that a cyclic load of 1.8m should be used to determine the loading
time in a full or partial depth asphalt pavement (>125mm) and a constant frequency be used for all
thicknesses.

\[
\text{Equation 29}
\]

Where;

\[v_m\] is the vehicle speed in ms\(^{-1}\)

The result of this analysis was then extended to that of a multi-layer asphalt pavement, Figure 41.
The results confirm the single layer findings that time of loading within an asphalt pavement should
be a constant for a given vehicle speed. The results show that the use of a multilayer asphalt
pavement results in a small change in the goodness of fit of the model and little change in the bias
of the results by using a constant loading time in a multilayer asphalt pavement. While the results
show that multilayer asphalt pavements can be accurately modeled in a linear elastic code to
calculate strain under a moving load, the model is sensitive to the chosen layer thicknesses and
more work is required on determining both the appropriate sub-layering of multilayer asphalts and
the effect of temperature profiles within the sub layering.

Based on this analysis it is recommended that a constant frequency be used for all layers of asphalt
in a multi-layer asphalt pavement and until the question of sub layering and temperature with depth
profile is answered, the effective modulus of the combined asphalt layers be used for the purpose
of design.

5.7 Use of Australian Dynamic Modulus Master Curves to Predict Strain

5.7.1 Calculation of Modulus

It has been identified that for Australian mixes at higher temperatures and low rates of loading there
can be a difference between the modulus of asphalt in tension and compression due to the stress
susceptibility of asphalt. For normal operating conditions this difference is negligible. However, at
higher operating temperature stress susceptibility begins to have an influence on modulus. In
typical, full or partial depth asphalt pavement the effect of confinement under loading is
compressive. This phenomenon is shown in the results of FWD testing at NCAT, Westrack and
MnRoads where the FWD stiffness has a limiting value of approximately 1000MPa. Examination of
the Australian mixes shows that the same limiting value appears when the asphalt is tested at
confining pressures of approximately 200kPa.

Therefore it is not recommended for modelling at higher temperatures (>30°C), that modulus values
determined from tensile, flexural or test without confinement tests be used for the purpose of
pavement design. It is recommended that modulus be determined based on the results of compressive tests with approximately 200kPa confinement.

The modulus values presented in Chapter 4 were all developed using the results of unconfined testing, as demonstrated these results will not match reality at higher temperatures. Therefore the confidence based master curves of Chapter 4 were recalculated using 200kPa confinement pressure. It is recommended that these values be used for the purpose of design.

Table 13 Master Curve Fitting Parameter

<table>
<thead>
<tr>
<th>Nominal max. Size (mm)</th>
<th>Binder Type</th>
<th>Master Curve Sigmoidal Fitting Parameters</th>
<th>Confidence Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>b</td>
<td>g</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>C320</td>
<td>1.878</td>
<td>0.043</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>1.860</td>
<td>-0.023</td>
</tr>
<tr>
<td></td>
<td>A15E</td>
<td>1.820</td>
<td>0.104</td>
</tr>
<tr>
<td></td>
<td>Multi</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>20</td>
<td>C320</td>
<td>1.715</td>
<td>0.157</td>
</tr>
<tr>
<td></td>
<td>AR450</td>
<td>2.328</td>
<td>-0.454</td>
</tr>
<tr>
<td></td>
<td>C600</td>
<td>2.363</td>
<td>-0.465</td>
</tr>
<tr>
<td></td>
<td>Multi</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Combining the angular frequency conversion with the stress pulse relationship obtained from the NCAT test results gives the following relationship (Equation 30) between vehicle speed (in km/hr.) and dynamic modulus test frequency.

\[
\frac{v}{f_{dm}} = \frac{1}{a} 
\]

Equation 30

Where;

\[ v = \text{vehicle speed in km/hr}^{-1} \]
\[ f_{dm} = \text{frequency in dynamic modulus test} \]

5.7.2 Speed Incorporated Design Modulus Charts

The effect of the strain pulse and frequency conversion can now be easily incorporated into a series of curves for each mix group.

Figure 42 to Figure 47 following show the Nomographs for the 95th percentile confidence interval for the six subgroups of Australian mixes.
It needs to be noted that these charts represent an equivalent stiffness to obtain strain under a moving vehicle. As such, the modulus is a converted modulus which more closely represents the relaxation modulus of the mix. It would therefore be technically incorrect to refer to these charts as dynamic modulus charts.
Figure 44 Design Modulus Chart DG14 AR450

Figure 45 Design Modulus Chart DG20 C320
Figure 46 Design Modulus Chart DG20 AR450

Figure 47 Design Modulus Chart DG20 C600
5.8 Laboratory to Field Conversions Conclusions and Recommendations

The analysis of dynamic modulus test results from NCAT, MnRoads and WesTrack, against field modulus determined from FWD testing, found that frequency in the dynamic modulus test should be considered as an angular frequency and that a shift of $1/2\pi$ on the frequency axis will allow the use of dynamic modulus values to determine the modulus resulting from a pulse loading in the field. Using this conversion it was found that dynamic modulus results at 5.3Hz ($1/(2\pi \times 0.03)$) could be used to accurately predict the modulus determined from FWD loading with a pulse width of approximately 0.03 seconds over a wide range of temperatures. The results of the optimisation found that the use of the mid-layer depth resulted in a slight underestimation of the effective asphalt layer modulus for day time testing and that if mid-layer depth is used a correction of +2°C is required to correct for the average temperature within the asphalt layers. Therefore, if modulus calculations are going to be undertaken at times of day other than typically mid-day, more work on modelling the full temperature with depth profile in the pavement structure would be needed to determine the effective temperature of the asphalt layers.

Using the pulse frequency conversion and temperature correction obtained from comparison with FWD testing, the multi-variable optimisation found that dynamic modulus could be used to accurately predict strain under a moving load using layer elastic analysis when time of load is corrected for the effective load length. It was found that when computing strain under a moving load, contrary to some published recommendations, the thickness of the asphalt layer was insignificant in determining strains. It was also found that the time of loading is more related to the length of the deflection response than the current approach of the use of a stress pulse.

The results of the optimisation on the thick asphalt sections of Phase II NCAT support the recommendation of Coffman that a vehicle acts as a cyclic load with a wave length of six feet, with the optimisation determining the wave length of 1.79m. Based on these findings the following frequencies in the dynamic modulus test are recommended for use in pavement design with an equivalent combined asphalt layer.

<table>
<thead>
<tr>
<th>Speed (km/hr.)</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recommended Frequency Dynamic Modulus $</td>
<td>E^*</td>
<td>_\text{Test (Hz)}</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>2.0</td>
<td>2.2</td>
</tr>
</tbody>
</table>

The analysis showed that multi layers were sensitive to the chosen sub layer thicknesses and more work would be required on determining both the appropriate sub layering of multilayer asphalts and the effect of temperature profiles in the sub layering, before a multilayer approach should be recommended over the use of the equivalent asphalt layer approach.
The analysis has shown that there is a direct link between laboratory modulus and strain under a moving vehicle and dynamic modulus can be used in the structural design of LLAP's. The use of the master curves will enable the determination of either threshold strains or cumulative distribution of asphalt strain in LLAP structures as a function of the climatic conditions and the traffic distribution spectrum. This calculated strain will provide the means to rationally evaluate the compliance of candidate LLAP structures with the limiting threshold strain or cumulative strain distribution empirically derived from the evaluation of international LLAP's.
6 Inter-Conversion of Current Australian Modulus Test Methods

6.1 Background

A fundamental parameter required for the characterisation of an asphalt mix in any mechanistic design procedure is the modulus or stiffness of the material. Traditionally, in Australia two methods have existed for the measurement of the modulus of an asphalt mix, the resilient modulus test and the 4 point flexural modulus test. Additionally, the Austroads Pavement Design Guide is based on the 2 point bending flexural modulus testing and as part of the Asphalt Pavement Solutions for Life (APS-FL) project, the Australian Asphalt Pavement Association (AAPA) introduced the dynamic modulus test and the use of master curves for the characterization of asphalt mixes.

While all these tests measure modulus (or stiffness) of the mix, none have the same definition of frequency (or time) or use the same loading shape. The different load shapes and definitions of frequency result in different modulus values being obtained for the same frequency and temperature for each test method. The different results obtained by the different test methods has led to confusion on how to report and convert modulus results obtained from one test method to another or convert to a field modulus.

The mathematics behind the conversion between the different modulus tests is complex and open to interpretation. Therefore instead of trying to solve the complex mathematics, this chapter looked at the direct inter-conversion of the modulus tests and recommends a standard approach to converting and reporting modulus results.

6.2 Introduction

Asphalt is a viscoelastic material whose response under load depends on the rate of loading and the temperature. Stiffness, being one of those responses is one of the most important properties of asphalt for pavement design. The concept of stiffness for asphalt mixes was introduced by Van der Poel (1954) who distinguished stiffness from the modulus (E) of elasticity. Various properties of asphalt mixes can be used to characterise the stiffness of asphalt mixes, including: creep compliance, relaxation modulus, complex modulus and resilient modulus.

Since various ways exist to measure stiffness and the stiffness measured in the laboratory is often used as an input into pavement design to predict response under load, an accurate way is needed to convert between different laboratory tests to convert to field stiffness and to accurately predict the response in a multi-layer pavement.

Australia uses two common tests for the characterisation of asphalt mixes and Austroads has placed high priority on the retention of these methods. While Australia has two test methods and the AGPT002 (2012) is based on a third, a standard reporting method has not been established. Adding to this the AAPA Asphalt Pavement Solutions for Life project has introduced a fourth test,
the dynamic modulus test. To avoid confusion the inter-conversion between dynamic modulus, resilient and 4 point flexural and 2 point flexural modulus need to be understood.

As part of the APS-FL and as previously shown in Section 5, a method has been developed for the inter-conversion of dynamic modulus to field response under a pulse load. However, as stated previously the dynamic modulus test is not the standard test procedure used in Australia. If a conversion can be achieved between the dynamic modulus test, resilient modulus test, and the 2 and 4 point flexural beam tests direct conversion to field response can be undertaken and the significant benefits offered by the dynamic modulus test can be employed with understanding of the relationships.

6.2.1 Scope of Study

Two principal hypotheses exist as to why there are differences between different modulus test methods; (i) stress susceptibility, where tensile loads result in lower stiffness than compressive loads and (ii) different definitions of frequency, where some tests are undertaken in the frequency domain and some are undertaken in the time domain.

This study will examine the validity if the two hypotheses, with regards to Australian mixes and test methods and determine the conversion, if any, between the common Australian test methods.

The initial portion of this chapter presents a summary of the background and methods for the determination of stiffness or modulus of asphalt mixes. The next portion examines the results of individual resilient and flexural modulus tests against those of the dynamic modulus test. The final section uses the conversions found from the individual tests to validate the single temperature/frequency corrections across the full temperature frequency space using master curves for, flexural modulus (2 point bending and 4 point bending), resilient modulus and dynamic modulus.

6.3 Theoretical Background

6.3.1 Observations

It is commonly observed that dynamic modulus results are higher than both resilient and 4 point bending modulus results, as shown in Figure 48 following for a typical Australian mix.
To explain the differences between resilient, flexural and dynamic modulus two explanations are generally used:

- Dynamic modulus, as a compressive modulus results in higher values than the resilient modulus test as a tensile modulus, due to stress susceptibility of the asphalt mix. That is the bulk stress in the dynamic modulus test is higher.
- Time of loading is not directly comparable as time of loading in the resilient modulus test needs to be shifted to the frequency domain. (I.e. appropriate frequencies need to be selected to convert loading time in the resilient modulus test to angular frequency in the dynamic modulus test.)

6.4 Differences in Test Results

6.4.1 Stress Susceptibility

Many researchers have concluded that the difference between the bending tests, indirect tests and direct compression tests are the results of different modes of loading; flexure, tension and compression respectively, Kim (1992).

Generally asphalt materials are assumed to be linear elastic or liner visco-elastic. While this assumption is true at lower temperature and higher frequency range (Antes et al. (2003)), at higher temperature ranges and low frequencies the assumption may not be valid. This effect was observed by Molenaar (1983) who observed that at temperatures of 35°C and 1 second loading the modulus
in compression was double that of the tensile modulus. Additionally, Pellinen (2001) found that beyond certain strain levels non-linearity (stress dependency) is exhibited by asphalt mixes regardless of test temperature. These two findings would suggest that it would be reasonable to assume that a difference in the modulus determined from tensile and the compressive tests could be due to a difference in the stress (bulk stress) applied to the sample. However, Pellinen also found that at small strain levels (like those used in modulus testing), stress susceptibility was only evident at high temperatures and low frequencies. Additionally, in strength tests the compressive strength of asphalt is significantly higher than the tensile strength.

The assumption that modulus measured in the direct compression tests is higher than that of the tensile tests (flexural fatigue and resilient modulus) due to stress susceptibility, appears to be only justified from the results of large strain tests and strength tests. In large strain test, the modulus of asphalt can be a magnitude higher as a result of changes in the bulk stress applied to the material. The assumption, however, of stress susceptibility being significant at the small strain and mid-range temperatures used in the modulus test appears to be questionable.

6.4.2 Time Conversion

In Physics text books, such as Tipler (1992), the period of loading, T, not t, is simply the period of a harmonic cycle in seconds and may be expressed as:

\[
\text{Equation 31}
\]

\[
T = \frac{1}{f} = \frac{2\pi}{w}
\]

Where;

- T is the period of the cycle
- f is the frequency of the loading, Hz
- and w is the angular frequency, rad.s^{-1}

For each of the loading shapes used in the Australian test methods, the period of loading, T, (the period of the cycle) is related to the pulse time, t, however it is not necessarily the same;

- For cyclic loading the period T is the pulse time, t.
- For the case of a harmonic frequency load the period, T, is in fact angular period and, is \(2\pi t\).

It is hypothesized by some researchers that because of these different periods that comparison of results obtained from one test to another is difficult and in some cases results do not appear to be directly related. More specifically, it is not always evident whether a test sets up a cyclic response or a harmonic response.

There is currently significant debate amongst researchers on how frequency is related to time in the dynamic modulus tests. The two primary schools of thought are the angular frequency approach.
vs. the pulse frequency approach. With researchers such as Dongre et al. (2006) recommend the angular frequency approach, \( t = \frac{1}{\omega} \), while researchers such as Katicha et al. (2007) recommend the pulse frequency, \( t = \frac{1}{f} \) approach.

It appears that the earliest use of the angular frequency approach, \( t = \frac{1}{\omega} \), in asphalt mixes was the work undertaken by Papazianin (1962) at the First International Conference on the Structural Design of Asphalt Pavements. This approach was then adopted by Shell for their development of a ME pavement design procedure, which was subsequently adopted as the basis of the AGPT002 (2012) characterisation method. However, the angular frequency approach has not been universally adopted with the US MEPDG following the \( t = \frac{1}{f} \) approach.

Dongre (2006) found that the exact solution of the IFT to calculate relaxation modulus from the dynamic modulus test was \( t = \frac{1}{\omega} \); this is somewhat contrary to the early recommendations of Van der Poel (1954) who suggested the conversion was only approximate.

Notwithstanding this, Dongre (2006) did not establish that the testing in dynamic modulus test was an angular frequency, which would require the above conversion. The issue is still not resolved amongst some researchers.

What is important in the Australian case is to establish how to compare the modulus results determined from the various test methods used in practice and relate them to a standard measure and field stiffness.

### 6.4.3 Stiffness in the Austroads Pavement Design Guide

The modulus values used in the AGPT002 (2012) have their origin in the set of Nomographs published by Shell (1978). These Nomographs are a combination of two Nomographs developed by Van der Poel (1954), for determining the stiffness of binder and Bonnaure et al. (1977) for determining the stiffness of a bitumen mix.

In developing the binder stiffness nomograph Van der Poel (1954) stated that the stiffness of the binder could be treated as either: the inverse of the creep compliance at a loading time, \( t \), or the dynamic modulus at an angular frequency, \( \omega = \frac{1}{t} \). What is important here is understanding that the conversion is a conversion between dynamic modulus and stiffness modulus (inverse of creep), the conversion is a conversion to time domain and that the conversion is only an approximate conversion (Van der Poel). However, other researchers such as Dongre et al. (2006) have stated that the conversion is an exact conversion.

In developing the mix stiffness nomographs, Bonnaure et al. (1977) used the modulus determined from extensive 2 Point Bending on Trapezoidal Beams (2PB-TR) with a sinusoidal load. To connect the bitumen stiffness to the asphalt mix modulus Bonnaure et al. (1977) used the stiffness modulus
of the bitumen from the Van der Poel Nomograph. The result is that the frequency used in the
nomographs is an angular frequency, $\omega = \frac{1}{\tau}$.

In recent developments of the AGPT002 (2012) there is a general assumption that the 4 Point
Bending test on Prismatic Beams (4PB-PR) measures the same modulus as the 2PB-TR, (JAMESON
(2001)). This assumption was supported by Comparison testing conducted in Europe by Francken
et al. (1994) which showed that 4PB-PR and 2PB-TR modulus values achieve results which are in
good agreement provided that: tests are performed at small strain and the sample geometry and
mass are taken into account in the calculation of modulus, Francken. An Austroads test method to
perform this type of small strain complex modulus measurement in line with European specifications
has recently been introduced (AGPT/T274). Whether the flexural modulus calculated as part of
fatigue experiments in the past using the AGPT/T233 (typically at high strain levels) provides
modulus values equivalent to these small strain experiments is not known.

6.4.4 Current Modulus Classification Methods in use in Australia

For the past two decades in Australia, the predominant method used for determining the modulus
of asphalt materials for both pavement thickness design and for material characterisation has
resilient modulus test (AS 2891.13.1-1995) using Indirect Tensile load on a cylindrical sample (IT-
CY). Under this test method the modulus is defined as the ratio of the applied stress to the
recoverable strain. More recently the 4 Point Bending on a Prismatic beam (4PB-PR) (AG:PT/T233
2006), has gained favour in some specifications (Vic Roads) for characterisation of Australian mixes
and by researchers such as Denneman et al. (2013) who have begun to report modulus master
curves developed from the 4PB-PR test. However Denneman (2013) has used sinusoidal loading
over the common haversine load used in Australia. In addition the AAPA APS-FL project introduced
the Direct Compression test on cylindrical samples (DC-CY) (AASHTO TP79) for the
determination of modulus and the recent introduction of EME 2 mixes has brought the 2 Point
Bending TRapezoidal 2PB-TR (EN12697-24) test to Australia.

To successfully implement different classification methods in practice, the differences in
characterisation of modulus obtained from the IT-CY, 4PB-PR, 2PB-TR and DC-CY modulus tests
need to be fully understood. Without a thorough understanding of the differences between the
modulus tests the implementation of four systems modulus classification into the one material
classification and design system of would be confusing for designers and specifies and could
diminish the technical credibility of the specifications, design system and testing standards.

6.4.5 Previous Research

Internationally, research such as the studies by Dongre et al. (2006), Kim et al. (1995) and Adhikari
et al. (2008) have reported differences between the results of resilient and dynamic modulus of
between 30 and 100%. Dongre (2006) concluded that the difference in the resilient modulus test
and the dynamic modulus test was a result of a different definition of time in both tests. However,
while Kim et al. also found that overall resilient modulus values were generally lower than the
dynamic modulus results, the researchers concluded that the difference between the dynamic
modulus and resilient modulus was a result of the dynamic modulus being conducted in the
compressive mode while the resilient modulus was undertaken in the tensile mode.

In section 5 it was found that the differences between field loading and dynamic modulus testing
(DC-CY) was due to differences in the definition of time and that the dynamic modulus test should
be considered an angular frequency with $t = \frac{1}{2\pi f}$ when compared to the time of loading imparted by
a vehicle's single pulse load.

6.5 Materials and Test methods

6.5.1 Modulus Test Methods

Throughout the world, the characterisation of asphalt mixes has been undertaken using a variety
of different loading forms, such as; sinusoidal, haversine, pulse, square and triangular wave forms
with and without, rest periods, all used in an attempt to simulate the response of a pavement to
vehicular loads. Internationally, the most commonly used wave forms to characterise asphalt
materials are, the sinusoidal and haversine waves (Huang (2004)), while in Australia it is the quasi
haversine step load of the IT-CY resilient modulus test.

For the four test methods being examined in this study the following describes the wave forms used
in the test method.

  condition temperature ($25^\circ\text{C}$) and a single pulse load of 0.04seconds rise time, with a
  controlled load.
- Flexural Modulus (4PB-PB), uses a standard load frequency of 10Hz, with a continuous
  load (AG:PT/T233 (haversine) AG:PT/T234(sinoidal)) and controlled displacement
- Direct Compression on Cylindrical (DC-CY) samples (A5HTO TP62 or AASHTO TP79)
  uses a continuous haversine load with a target strain within the sample
- Flexural 2 Point Bending on Trapezoidal (2PB-TR) samples (EN12697-24) uses a
  continuous sinusoidal load and controlled strain within specimen.

Clearly the loading shapes used in the test methods are different and all cannot be the same as per
the assumption of Bonnaure et al. (1977).

6.5.2 Strain in Beam Fatigue

Another complication occurs in the 4PB-PR (AG:PT/T233) used in Australia, which is a controlled
displacement test with a haversine load. In the controlled deflection test, the deflection input is
haversine, which bends the beam with the same peak-to-peak magnitude, at least intially, as the
sinusoidal test except in one direction only. However, due to the viscoelastic nature of asphalt the
beam creeps under load. This creep results in permanent deformation in the initial loading cycles. As a result of this deformation in the test the neutral axis position of the beam also shifts down. After a few loading cycles, typically less than 50, the neutral axis in the centre of the beam shifts down and as a result the haversine load has changed to a sinusoidal load, with the beam being push and pulled in the test. This is shown conceptually Figure 49 following.

![Figure 49](image)

Figure 49 Flexural Fatigue Test (after NCHRP 9-44(2013))

The figure shows conceptually what happens in the beam fatigue, that due to the shifted position of the beam, the developed stress/strain immediately changes from haversine, tension only, to sinusoidal, alternating tension and compression. Due to the change from tension only to alternating tension and compression the magnitude of the strain developed is only half that of the initial strain applied, at the beginning of the test, as shown in Figure 49. At the end of the test, when the load is removed, the beam remains in the bent position showing permanent deformation.

6.6 Comparison of Modulus Tests

6.6.1 Stress Dependency

If the difference between dynamic modulus and resilient modulus is explained by the difference in the mode of loading (tensile, flexure or compressive), then at the test temperature and the strain levels applied in the modulus test the mix response should be nonlinear and the asphalt mix should exhibit stress susceptibility behaviour.

So the question is 'Does stress dependency exists in asphalt mixes under the loading conditions used in the resilient modulus test used in Australia?' To answer this, the results of the dynamic modulus test performed at different confining pressures (different bulk stresses) were examined to see when the typical Australian asphalt mixes exhibit stress dependency. Figure 50 following; shows the dynamic modulus plot of a typical Australian mix tested at 4 confining levels (0, 50, 100 and 200kPa) in this case an AC 14 AR 450 mix.
The examination of the results show that at the standard Australian reference temperature of 25°C the AR450 mix only begin to exhibit stress susceptibility when the reduced frequency of loading falls below approximately 0.1Hz, or a haversine pulse width of 10 seconds in the dynamic modulus test. By comparison, the conditions used in the resilient modulus in Australia (25°C and rise time of 40msec), which equate to a frequencies of 12Hz, (1/τ) or 2Hz (1/2τ) respectively, using the two frequency assumptions which are currently being debated. It can be seen in Figure 50 that at these set frequencies (and the 25°C test temperature), there is no difference in the predicted modulus. This result indicates that under the standard Australian test condition used in the resilient modulus test, stress susceptibility should have little to no effect on the modulus of typical Australian mixes.

The implication of this is that under the standard test conditions Australian asphalts are not stress susceptible and the mode of loading (tension, compression or bending) will have little to no effect on the measured modulus. That is, the mode of loading cannot explain the differences obtained between the test methods.

This finding is supported by those of Kallas (1970) and Khanal et al. (1995) both of whom found little differences, if any, between the modulus of asphalt under harmonic compressive and tensile loading except at low frequencies and high temperatures. This supports the view of Dehlen (1968) who concluded that non-linearity was not significant for practical design of pavements.
6.6.2 Resilient to Dynamic Modulus

In order to compare between the dynamic and resilient modulus a cross referencing of Fulton Hogan’s database of resilient and dynamic modulus testing undertaken since 2006, was carried out and resulted in 46 mixes with corresponding resilient (IT-CY) and dynamic modulus tests (DC-CY). This database has an extensive range of aggregate types from throughout Australia and covers the full range of conventional and polymer modified binders used in Australia.

To cover the two proposed loading definitions debated by researchers of:

\[
\begin{align*}
\varepsilon & = 1 - \varepsilon_1 - \varepsilon_2 \\
\varepsilon_1 & = \frac{1}{2} \\
\varepsilon_2 & = \frac{1}{2} 
\end{align*}
\]  

Equation 32

Two frequencies in the DC-CY dynamic modulus were considered, 12.5 and 2Hz for the 0.08sec loading time of the Australian IT-CY test.

Table 15 following, shows the mix descriptions, measured resilient modulus and the dynamic modulus (DC-CY) at 2 and 12.5Hz of the 46 mixes in the Fulton Hogan database where a comparison could be undertaken. Figure 51 following, shows a plot of the dynamic modulus results at 12.5Hz and 2Hz versus the average resilient modulus for each asphalt mix.

<table>
<thead>
<tr>
<th>Mix Description</th>
<th>Resilient Modulus (MPa)</th>
<th>Dynamic Modulus E* MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2Hz</td>
</tr>
<tr>
<td>AC10 5.7% Mex AP</td>
<td>3674</td>
<td>3653</td>
</tr>
<tr>
<td>AC10, 5.7% C450</td>
<td>5174</td>
<td>4586</td>
</tr>
<tr>
<td>AC10, 5.7% Mex AP + Sasobit (New)</td>
<td>3705</td>
<td>3558</td>
</tr>
<tr>
<td>AC10, 5.7% Mex AP + Sasobit</td>
<td>3812</td>
<td>3675</td>
</tr>
<tr>
<td>AC10, 5.7% C320</td>
<td>4673</td>
<td>4579</td>
</tr>
<tr>
<td>AC10, 5.7% A10E</td>
<td>2016</td>
<td>1684</td>
</tr>
<tr>
<td>AC10, 5.7% A15E</td>
<td>1988</td>
<td>2595</td>
</tr>
<tr>
<td>AC10, 5.7% Multi 1000</td>
<td>5024</td>
<td>4982</td>
</tr>
<tr>
<td>AC10, 5.7% Multi 600</td>
<td>4625</td>
<td>3856</td>
</tr>
<tr>
<td>AC10, 5.7% A20E</td>
<td>2521</td>
<td>2104</td>
</tr>
<tr>
<td>AC10, 5.7% A35P (EVA)</td>
<td>4406</td>
<td>4877</td>
</tr>
<tr>
<td>AC10, 5.7% Mexphalte Fuelsafe</td>
<td>3217</td>
<td>4336</td>
</tr>
<tr>
<td>AC10, 5.7% Mexphalte Airport</td>
<td>3783</td>
<td>3653</td>
</tr>
<tr>
<td>AC10 LCC Qld.</td>
<td>5739</td>
<td>4978</td>
</tr>
<tr>
<td>AC14 R116 C320 Rutherford</td>
<td>5803</td>
<td>4863</td>
</tr>
<tr>
<td>AC14 Wallgrove C320</td>
<td>6864</td>
<td>7193</td>
</tr>
<tr>
<td>Material Description</td>
<td>Weight</td>
<td>Length</td>
</tr>
<tr>
<td>----------------------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td>AC20 + AS 2150 4.2% C320 (30% Coarse RAP)</td>
<td>8337</td>
<td>7344</td>
</tr>
<tr>
<td>DG14 4.72% A15E ex-Crestmead</td>
<td>6192</td>
<td>5308</td>
</tr>
<tr>
<td>14H 4.9% C600, Vic</td>
<td>8055</td>
<td>7900</td>
</tr>
<tr>
<td>DG20 4.65% C320 25% RAP Qld</td>
<td>7178</td>
<td>8372</td>
</tr>
<tr>
<td>SMA14 + FGLS &amp; HL 6.5% C450</td>
<td>4675</td>
<td>4665</td>
</tr>
<tr>
<td>AC14 + Hydrated Lime 5.2% C450</td>
<td>5801</td>
<td>6655</td>
</tr>
<tr>
<td>AC14 + Hydrated Lime 5.2% C450</td>
<td>6780</td>
<td>6655</td>
</tr>
<tr>
<td>SMA10, S50R</td>
<td>2828</td>
<td>2931</td>
</tr>
<tr>
<td>SMA10, C320</td>
<td>4117</td>
<td>4304</td>
</tr>
<tr>
<td>SMA10, C320</td>
<td>4158</td>
<td>4304</td>
</tr>
<tr>
<td>14TCI Base S2, Vic</td>
<td>3412</td>
<td>2919</td>
</tr>
<tr>
<td>14TCI Base S1, Vic</td>
<td>4071</td>
<td>4511</td>
</tr>
<tr>
<td>14TCI Intermediate S1, Vic</td>
<td>6401</td>
<td>5848</td>
</tr>
<tr>
<td>14TCI Intermediate S2, Vic</td>
<td>7052</td>
<td>5973</td>
</tr>
<tr>
<td>AC10, 5.7% Mexphalte C</td>
<td>997</td>
<td>694</td>
</tr>
<tr>
<td>AC20 WA Base 5.2% 320 with Hydrated Lime &quot;Conforming Mix&quot;</td>
<td>2824</td>
<td>3600</td>
</tr>
<tr>
<td>AC20 Base 5.3% BP C320 + Redicote N422 M2B3 &quot;Job Mix&quot;</td>
<td>2936</td>
<td>3163</td>
</tr>
<tr>
<td>AC10 5.7% C450</td>
<td>4537</td>
<td>5164</td>
</tr>
<tr>
<td>AC14 &quot;C&quot; 6.55% C320</td>
<td>3982</td>
<td>4311</td>
</tr>
<tr>
<td>A14 &quot;MD&quot; 6.28% C320</td>
<td>5329</td>
<td>4847</td>
</tr>
<tr>
<td>AC14 &quot;VC&quot; 6.62% C320</td>
<td>3996</td>
<td>4510</td>
</tr>
<tr>
<td>AC &quot;MD&quot; 6.28% C320</td>
<td>5246</td>
<td>4847</td>
</tr>
<tr>
<td>AC14 &quot;VF&quot; 7.4% C320</td>
<td>4446</td>
<td>4847</td>
</tr>
<tr>
<td>AC14 &quot;F&quot; 5.9% C320</td>
<td>6879</td>
<td>6997</td>
</tr>
<tr>
<td>AC14 5.4% C600</td>
<td>4426</td>
<td>5289</td>
</tr>
<tr>
<td>AC14 5.4% AR 450</td>
<td>4073</td>
<td>5060</td>
</tr>
<tr>
<td>AC14 5.4% C320</td>
<td>4118</td>
<td>4491</td>
</tr>
<tr>
<td>AC14 Multi100</td>
<td>5386</td>
<td>5460</td>
</tr>
<tr>
<td>AC14 5.4% C1000</td>
<td>7583</td>
<td>6422</td>
</tr>
</tbody>
</table>
Figure 51 Resilient vs. Dynamic Modulus

The results and figure show that a clear and strong correlation exists between the dynamic modulus and the resilient modulus and that the correlation is not mix dependent. The results show that at the cyclic frequency of 12.5Hz the dynamic modulus results, while highly correlated to the resilient modulus are offset by a factor of approximately 2,500MPa, i.e. the results of the dynamic modulus test will be approximately 2,500MPa higher than that of the resilient modulus test. At the angular frequency of 2Hz, the results are again highly correlated but this time centred on the line of equality. The centring on the line of equality indicate that equivalent results are obtained between the dynamic modulus test and the Australian resilient modulus test at a frequency of 2Hz, the angular frequency in the dynamic modulus test.

The results of the comparison of the dynamic modulus frequency against that of the pulse load in the resilient modulus IT-CY test confirm the results found in the comparison to field study that the dynamic frequency should be considered a cyclic frequency. Given that the conversion between IT-CY test and the DC-CY tests is equivalent to that found from the DC-CY test to field study in Chapter 5, it can be concluded that time is equivalent and that:

$$f_{dynamic} = f_{field}$$  

Equation 33

Where:

$$T_{hp} = \text{load duration sections}$$
\( R \) is the rise time (typically 0.04sec)

and

\[
\frac{f_{\text{fres}} c}{\nu (\nu - 1)}
\]

Equation 34

where;

\( f_{\text{dm}} \) is the equivalent dynamic modulus frequency

6.6.3 Dynamic Modulus to Flexural Beam Modulus

Historically, it is known that the Austroads beam fatigue test produces results which are less than those of the dynamic modulus results. The standard test conditions in the Austroads AGPT/T233 method use deflection controlled haversine load with a 10Hz load frequency. The test is also typically run at a displacement resulting in a relatively high strain condition of 400με in the beam specimen. This introduces an additional complexity in comparing the results of the test to those of other modulus tests, which are run in small strain configuration. For this reason, the new Austroads AGPT/T274 method modulus testing is performed at 50με (sinusoidal).

As was established in Section 6.6.1 the reason for the difference in the modulus results cannot be attributed to stress dependency, under the standard loading conditions. Given the 4PB-PR test constantly measures modulus values which are less than that of the DC-CY test a direct conversion between frequencies was not undertaken (i.e. \( f_{\text{fPR-PR}} = f_{\text{DC-CY}} \)) and only a frequency conversion was undertaken.

As previously described, the test which the results are based, initially applies a haversine load and after a few cycles the test applies a continuous sinusoidal load. Clearly a continuous sinusoidal load is a different wave shape to the quasi haversine pulse found in field loading or as used the Australian resilient modulus test.

As previously shown the dynamic modulus (DC-CY) test should be considered angular frequency test, when compared to single pulse loads such as in the IT-CY or field loading. What is not known is the conversion which should be applied between the 4 point bending test and frequency in the dynamic modulus test. To undertake this analysis, three conversions were assessed, as shown following:

\[
\frac{f_{\text{fPR-PR}} c}{\nu (\nu - 1)}
\]

Equation 35

\[
\frac{f_{\text{fPR-PR}} c}{\nu (\nu - 1)}
\]

Equation 36

\[
\frac{f_{\text{fPR-PR}} c}{\nu (\nu - 1)}
\]

Equation 37

Where;

\( f_{\text{fPR-PR}} \) is the frequency of the 4PB-PR, flexural test, and

\( f_{\text{DC-CY}} \) is the frequency of the DC-CY, dynamic modulus test.
For the 10Hz frequency used in AGPT/T233, the equivalent harmonic frequency in the dynamic modulus test would be 10, 1.6Hz or 3.2Hz for the three options respectively.

Figure 52 following shows the modulus obtained from the DC-CY dynamic modulus test at the angular equivalent frequency of 3.2Hz and the initial modulus measured from the 4PB-PR flexural test for 35 mixes tested in the Fulton Hogan Laboratory, since 2004.

As can be seen in the figure, at a frequency of 3.2Hz results in the DC-CY dynamic modulus test are highly correlated with the 4PB-PR flexural test and centred on the line of equality. The results show that the 4PB-PR, when tested at high strain for fatigue, should not be considered a dynamic modulus test and that the results appear to exist in the time domain, with a time of loading given by:

\[
\frac{\sigma_{1}}{\sigma_{2}} = \frac{1}{2} \frac{\epsilon_{1}}{\epsilon_{2}}
\]

Equation 38

and

\[
\frac{\epsilon_{1}}{\epsilon_{2}} = \frac{1}{\sin \omega t}
\]

Equation 39
This finding compares very favourably to the results found by Ulditz et al. (2006) at the Westrak test track that the equivalent frequency of the flexural beam fatigue test to convert to field pulse loading (FWD loading) was 15Hz or 35ms which is nearly exactly the time of loading of an FWD.

6.6.4 Summary

The results shown that Dynamic modulus is only different to both resilient modulus and flexural modulus (haversine), if and only if, an inconsistent period (T) of the cycle is used. Stress dependency was found to play no part in the conversion of modulus between the Australian resilient and flexural modulus test and the dynamic modulus test for Australian mixes.

6.7 Master Curve Analysis

To further expand on the results found in the previous section for a single time and temperature the study was extended to examine the effects over the full range of temperatures and frequencies. This was done by developing a full set of master curves for 4 mixes and the 4 test methods used in Australia. For this study two sets of testing were undertaken:

- Set 1, which comprised of two mixes-a French EME2 mix and a high RAP C320 mix. Testing in this set was undertaken by ARRB for IT-CY resilient modulus and 4PB-PR. (It should be noted that in undertaking this study the research was undertaken in accordance with the new Austroads AGPT/T274 test protocol.) The DC-CY dynamic modulus testing was undertaken by both the University of the Sunshine Coast (USC) and by Fulton Hogan and two sets of results are shown.

- Set 2, contains the results of the development of two Australian EME2 mixes by Fulton Hogan. In this set of results all testing was undertaken by Fulton Hogan at the national laboratory, with one supplemental 4PB-PR beam undertaken by the Shell laboratories in France (EN12697-24). Along with the IT-CY, DC-CY and 4PB-PR test Fulton Hogan undertook a full temperature frequency sweep test using the 2PB-TR (EN12697-24) test (the reference test of AGPT-Part2 (2012)). For all test air voids were targeted at 5% +/- 1%.

6.7.1 Un-Shifted Data

The differences between the IT-CY, DC-CY and the 2PB-TR and 4PB-PR results can be seen in the following, Figure 53 a),b),c) and d), which shows the resulting master curves of the French EME2, the C320 high RAP mix, the NSW EME2 and the QLD EME 2 mix respectively.
As can be seen from Figure 53(a), (b), (c) and (d) across the full temperature frequency spectrum the DC-CY results are higher than the 4PB-PR results. For one mix, the NSW EME, the results did tend to converge with the DC-CY but were still always lower. These results would tend to confirm the finding of the single point high strain study that the tests exist in a different time domain.

The results show that the IT-CY results do tend to converge to the 4PB-PR results at low temperatures, they are however higher at high temperatures. The results show that as with the single point data the IT-CY samples modulus is always lower than that of the DC-CY dynamic modulus results.

The observation can be made that the 2PB-TR results appear to be equivalent to the DC-CY results and the same frequency should be used. This same observation was found with the single test undertaken by Shell Laboratories in France. This is somewhat contrary to the findings of Francken et al. (1994) who found that DC-CY modulus was out by a factor of two compared to the bending tests; this however was based on only 1 observation. This shows that more work is required on this conversion.

The result show that the testing of the C320 high by FH appears to be an outlier, with significantly different results to USC and the DC-CY dynamic modulus results obtained more in-line with that of
a C320 without RAP. While the results are included in the subsequent analysis, results should be view with caution.

6.7.2 Shifted Master Curves

The previous section (Section 4) established that frequency shift factors were valid for a single temperature and time of loading to equate modulus measured by one test to another. To determine whether the time shift factors found from the single time and temperature testing are valid across the whole time frequency domain, the results of the DC-CY, 4PB-PR and 2PB-TR were shifted with the correction factors obtained in section 4 for the 4 mixes tested. The results of the shifting is found in Figure 54 a,b,c and d for the French EME2, the C320 high RAP mix, the NSW EME2 and the QLD EME2 mix respectively. The DC-CY and 2PB-TR tests were shifted to $t_p$ using Equation 38 and the 4PB-PR test was shifted using Equation 39.

![Figure 54 Shifted Master Curves](image)

It can be seen from the figures that when corrected the 4PB-PR results for both the sinusoidal and the haversine loading fall closely around and both above and below the DC-CY master curve, the same is true for the IT-CY test and the 2PB-TR tests.

Given that the variability of the corrected results are well within the variability in modulus due to sample preparation (voids and binder differences) and the results fall above and below each other,
it can be concluded that the time shift factors found from the single time and temperature testing are valid across the whole time frequency domain and the shift factors can be used to shift data obtained from one test to a reference test at any temperature or loading time.

6.8 Summary Conclusions

The results of the study found that 3 of the 4 test methods have a different definition of time and need frequency shift factors to shift between the time and frequency domain. It was found the shift factors could be established from single time/temperature testing and that the time shift factors found from the single time and temperature testing are valid across the whole time frequency domain.

For the purposes of standardisation, the modulus results need to be converted to a reference stiffness value. Comparable stiffness is obtained, with reference to the Australian IT-CY test from the three test methods by using a constant definition of time (inter-conversion) as shown in Table 16, following.

<table>
<thead>
<tr>
<th>Conversion From</th>
<th>Conversion To</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IT-CY(time)</td>
</tr>
<tr>
<td>IT-CY(time)</td>
<td></td>
</tr>
<tr>
<td>4PB-PR(frequency)</td>
<td></td>
</tr>
<tr>
<td>DC-CY(frequency)</td>
<td></td>
</tr>
</tbody>
</table>

It is evident from this study the frequency in the common test can have different physical meaning and the reporting of modulus at a frequency, without reference to the loading type is confusing. To remove this confusion, it is recommended that standard practice be established for the reporting of modulus values from different test methods and is referenced back to an equivalent design modulus. The study underlines the considerable challenges in comparing the modulus results from various test methods. It would be highly recommendable to harmonise testing across Australia and implement a standard reporting method. The interconversions have a direct practical application in the recommended design approach of Chapter 9.

N.B. The conversion between IT-CY and 4PB-PR, would suggest that 12.5Hz frequency in 4PB-PR is equivalent to 0.04s rise time in IT-CY. This is close to current procedures in AGPT-Part 2, which assume unity between IT-CY and 4PB-PR is reached at 14.8 Hz, Austroads (2008).

For both the low strain 4PB-PR results and the 2PB-TR only limited data has been used and the results should be confirmed over a wider range of mixes.
7 Development of a Field Calibrated FEL model for Pavement Design

The purpose of this chapter is to develop a mathematical procedure for the incorporation of the FEL concept into the design of LLAP. As opposed to previous research projects that studied these concepts separately, the proposed procedure will incorporate the asphalt healing phenomenon directly into the FEL. To enable the design of LLAP in Australia the proposed procedure will need to incorporate the four major factors affecting the fatigue response of asphalt mixes; (i) binder type, (ii) binder content, (iii) temperature and (iv) the magnitude of the rest period applied after each loading cycle.

7.1 Background

Current conventional mechanistic design procedures, design pavements to fatigue, or put simply crack after a particular amount of traffic. This study proposes a different design concept. Instead of designing when the pavement will experience structural cracking, the concept is to design a pavement where structural cracking will never form. To design structural cracking out of the pavement, the concept will be to design the pavement structure in such a way that critical stress and/or strains remain below a ‘threshold’, or FEL level. By remaining below the FEL, at some point in the life of the pavement a balancing point will be achieved and no net damage will occur in subsequent loading cycles. As no subsequent damage occurs, micro cracking will never form into macro cracking, which then ensures no structural cracking and subsequent long-life performance.

The concept of endurance limits is not new, structural and aeronautical engineers have been using the principle for years in fatigue analysis of metals. However, asphalt behaviour is not as simple as metals. Asphalt material properties change with temperature and loading speed, along with this asphalt material can undergo healing (repair of damage), all these factors may affect the FEL and for these reasons no single FEL has not been established for asphalt mixes. Because a single FEL may not exist, the development of an effective and practical approach for the incorporation of the FEL across different environments, mixes and traffic speeds has proven difficult and to date has not been developed.

While the incorporation of a FEL into pavement design has proven difficult, if a threshold criteria could be incorporated into a practical pavement design process, a limiting thickness of asphalt could be determined, beyond which any increase in design thickness will result in no increase in the structural capacity of the pavement. This, if developed, will have a significant potential in lowering cost of construction and increasing sustainability.

As part of the APS-fL project, a validated procedure has been developed (refer Chapter 5) for the direct conversion of laboratory determined dynamic modulus to field stiffness and the subsequent prediction of strain under a moving load. The accurate prediction of strain was the first step in development of a LLAP design procedure, this prediction will then allow strain to be combined with the results of laboratory FEL analysis from sources such as the NCHRP 9-44 study, Thompson et
al. (2006), Australian mixes and actual performance from the NCAT accelerated test track data, to develop and calibrate a practical design method for LLAP design.

While current approaches for LLAP design make use of the single FEL, the latest research undertaken, such as Thompson et al. (2006) and the NCHRP 9-44 project (2013) have established that there is no single FEL for asphalt mixes, with the endurance limit changing as a function of temperature and mix type. This has resulted in significant debate over what is the single critical threshold value which should be used for LLAP design for different environments and different asphalt types. As a result, none of these threshold limits have been widely accepted or is transportable to different regions. As no validated LLAP modelling approach exists for a wide variety of mixes and operating conditions, this study examined the applicability of using three different threshold modelling methods:

- the single FEL criteria,
- the cumulative distribution of strain concept, and
- varying a FEL criterion which is a function of mix properties and operating conditions.

These three approaches were then assessed and calibrated against the performance of actual pavements at the NCAT test track. The assessment also assessed the transportability of the criteria to different loading conditions and environments and, most importantly, the practicality for use in pavement design. Based on this a recommended modelling approach is developed for validation against Australian long-life flexible pavements.

7.1.1 Scope

The scope of work covered in this chapter as part of the overall project comprised of:

- Examination of laboratory and other field based FEL and the factors which affect the FEL of asphalt mixes.
- Comparison of overseas laboratory studies to Australian data.
- Development and calibration of a modelling approach for the design of LLAP asphalt pavements incorporating FEL limits based on the results of the full scale NCAT test track findings.

7.2 Laboratory Studies on FEL

The early work on endurance limits by Monismith et al. (1972) recommended a single FEL (70mε) for LLAP design. However, subsequent research by Thompson et al. (2006) showed that the use of the single 70mε level was conservative and that the FEL was reached at significantly different strain levels depending on the binder type and or content. This finding was reinforced by the findings of the NCHRP 9-44 (2013) study which found the FEL was not only a function of binder type and content but, additionally, the test temperature and length of the rest period. These studies found
that the original assumption of a single FEL is not valid for a range of asphalt mixes and temperatures.

Thompson et al. (2006) attributed the differences in FEL to be a result of differences in binder. However, because of the lack of data on the binders used in the study, Thompson could only undertake an assessment against the stiffness. Nevertheless, Thompson found a high correlation existed between FEL and stiffness. This finding was confirmed and expanded by the NCHRP 9-44 (2013) study, which found that the use of mix stiffness alone was as accurate as the use of binder content, binder type and temperature to relate to the FEL. The conclusion of both of these studies was that the basic material property, stiffness, is an extremely good surrogate for mix variables and stiffness alone may be a practical approach and directly related to the FEL. The stiffness-FEL relationship is shown conceptually on Figure 55 following, which shows the recommended stiffness-FEL relationships from the NCHRP 9-44 (2014) study for both a 1 second and 10 second rest period and the data and the fitted relationship to the Thompson et al. (2006) findings. (N.B. In the figure the measured strain from the Thompson et al (2006) data has been halved to account for the different test methods (haversine vs. sinusoidal).)

While both approaches show a strong relationship between stiffness and FEL, there is a distinct difference in the predicted FEL, with different values and shapes of the stiffness-FEL curves. Notwithstanding the different values, it is evident that both approaches show a strong relationship between stiffness and FEL. These differences are most likely due to different definitions of the FEL.
being used in the testing, with the FEL recommended by Thompson et al. representing the point where cracking will not occur, but some damage may still occur to the mix, while the HI approach recommended by the NCHRP 9-44 study represents the case where no damage will occur (including internal heating).

While there is a difference in the value and shape of the curves, in all cases as stiffness increases the FEL decreases asymptotically and will most likely reach a limiting value, with the limiting value most likely being a function of the rest period. What is important is the not the definition of the FEL, it is the adoption of the approach which most closely represents field performance. Of the two approaches, it is believed that the FEL recommended by Thompson et al. (2006) will more closely match the definition of the FEL used in this project of `the maximum tensile strain at which, whilst damage might occur, the asphalt will never develop macro-cracking requiring deep structural treatment`.

The findings of both these two extensive studies undertaken by the University of Illinois and Arizona State University have shown that the use of the basic material property, stiffness, can be directly related to the FEL of a mix and can allow for healing. If this concept was to hold true in actual pavements this simplified approach could provide a practical tool which could be used for the purpose of LLAP design incorporating binder changes, healing and temperature effects.

7.2.1 Other FEL Recommendations

Thompson et al. (2006) and the 9-44 (2013) study showed that the FEL of an asphalt mix varies due to binder grade, binder content, air voids and temperature and as found by Lytton (2005) and Zeiada (2012) with the length of rest periods. It is therefore not surprising that different researchers have recommended different FEL for LLAP design, based on the mixes they assessed, the operating (or testing) temperature and the environment, such as:

- In Japan Nishizawa (1996) analysed in-service pavements in Japan and recommended an endurance limit of 200me.
- In Kansas, Wu et al. (2004) using back-calculated Falling Weight Deflectometer (FWD) data, reported strains at the bottom of the asphalt layer between 96 and 158me for a long-life pavement.
- Bhattacharjee et al. (2009) obtained endurance limit values through uniaxial testing which ranged from 115-250me.
- Thompson and Carpenter suggested fatigue endurance limits under certain circumstances may indeed be above 350me.

7.3 Single FEL Design Approach

Notwithstanding the acceptance of the FEL concept, the wide-ranging recommend FEL found in the literature has resulted in confusion on the actual value of the FEL which should be used and
when the FEL can be used in pavement design. As a result the use of FEL for routine pavement
design has not gained general acceptance. In addition to the inconsistency of the published FEL,
there is complexity in the translation of the limited laboratory conditions (single temperature and
rest period) to the multitude of conditions experienced in the field with complex vehicle loadings,
rest period and a range of operating temperatures and conditions, which all result in changing FEL.

It is clear from the examination of both the results of the laboratory testing and field based
recommendations that the use of a `single` FEL is not practical for a range of mixes and is not
transportable to different environments. Any LLAP design procedure must include a variable FEL
which can cover changes in temperature and mix type. The fact that different FEL's are obtained
for different mixes and binder types and that the FEL changes with changes in temperature make
the use of a single FEL impractical for general pavement design. As such, the use of single FEL
was excluded for development of the APS-fl project, as it would not be transportable between the
different mixes used across Australia and to the different environments of Australia.

Previous research has found that any practical design approach needs to incorporate a variable
FEL, which is able to accommodate changes in temperature, rest periods and changes in mixes.

7.4 Design Endurance Limit as a Distribution of Strain at Failure

Realising the significant limitations in applying a single endurance limit for the purpose of design,
the NCAT researchers developed the concept of using the Cumulative Distribution of Strain (CDS)
as a method for the design of LLAP. In reality, the CDS approach determines a pavement which
limits the number of load applications which can exceed the FEL of the asphalt, for a given
environment, resulting in a LLAP.

The CDS allows incorporated changes in the FEL, as a result of changes in temperature, by the
use of the distribution of strains and may be a practical approach for pavement design. The concept
of the CDS approach is that if the distribution of strains (at the underside of the asphalt layer) is
kept below a tolerable distribution of strains (the threshold distribution), a LLAP will be achieved.
The concept of the cumulative distribution of strain is illustrated in the Figure 56 following.
The CDS concept was developed by the NCAT researchers by plotting the cumulative measured strain for each of the structural sections (test cell) of the NCAT test track. In examination of the distribution of strains of the structural sections the researchers found, that as logically expected, strains increased with increased temperature as a result of the decrease in the stiffness of the asphalt. Surprisingly though, the researchers found that as the stiffness of the asphalt decreased with increasing temperature, strains in excess of 400με were recorded on sections with no cracking. These field observations are well above the 70με adopted by early researchers, again showing the difficulty in using a single FEL for LLAP design.

The threshold distribution proposed for design was developed by comparing the calculated cumulative distribution of strain for each test section against the field performance. When comparing the results the researchers found that a distinct difference was observed in the distributions between those sections which cracked (failed) and those sections which did not.

The examination of the results suggested to the researchers that there was a limiting value to the CDS or a threshold strain distribution, which if the strains in the pavement remained below, the pavement would avoid cracking and therefore be a LLAP. NCAT subsequently published this distribution as an interim threshold distribution for LLAP design.

### 7.4.1 Issues with Cumulative Distribution of Strain

While the cumulative distribution of strain is considered to be a practical, simple and rational design approach and able to include the changes in the FEL with changes in temperature or mix properties,
the use of the published CDS across the range of temperatures experienced in Australia and loading conditions does have some limitations, namely:

- The maximum loading on the NCAT test track to date, is approximately $6 \times 10^7$ ESA. While this a LLAP, this load level has not been agreed to define LLAP and the CDS may not equate to a true LLAP

- The assumption that there is one distribution of strain which can be used as a threshold level for pavement design has limitations in:
  - The threshold level is the same regardless of mix type, ignoring mix composition. This is contrary to evidence that stiffer asphalt mixes have lower threshold limits than more flexible mixes.
  - The threshold level is the same regardless of the environment, meaning that the thickness in requirements in Brisbane (hotter climate and therefore higher strains) will be greater than Melbourne (cooler climate and lower strains). This is contrary to testing which shows increasing FEL with temperature and the empirical evidence that longer lives are experienced at higher temperatures.
  - There is no fundamental test developed that could be used to predict the required CDS for alternative mixes or which could be used for design and quality control purposes.

7.5 Stiffness Based FEL Relationships

The NCHRP 9-44 study (2013) recommended that the basic material property, stiffness, be used to develop a practical stiffness-FEL relationship for LLAP design. The advantage of the use of stiffness is that it can be used as a surrogate for changes in mix properties and temperature, eliminating many of the issues associated with the CDS approach, namely transportability with temperature and changes in asphalt mixes.

7.5.1 Incorporating Australian Mixes

Before the use of a single stiffness-FEL relationship could be recommended for routine pavement design in Australia, it needed to be established whether the single stiffness-FEL relationship found for a range of US mixes held for Australian mixes. To examine this, a range of multi strain 4PB-PR fatigue tests undertaken on Australian mixes in the Fulton Hogan historical database were assessed to determine the FEL of each mix. The FEL of each mix was determined using the constant PV of the RDEC approach and the constant strain-Nf to failure level ($1.1 \times 10^7$) recommended by both Thompson et al. (2006) and the NCHRP 9-38 (2011) project.

The results of this analysis is summarised in Table 17 following, which documents the mixes used in the analysis, the slope and intercept of the straight line strain-Nf to failure curve and the estimated FEL. These results are also shown graphically on Figure 57 following.
Table 17 FEL Australian Mixes

<table>
<thead>
<tr>
<th>Mix Description</th>
<th>Binder %</th>
<th>Binder Type</th>
<th>Fatigue Equation Constants</th>
<th>Initial Stiffness (MPa)</th>
<th>FEL (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC14 5.4% A5E</td>
<td>5.4</td>
<td>A5E</td>
<td>2127 6.9</td>
<td>9722</td>
<td>150</td>
</tr>
<tr>
<td>EME NSW Bass Point Mix</td>
<td>5.6</td>
<td>EME2</td>
<td>3295 6.1</td>
<td>9021</td>
<td>168</td>
</tr>
<tr>
<td>Fulton Hogan PortPhalt</td>
<td>5.4</td>
<td>APH</td>
<td>1243 9.1</td>
<td>8580</td>
<td>169</td>
</tr>
<tr>
<td>AC20 C320 Wallgrove (AS 2150)</td>
<td>4.2</td>
<td>C320</td>
<td>2740 5.6</td>
<td>9178</td>
<td>109</td>
</tr>
<tr>
<td>14mm Granite C320 Perth</td>
<td>4.5</td>
<td>C320</td>
<td>3676 5.2</td>
<td>7996</td>
<td>110</td>
</tr>
<tr>
<td>AC20 AR 450 High RAP</td>
<td>4.7</td>
<td>AR450</td>
<td>3801 4.7</td>
<td>8519</td>
<td>81</td>
</tr>
<tr>
<td>C320 Geelong High RAP</td>
<td>4.9</td>
<td>C320</td>
<td>3413 4.8</td>
<td>7947</td>
<td>77</td>
</tr>
<tr>
<td>14TCI Base (sample 1)</td>
<td>4.9</td>
<td>C320</td>
<td>3587 5.2</td>
<td>8848</td>
<td>110</td>
</tr>
<tr>
<td>14TCI Intermediate sample 1</td>
<td>4.9</td>
<td>C320</td>
<td>2880 5.6</td>
<td>9111</td>
<td>111</td>
</tr>
<tr>
<td>MSTR Bunbury</td>
<td>5.2</td>
<td>C320</td>
<td>3521 6.0</td>
<td>4790</td>
<td>170</td>
</tr>
<tr>
<td>AC14 Hun 5.3% (NZ)</td>
<td>5.3</td>
<td>PG64</td>
<td>5163 5.0</td>
<td>5681</td>
<td>134</td>
</tr>
<tr>
<td>AC14 5.4% C320</td>
<td>5.4</td>
<td>C320</td>
<td>3441 5.3</td>
<td>6828</td>
<td>107</td>
</tr>
<tr>
<td>AC14 C600</td>
<td>5.4</td>
<td>C600</td>
<td>2816 5.8</td>
<td>7256</td>
<td>122</td>
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<tr>
<td>DG10 C170 W.A. Granite</td>
<td>5.6</td>
<td>C170</td>
<td>4853 5.0</td>
<td>6125</td>
<td>126</td>
</tr>
<tr>
<td>AC10 5.7% C450</td>
<td>5.7</td>
<td>AR450</td>
<td>3599 5.2</td>
<td>8947</td>
<td>109</td>
</tr>
<tr>
<td>AC10 5.7% C320</td>
<td>5.7</td>
<td>C320</td>
<td>3266 5.3</td>
<td>7730</td>
<td>109</td>
</tr>
<tr>
<td>DG10 Granite Perth 5.1%</td>
<td>5.1</td>
<td>C320</td>
<td>4578 5.0</td>
<td>5738</td>
<td>120</td>
</tr>
<tr>
<td>10mm DG Granite (21) Perth</td>
<td>5.1</td>
<td>C320</td>
<td>4955 5.0</td>
<td>7214</td>
<td>128</td>
</tr>
<tr>
<td>10mm DG Granite (22) Perth 2</td>
<td>5.1</td>
<td>C320</td>
<td>5012 5.0</td>
<td>7218</td>
<td>131</td>
</tr>
<tr>
<td>AC14 Hun 4.65% Bitumen PGT64 15% RAP 10</td>
<td>4.65</td>
<td>PG64</td>
<td>2467 7.2</td>
<td>5807</td>
<td>195</td>
</tr>
<tr>
<td>DG10 Granite</td>
<td>5.1</td>
<td>A15E</td>
<td>5765 4.8</td>
<td>6021</td>
<td>133</td>
</tr>
<tr>
<td>DG14 4.72% A15E ex-Crestmead</td>
<td>4.7</td>
<td>A15E</td>
<td>5570 5.0</td>
<td>7510</td>
<td>149</td>
</tr>
<tr>
<td>AC14 5.4% Toner Binder (MK II) 2 M2 B2</td>
<td>5.4</td>
<td>A10E +</td>
<td>2268 8.5</td>
<td>3594</td>
<td>265</td>
</tr>
<tr>
<td>AC14 5.4% A10E</td>
<td>5.4</td>
<td>A10E</td>
<td>4012 6.7</td>
<td>3504</td>
<td>262</td>
</tr>
<tr>
<td>AC10 5.7% A10E</td>
<td>5.7</td>
<td>A10E</td>
<td>6768 5.5</td>
<td>3342</td>
<td>243</td>
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<tr>
<td>AC10 5.7% A15E</td>
<td>5.7</td>
<td>A15E</td>
<td>7014 5.7</td>
<td>3423</td>
<td>284</td>
</tr>
<tr>
<td>AC10 5.7% A20E</td>
<td>5.7</td>
<td>A20E</td>
<td>6653 5.1</td>
<td>4347</td>
<td>188</td>
</tr>
<tr>
<td>WA Testing PMB AC14 Ex Port Hedland</td>
<td>5.1</td>
<td>A10E</td>
<td>5350 5.0</td>
<td>6966</td>
<td>142</td>
</tr>
<tr>
<td>10mm Granite /75 blows C320 + PMB</td>
<td>5.0</td>
<td>A15E</td>
<td>6412 5.3</td>
<td>5002</td>
<td>200</td>
</tr>
<tr>
<td>DG10 Granite</td>
<td>4.8</td>
<td>A15E</td>
<td>5765 4.8</td>
<td>6021</td>
<td>133</td>
</tr>
<tr>
<td>14mm MS R Basalt A20E ex-Bunbury</td>
<td>5.0</td>
<td>A20E</td>
<td>5490 5.0</td>
<td>4826</td>
<td>145</td>
</tr>
<tr>
<td>Material Description</td>
<td>E (%)</td>
<td>A (%)</td>
<td>C (%)</td>
<td>D (%)</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
<td></td>
</tr>
<tr>
<td>AC14 5.0% A35P Racetrack</td>
<td>5</td>
<td>A35P</td>
<td>6286</td>
<td>4.2</td>
<td>6891</td>
</tr>
<tr>
<td>AC14 5.4% A35P</td>
<td>5.4</td>
<td>A35P</td>
<td>3578</td>
<td>5.6</td>
<td>7789</td>
</tr>
<tr>
<td>AC10 5.7% A35P (EVA)</td>
<td>5.7</td>
<td>A35P</td>
<td>5115</td>
<td>5.0</td>
<td>7120</td>
</tr>
<tr>
<td>14mm Granite PMB</td>
<td>4.8</td>
<td>A35P</td>
<td>4648</td>
<td>5.0</td>
<td>7010</td>
</tr>
<tr>
<td>20mm Granite EVA ex-Hazelmere</td>
<td>4.3</td>
<td>A35P</td>
<td>5206</td>
<td>4.4</td>
<td>10273</td>
</tr>
<tr>
<td>14mm Granite EVA ex-Hazelmere</td>
<td>4.8</td>
<td>A35P</td>
<td>3824</td>
<td>5.1</td>
<td>6865</td>
</tr>
<tr>
<td>10mm Granite /75 blows C320 + EVA</td>
<td>5.1</td>
<td>A35P</td>
<td>9785</td>
<td>3.6</td>
<td>8920</td>
</tr>
<tr>
<td>DG10 EVA ex-WA</td>
<td>5.1</td>
<td>A35P</td>
<td>5834</td>
<td>5.1</td>
<td>5507</td>
</tr>
</tbody>
</table>

Figure 57 FEL Stiffness Curve Australian Mixes

What can be seen in the figure is that the results of the conventional, elastomeric (A10 and A15E) and plastomeric (A30 and A35P) mixes, follow the same shape as the results found in the Thompson et al. (2006) study. This finding is not overly surprising as both datasets contained both conventional and SBS modified mixes and under the same loading conditions (haversine). As with the results of Thompson et al. (2006) the SBS modified mixes in general fall slightly above that of the conventional mixes, suggesting a different FEL-stiffness relationship may be required for modified mixes, but for practical purposes may not be warranted.

Given the similarities in the results both the Thompson et al. (2006) data and the Australian, the data was combined to develop a standard stiffness-FEL relationship which can be used to estimate the FEL for conventional, EVA and SBS modified mixes and is described in Equation 40 following:
Where:

\[
\text{FEL is the Fatigue Endurance Limit} \\
S_{\text{mix}} \text{ is the stiffness of the mix, and} \\
k_1 \text{ is an adjustment factor for differences in rest periods and/or confidence levels.}
\]

While it was initially believed and recommended in the 9-44 study (2013) that a single FEL could be established that related stiffness to the FEL for any mix, this only appears to hold true for a range of conventional and modified asphalts, with binder contents in the range of typical asphalt mixes (4.2-5.5% binder) and it does not appear to be true for highly modified mixes. This can be seen in Figure 57 which shows that for mixes produced with high modification or high binder contents significantly higher FEL's can be achieved and a shift factor may be needed for those mixes.

It is also noted that the two mixes examined produced with high RAP contents (>30%) exhibited significantly lower FEL. While this dataset is only small and may not hold with the use of rejuvenators, until additional studies are undertaken it is not recommended that high RAP mixes (>30%) be used in the fatigue susceptible layers of a LLAP.

7.6 Calibration of FEL for Pavement Design

7.6.1 Shifting of Cumulative Distribution of Strain

To overcome the limitations of the single environment used in the development of the CDS and to modify the CDS to incorporate a measure of the mix property, the results of the Phase II NCAT test sections were re-examined in an attempt to incorporate either the standard stiffness-FEL relationship shown in Equation 40 and the NCHRP 9-44(2013) recommendations into the CDS approach.

It was hypothesised that if the single stiffness-FEL relationship holds true under field loadings and temperatures conditions, the relationship between stiffness and FEL may provide a practical tool for the shifting of the threshold distribution curve as a function of changes in mix type and/or environments.

To test this hypothesis, the following approach was undertaken:

- The effective modulus master curve of the combined asphalt layers for each section of the Phase II structural sections (N2-N8) was determined in accordance with the previously developed and calibrated method (Chapter 5), with the resulting master curve fitting parameters shown in Table 18 following.

Table 18 Master Curve Fitting Parameters
<table>
<thead>
<tr>
<th>Test Cell</th>
<th>Temperature Shift Factors (T_{ref}=20^\circ C)</th>
<th>Sigmoidal Fitting Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>N2</td>
<td>a = 0.0003, b = -0.1173, c = 1.763, d = 2.828</td>
<td></td>
</tr>
<tr>
<td>N3</td>
<td>a = 0.0003, b = -0.1163, c = 1.810, d = 2.774, g = -0.711, h = -0.363</td>
<td></td>
</tr>
<tr>
<td>N4</td>
<td>a = 0.0005, b = -0.1291, c = 1.688, d = 2.896, g = -0.770, h = -0.347</td>
<td></td>
</tr>
<tr>
<td>N5</td>
<td>a = 0.0004, b = -0.1193, c = 1.688, d = 2.896, g = -0.770, h = -0.347</td>
<td></td>
</tr>
<tr>
<td>N6</td>
<td>a = 0.0003, b = -0.1184, c = 1.715, d = 2.882, g = -0.709, h = -0.372</td>
<td></td>
</tr>
<tr>
<td>N7</td>
<td>a = 0.0004, b = -0.1221, c = 1.787, d = 2.897, g = -0.998, h = -0.457</td>
<td></td>
</tr>
<tr>
<td>N8</td>
<td>a = 0.0004, b = -0.1221, c = 1.787, d = 2.897, g = -0.769, h = -0.5060</td>
<td></td>
</tr>
</tbody>
</table>

The stiffness was determined for the effective asphalt layer, over the full range of mid-point layer temperatures experienced at the NCAT test section (0, 5, 10, 15, 20, 30 and 40°C) at the equivalent loading frequency in the DC-CY dynamic modulus test (2Hz), equating to the design speed of 80km/hr.

The design axle loading used in the forward calculation was the 90kN (axle weight on the NCAT test track).

For each mid-point temperature, the horizontal strain at the base of the combined asphalt layer, the critical location for fatigue cracking, was calculated using the elastic material properties of the base and subgrade support values from Timm (2009) and Layered elastic theory.

For each temperature the calculated strains at the asphalt stiffness were plotted against each other for each Phase 2 structural test section as shown in Figure 58 following.
It is not surprising and expected to see, that a clear relationships exists for each mix between stiffness and strain and that that stiffness alone may be able to be used to 'shift' the threshold CDS. The results show, as logically expected, that as stiffness decreases with increasing temperature the strain also increases and additionally, as can be seen from section N3 and N4, softer mixes produce higher strains. This not surprisingly shows the effect of both the changes in temperature and mix type are captured in the stiffness-strain relationship for a pavement.

7.6.2 Development of a Stiffness Based FEL

When the stiffness-strain relationships are more closely examined as shown in Figure 59 following, it can be seen that two sections N4 and N3, which did not crack in the Phase II and the subsequent studies, had strain levels across the whole stiffness range which are lower than the sections which cracked. As with the CDS approach the figure suggest that a deviation exists between sections which did not exhibit structural failure (lack of cracking) (N3 and N4) and those which did exhibited failures (cracking) (N2, N5, N6, N7 and N8). As can be seen in the figure, the sections which failed had strain levels across the whole stiffness (temperature) range, which were higher than the sections which exhibited no signs of failure. These field observations support the results of the laboratory findings in that;
stiffness as found by Thompson et al. (2006) in the 9-44 study (2013) and for Australian mixes is a good surrogate for binder and temperature effects,

- FEL is not a constant and varies as a function of stiffness,
- and, structural failure or lack of, is a function of induced strains.

These observations also show that strain alone is an effective tool for assessing the support provided to the asphalt layer(s) from all underlying layers. This observation of the structural sections on the NCAT test track would support that a single stiffness-FEL may alone be a practical tool for the design of LLAP and the design procedure may not need to include the CDS approach.

Figure 59 Modulus Strain Relationships NCAT

While it might be argued that sections N3 and N4 have not been fully established as LLAP, as described by Timm et al. (2009) sections N3 and N4 strain profiles can be used to represent the least conservative strain profiles which are able to withstand trafficking without fatigue cracking. Therefore, at least initially, these two sections can be used to determine the deviation or upper limit for a field-based FEL and will provide an excellent starting point for development of a field based FEL design model. However, due to this limitation it is not recommended that the FEL relationship developed from the analysis of the NCAT test sections be used for design and that any recommendation needs to be further validated from investigation of actual LLAP’s.
Comparing the relationship between strain and stiffness for the two LLAP sections to the often used laboratory FEL of 70 to 100mε, it can be seen that for both sections strain values are constantly higher than these values. The results from NCAT clearly show that measured strains in the field can easily exceed the previously accepted laboratory FEL and in fact, strains in excess of 400mε can be experienced without any fatigue damage occurring. The infield performance shows that the 70 to 100mε level is clearly conservative and confirms that the use of a single FEL will not provide a useable design approach.

While different binders (conventional and SBS modified) were used in the N3 and N4 sections, it can be seen that the relationship between stiffness and strain for these two sections, for all practical purposes is identical. The relationship was the same even though section N4 was placed with lower stiffness mix which resulted in higher strains at a given temperature. This clearly shows that any FEL design procedure must incorporate a mix property in the process and the basic material property, stiffness, can be used to do this. As if non-stiffness based FEL was developed, the approach may predict failure of section N4 before N3 which was not observed.

The findings from the NCAT test track confirm the findings of the laboratory test studies, (NCHRP 9-44 (2013) study, Thompson et al. (2006) and validated with Australian mixes), that the use of a variable FEL limit directly related to stiffness, is able to capture the effects of changes in temperature and mixes in the field and the tolerable strain and may be able to provide a simple practical approach for design which can be calibrated to field performance.

This simple approach offers significant advantages over the previous CDS approach, in that:

- It is transportable to different stiffness (low modified) mixes, through binder properties being a direct input to stiffness
- It is transportable to different environmental conditions through temperature being a direct input to stiffness.

7.6.3 FEL Relationship and Laboratory Shift from NCAT

As previously stated, Timm et al. (2009) recommended that the strain profiles of sections N3 and N4 be used to represent the least conservative strain profiles that were able to withstand trafficking without fatigue cracking. Due to the LLAP performance of these two sections the sections were extended into the two latter studies and have subsequently received triple the traffic of any other section in the 2003 study, and have shown no signs of structural failure. It is therefore rational that these two sections be used, at least initially, to determine and calibrate the upper limit for a field-based fatigue threshold. While it needs to be understood that the cells N4 and N3 at the current time have only experienced a traffic level (6x10^7) which is not generally considered to be a true LLAP, they are close to if not at the accepted endurance level and will provide a good starting point for development of the threshold. Given this potential limitation, any LLAP design procedure
calibrated from the NCAT test track data will need to be further validated against actual LLAP asphalt pavements, it is anticipated that there will still be some shift from the NCAT calibration to account for potential aging and higher traffic levels of actual LLAP.

Further to developing and calibrating the stiffness-FEL relationship, the analysis will need to establish whether any laboratory to field conversion exists. This is relatively simple to accomplish and can be undertaken by comparing the recommendations of the NCHRP 9-44 (2013) study and the standard relationship (Equation 40) developed from the Australian mixes and the Thompson et al. (2006) data, against the threshold established from the structural sections of the NCAT test track. This can be seen in Figure 60 following which shows the stiffness strain relationship of sections N3 and N4, the standard relationship and the two of the NCHRP9-44 recommended relationships. When comparing the results it needs to be understood that in the standard relationship, there is no rest period, for the 9-44 study the recommended FEL were made for rest periods of between 1 to 20 seconds. Plotted on the chart is the case of a 20 second rest period, which has the highest recommended permissible strains.

![Figure 60 NCAT Calibrated FEL Relationship](image)

The figure shows that at high temperatures the FEL from the standard relationship is a factor lower than the values which have been shown to not produce fatigue in the field, but they converge at the lower temperatures. The comparison with field performance does however suggest the shape of the FEL relationship of the standard equation closely matches the shape of that of the observed field performance and might be out by a shift factor. The requirement for a shift is not surprising as the relationship was developed from testing undertaken without any rest periods, as found by...
numerous researchers, such as Zeiada (2012), rest periods have a significant effect on the endurance limit, with recorded FEL being up to double in samples with rest periods and compared to samples without rest periods.

The figure also shows that the Healing Index (HI) approach as proposed in NCHRP 9-44 (2013) does not appear to match field performance across the range of modulus values encountered at NCAT. It is clear that at higher temperatures, even at high rest periods (20sec), the approach underestimates the strain values which have been shown to not cause damage. For the N3 and N4 sections strains of well in excess of 300mε were experienced on the test track, while the recommendations only allows a maximum of approximately 180mε at high temperatures and the relationship crosses the field observation curve at a stiffness level of 4000MPa. The results would tend to indicate that it will be difficult to apply a simple correction factor to the recommended approach. A simple calibration factor will result in either underestimating the fatigue endurance limit at higher temperatures or over estimating the endurance limit at lower temperatures.

While it is clear and expected that there will need to be a shift factor between laboratory and field FEL, for the standard relationship, this may be a simple constant applied to the calculated strain and this shift factor may be all that is required to shift the results from the laboratory to field.

Using both the results of the NCAT sections N3 and N4 and the standard relationship a preliminary relationship and laboratory to field correction can be obtained, as shown in the Equation 41 following and as shown in Figure 60, previously.

\[
FEL = 1.4 \times S_{mix}^{-0.9}
\]

Equation 41

Where;

- \( FEL \) = is the NCAT Fatigue Endurance Limit; and
- \( S_{mix} \) = asphalt mix stiffness (modulus) (MPa)
- \( k_1 = 1.4 \) and is the laboratory to field adjustment factor.

7.7 Conclusions

The examination of extensive overseas research showed that there is a clear and strong relationship between mix stiffness and the FEL of asphalt mix. The international research has found that as stiffness of the mix increases the FEL decreases asymptotically and most likely reaches a limiting value. The research work showed that the use of the basic material property, stiffness, is directly related to the FEL of a mix and can be used to allow for changes in binder, temperature and healing.

The examination of the LLAP sites from the NCAT Phase II study confirmed, in field, the variable nature of the FEL relationship and that LLAP can withstand strains significantly higher than previously recommended when the asphalt has low stiffness without undergoing damage. The examination of the two LLAP on the NCAT test track showed a direct relationship exists between
infield stiffness-strain curve of the two undamaged sections and the stiffness-FEL developed from Australian and US mixes.
8 Validation with Australian Long Life Asphalt Pavement Sites

8.1 Introduction

The calibration of the stiffness-FEL relationship developed from the NCAT test track found that a single stiffness-FEL relationship could be used as a practical design tool for LLAP design and this relationship was directly related to laboratory testing. The results found that the difference in binder content and grades as well as temperature could be adequately covered using this single stiffness-FEL relationship. However, it still needs to be remembered that this relationship is only a convenient observational relationship and must be only applied within the limits of the data used to develop the relationship. Given the relationship is not a fundamental; the application of the single relationship to the Australian environment and mixes is not recommended.

Given there is expected to be differences in traffic loading, environments, binder sources and most especially aging, it would be expected that further validation may be required from the NCAT relationship to Australian field performance. However, when investigating this, it needs to be recalled (see Chapter 5) that typical Australian asphalt mixes fall into a relatively small volumetric window. Also, Australian mixes, in terms of stiffness, binder content and gradation are no different from US mixes, as confirmed in the comparison of Australian and US FEL. Therefore, it may be the case that the practical single relationship found between stiffness-FEL for US data on the NCAT test track may require only a small, if any, shift to be transportable to Australian mixes and conditions.

8.2 Australian Long Life Pavement Sites

In 2009 the then RTA of NSW (now Roads and Maritime Services, RMS)) undertook a study on the performance, composition and condition of the different pavement structures at a number of locations throughout the state in order to develop the STEP remaining life procedure. One of the outcomes of this study was an extensive database of pavement condition, structural capacity, layer thicknesses, materials and visual condition of the pavements. This database was made available to AAPA for use in the APS-FL life project for development and validation of a LLAP design procedure for Australia.

In order to validate the stiffness-FEL model developed from the NCAT test track, the RMS step database was examined to find both partial and full depth asphalt pavements which may be either a non LLAP or a LLAP. This subset of the RMS step database was then re-examined to separate out potential LLAP in the database. This subset was obtained by filtering for pavements which;

- had greater than 140mm of combined asphalt thickness
- were greater than 20 years old,
- had cumulative traffic greater than 30million ESA,

The subsequent subset of the data can be found in Appendix B of this report.
Additional sites were added to the analysis, which could be used to establish the break point between LLAP and fatigue susceptible pavements. These were sites which had greater than 240mm of asphalt and have fatigue cracking, regardless of the traffic loading and or age.

The potential LLAP sections were then individually assessed and categorised into three categories:

- LLAP,
- Possible LLAP
- or Non LLAP

The categorisation was undertaken by examining the results of the visual survey undertaken at the time of testing, the current visual condition of the pavement and any comments on asphalt condition and by examining the assessment of the remaining life of the pavement determined as part of the visual assessment.

In addition to the RMS sites the data collected by Sharp (2001) for the AAPA pilot study was inspected to find any additional sites which could be used to establish the FEL for Australian LLAP design. The data collected by Sharp(2001) and subsequently by Foley (2008) was examined to find Australian LLAP sites which would provide valuable information in the establishment of the design level FEL for asphalt pavements. In his study Sharp (2001) identified a number of LLAP and undertook site investigation of asphalt material properties and layer thickness, Foley (2008) undertook subsequent FWD testing and analysis on these sites.

Table 19, following, summarises the sections of pavement identified as LLAP, while Table 20 summarises the sections identified as Non-LLAP. The sites obtained from Sharp (2001) and Foley (2008) are marked with an asterisk (*) in Table 19 following. The full details of the geotechnical investigation and material properties for the LLAP sections and the non LLAP sections can be found in Appendix B.

Table 19 Australian LLAP Validation Sites

<table>
<thead>
<tr>
<th>Site ID</th>
<th>Road/Location</th>
<th>Cumulative Traffic</th>
<th>Const. Year</th>
<th>Pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-S22</td>
<td>New England Highway Beresfield</td>
<td>8x10^7</td>
<td>1x10^8</td>
<td>1970</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>190-210mm Asphalt Fine to Course Gravel</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Poorly Graded Sand</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SG</td>
</tr>
<tr>
<td>H-S27</td>
<td>Pacific Motorway, Cheero Point</td>
<td>5x10^7</td>
<td>6x10^7</td>
<td>1990</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>180-205mm Asphalt Fine to Coarse Gravel</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fine to Coarse Gravel</td>
</tr>
<tr>
<td>H-S28</td>
<td>Pacific Motorway, Mooney</td>
<td>5x10^7</td>
<td>6x10^7</td>
<td>1990</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>265-285 Asphalt Fine to Coarse Gravel</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Fine to Coarse Gravel</td>
</tr>
<tr>
<td>H-S29</td>
<td>Pacific Motorway</td>
<td>5x10^7</td>
<td>6x10^7</td>
<td>1989</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>140-160mm Asphalt</td>
</tr>
<tr>
<td>Site</td>
<td>Location</td>
<td>Depth</td>
<td>Year</td>
<td>Material Details</td>
</tr>
<tr>
<td>-------</td>
<td>----------</td>
<td>-------</td>
<td>------</td>
<td>------------------</td>
</tr>
<tr>
<td>H-S48</td>
<td>Maitland Road Westgate</td>
<td>6x10⁷</td>
<td>1970</td>
<td>130-140mm Asphalt, 260-370mm Gravel, 400-200mm Clayey Gravel, Clay</td>
</tr>
<tr>
<td>N-S56</td>
<td>Pacific Highway, Cumbalum</td>
<td>1x10⁷, 2x10⁷</td>
<td>1988</td>
<td>200mm Asphalt, 300mm Argillite, Silty Sand</td>
</tr>
<tr>
<td>SOU-S035</td>
<td>Picton Road</td>
<td>4x10⁷, 5x10⁷</td>
<td>1966</td>
<td>215-280mm Asphalt, 1000mm Sandy Iron Stone Gravel, Bedrock</td>
</tr>
<tr>
<td>Syd-S11</td>
<td>Botany Road, Banksmedow</td>
<td>4x10⁷, 4x10⁷</td>
<td>1977</td>
<td>205mm Asphalt, Fine to Coarse Gravel, Poorly Graded Sand</td>
</tr>
<tr>
<td>Syd-S13</td>
<td>Hume Motorway, St Andrews</td>
<td>1x10⁸</td>
<td>1973</td>
<td>260-270mm Asphalt, Clayey gravel, SG</td>
</tr>
<tr>
<td>Syd-S14</td>
<td>Milperra Rd, Condell Park</td>
<td>4x10⁷, 4x10⁷</td>
<td>1966</td>
<td>150mm Asphalt, 650mm Fine to Coarse Gravel, Clayey Sand</td>
</tr>
<tr>
<td>Syd-S15</td>
<td>Hume Motorway, St Andrews</td>
<td>1x10⁸</td>
<td>1973</td>
<td>300mm Asphalt, 175mm Gravel, Clay</td>
</tr>
<tr>
<td>Q5*</td>
<td>Bruce Highway, Kallangur, 1.3-1.5km South of Boundary Road (SB)</td>
<td>3x10⁷</td>
<td>1979</td>
<td>320mm Asphalt, Working Platform Subgrade</td>
</tr>
<tr>
<td>Q6*</td>
<td>Bruce Highway, 2.2-2.4km South of Boundary Road (SB)</td>
<td>3x10⁷</td>
<td>1979</td>
<td>300mm Asphalt, Working Platform Subgrade</td>
</tr>
<tr>
<td>N4*</td>
<td>Camden By-Pass Narellan Rd to Macarthur Bridge (SB)</td>
<td>9x10⁶</td>
<td>1976</td>
<td>180mm Asphalt, 220mm DGB20, 330mm SGS</td>
</tr>
<tr>
<td>N7*</td>
<td>Alpha Street, Patrick to Flushcombe Rd, EB</td>
<td>3x10⁶</td>
<td>1974</td>
<td>225mm Asphalt, Subgrade</td>
</tr>
<tr>
<td>V6*</td>
<td>Atherton Road, Oakleigh, Drummond St to Atkinson St (EB)</td>
<td>4x10⁶</td>
<td>1971</td>
<td>190mm Asphalt, 75mm FCR, Sandy Clay</td>
</tr>
</tbody>
</table>

For most of the RMS sites, 3 individual test pits, FWD tests and borehole logs were undertaken within the section. While for each section similar profiles were found, there were differences in the thickness of each individual layer and material types. The combination of the result of the 3 locations per RMS site, and the use of the lower 90th percentile deflection for the Sharp (2001) sites, resulted in 28 individual Australian LLAP sites being used in the analysis. To further expand on this data set and add an extra degree of confidence, these 28 sites were then with combined with the data obtained for the three lowest stiffness Valmon test sites and the structural sections of the NCAT test track data. The resulting LLAP database comprising of 33 pavements, which was selected to establish the FEL for LLAP design is statistically significant for the establishment of the FEL criteria.
In addition to the LLAP identified for use in this study a significant number of additional sites are available in the literature from other studies such as, Foley(2008), Sharp(2001) and Rickards et al.(2012). If these sites were included in the analysis, it would have been possible to have over 50 LLAP sites. However, as with the stiffer analysis sections in the TRL UK Valmon data, these sites are principally deep strength asphalt pavements over modified, stabilised or cemented base. As the strain levels in these pavements are well below that of the FEL the additional information obtained from the analysis of these sites would offer little to no benefit to the validation of the FEL, as what is important is to distinguish the shift from indeterminate pavement to a true LLAP.

8.2.1 Current Condition of LLAP

Figure 61 to Figure 71 following show the current condition of the Australian LLAP sites, with the exception of the Hume Motorway (St Andrews) which is not included as it was recently widened (2010) with an extra lane in each direction.

![Figure 61 H-S22 New England Freeway](image-url)
Figure 62 Pacific Motorway (N.B), Bar Point

Figure 63 Pacific Motorway (N.B), Mooney Mooney
Figure 64 Pacific Motorway Ourimbah, (SB)

Figure 65 Pacific Highway, (Maitland Road) (NB)
Figure 66 Pacific Highway Cumbalum (NB)

Figure 67 Picton Road, Avon (EB)
Figure 68 Botany Road, Port Botany, (NB)

Figure 69 Milpera Road, (WB), Condell Park
Figure 70 Bruce Highway (Q5), Kallangur (SB)

Figure 71 Bruce Highway (Q6), Kallangur (SB)
Figure 72 Camden By-Pass (SB)

Figure 73 Alpha Street Blacktown (EB)
If the current condition of the pavements is compared against the definition of a LLAP used in this study of ‘a pavement which, while damage might occur, the asphalt will never develop macro-cracking requiring deep structural treatment’, it can be seen from the figures all pavement sections, with the exception of the Pacific Highway (Maitland Road), have no evidence of structural cracking or macro cracking which would require structural treatment. While some pavement sections have an older oxidise surface and sections of the Sydney Newcastle freeway have a ravelling open grade no damage extends beyond the surface of the pavement. The Pacific Highway (Maitland Road) is now showing the early signs of cracking as can be seen in Figure 65, previously. While this site may have a long and indeterminate life it is not deemed a LLAP site and subsequently was not included in the LLAP analysis.

8.2.2 Indeterminate Structures

As mentioned, to validate the threshold limit between an indeterminate structures and LLAP pavements, a number of non LLAP sections are considered in this study. In reality more information is obtained from the determinate and indeterminate pavement structures than that of the LLAP, as in concept, the thickest pavement to experience any degree of fatigue failure can be used to establish the limit between the indeterminate structures and LLAPs.

The sections which are included in the analysis as being indeterminate were all sections of asphalt pavement greater than 30 years old and greater than 150mm of asphalt which had experienced any signs of fatigue cracking. In addition and any pavement which had greater than 200mm of combined asphalt thickness which had experienced fatigue cracking has been included with the indeterminate structures. These non-LLAP sections are found in Table 20 following.
### Table 20 Australian Non LLAP Pavement Validation Sites

<table>
<thead>
<tr>
<th>Site ID</th>
<th>Road/Location</th>
<th>Cumulative Traffic</th>
<th>Construction Year</th>
<th>Pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-S23</td>
<td>New England Highway</td>
<td>8x10^7</td>
<td>1970</td>
<td>220mm Asphalt 230mm Fine to course gravel 810mm Poorly graded sand S G</td>
</tr>
<tr>
<td>Sou-S086</td>
<td>Princess Highway, South of Gerringong</td>
<td>9x10^6</td>
<td>1969</td>
<td>200mm Asphalt 200mm Unbound gravel 300mm Gravelly clay S G</td>
</tr>
<tr>
<td>Syd-S26</td>
<td>Bunnerong Rd. Matraville</td>
<td>1.4x10^7</td>
<td>1960</td>
<td>160mm Asphalt 200mm Poorly graded gravel 300mm Poorly graded sand S G</td>
</tr>
<tr>
<td>Syd-S23</td>
<td>The Grand Parade, Monterey</td>
<td>4.5x10^6</td>
<td>2000</td>
<td>200mm Asphalt 150mm Poorly graded gravel 550mm Poorly graded sand S G</td>
</tr>
<tr>
<td>Syd-S46</td>
<td>Rocky Point Road, Beverly Park</td>
<td>1.6x10^7</td>
<td>1966</td>
<td>160mm Asphalt 240mm Poorly graded gravel 550mm Poorly graded sand S G</td>
</tr>
<tr>
<td>H-S39</td>
<td>Sydney Newcastle Freeway</td>
<td>8.9x10^7</td>
<td>1970</td>
<td>155-165mm Asphalt 440mm Fine to course gravel Clay</td>
</tr>
<tr>
<td>H-S24</td>
<td>Sydney Newcastle Freeway</td>
<td>8.9x10^7</td>
<td>1970</td>
<td>240-260mm Asphalt 220-330mm Fine to course gravel</td>
</tr>
<tr>
<td>Syd-S24</td>
<td>Woodville Road, Merrylands</td>
<td>3.7x10^7</td>
<td>1970</td>
<td>190-180mm Asphalt 300mm Fine to course gravel (some poorly graded) Clay</td>
</tr>
<tr>
<td>Syd-S25</td>
<td>Woodville Road, Guilford</td>
<td>3.7x10^7</td>
<td>1970</td>
<td>150mm Asphalt 200-450mm Poorly graded gravel</td>
</tr>
<tr>
<td>Syd-S33</td>
<td>Bunnerong Road, Matraville</td>
<td>1.2x10^7</td>
<td>1960</td>
<td>150mm Asphalt 320mm Fine to course gravel Poorly graded sand</td>
</tr>
<tr>
<td>Sou-S089</td>
<td>Princess Highway, Conjola</td>
<td>1.2x10^6</td>
<td>1985</td>
<td>240mm Asphalt 120mm Quartz gravel Sandy Gravel</td>
</tr>
</tbody>
</table>

2 It is known that stripping of asphalt has occurred with the mixes placed at in these locations. However, as no stripping was observed in the cores, for conservatism this site has been included in the fatigue susceptible pavements.

The analysis of the database of non-LLAP again supports the finding obtained from LLAP studies undertaken internationally that a limiting thickness of around 280mm exists for asphalt pavements, regardless of support to the asphalt layer, with the thickest section of asphalt pavement to experience fatigue cracking being 260mm.
8.2.3 Condition Assessment (FWD)

For each of the sites in the RMS STEP database full Falling Weight Deflectometer (FWD) survey results were available, in terms of the:

- Full load deflation bowls,
- Surface temperature
- Air Temperature Data

All FWD testing undertaken as part of the STEP database development was conducted using a standard approach and in all cases the pressure on the load plate was targeted at 700kPa. The use of a standard 700kPa load pressure is convenient as it equates to a load of 49kN. The advantage of the 49kN is that the loading is close to that of the recommended half axle loading of 45kN, meaning that the effect on non-linearity on granular materials will be little to nothing.

The results of the FWD survey can be seen in Figure 75 and Figure 76 following which show the full deflection bowls for the LLAP sites and the non LLAP sites respectively.
Figure 76 Deflection Bowls Non LLAP

What the figures show is that the overall deflection ranges of the two sets of data are close to identical. The implication of this, as expected, is that low or high deflection alone cannot be used to define a LLAP and as found with the calibration undertaken on the NCAT test track, strain appears to be much more significant in predicting the occurrence of LLAP.

8.2.4 Mid Layer Depth

Additionally, the database included the mid layer asphalt temperature of the combined asphalt layers. The mid layer temperature was calculated using the surface temperature measured in the FWD testing and the modified Bells equation developed by Roberts et al. (2010) and as shown in Equation 42 following.

$$
T_{m} = T_{s} \times \left( 0.77 + 0.649 \times T_{s} + 0.044 \times T_{s} \right) \times \left( 2.24 \times h - 1.42 \right) + \left( 4.79 \times T_{s} \right)
$$

Equation 42

Where;
MMAT = Mean Month Air Temperature, for the month of testing

\( T_o \) = surface temperature at time of day

hr = time of test on 24 hour (decimal) clock. (eg: 14.33 = 2.20 pm)

\( h_i \) = Combined asphalt layer thicknesses (mm)

\( T_2 \) = temperature at mid-point in pavement layer

\( \text{AAF} \) = Adjustment Factor, \( = 0.0175 \times \text{MMAT} + 0.6773 \). 

While it is understood that ARRB are currently undertaking further studies on the improvement of the prediction of pavement temperature with depth equations for Australian condition, at this point there appeared to be no advantage in using an alternative approach to the method developed by Roberts and as such the ARRB modified Bells equation was used in the analysis.

If an alternative approach emanating from the ARRB studies is developed and recommended in the future, it would be recommended that the calibration exercise be re-undertaken to ensure the linkage with actual field performance.

8.3 Insitu Material Assessment

While some may question the accuracy of back-calculation methods, back-calculation remains the most accepted and practical method to assess the insitu stiffness of pavement layers. Most significantly, for the sites used in this analysis the FWD testing was undertaken at the location of each test-pit, meaning exact layer thicknesses and material type were known. This significantly reduces potential inaccuracies in the back-calculation.

At present there are three main back-calculation methods that are widely used. The three methods are primarily based on the forward calculation procedure. In a study for the Texas DOT Uzan (1994) analysed several existing back-calculation procedures and concluded that the main differences among all procedures are related to the forward calculation model used to predict the pavement response as well as the error minimization scheme utilised. The forward calculation schemes investigated included, numerical integration (Linear Elastic (LE) methods, matrix based Finite Element (FE) methods and approximation methods (Method of Equivalent Thicknesses (MET)). He found that although the approximation methods are faster, in some cases they may lead to unacceptable errors in the forward calculation that in turn would lead to errors being reflected in the computed modulus values.

However, by far the most important parameters to ensuring correct results from the back-calculation process found by numerous researchers, is to incorporate the right set of rules to define the pavement system (layer ratios, maximum and minimum modulus value etc.). These ‘rules’ have been found to be rather more important than the method used in the forward calculation and the method used to determine the minimum error. This was reinforced by Appea et al. (2002) who found that the ‘considerations such as the allowable ranges and seed values appeared to have more impact on the back-calculated moduli than the software package used.’
8.3.1 Layer Elastic Analysis

The best overall solution at the present time is to use the numerical integration methods based on linear elastic solutions. It was Burmister who provided the first theoretical solutions for a system of two or more elastic pavement layers, using a series of Bessel functions. In 1962 Schiffman expanded this knowledge and provided solutions for an n-layered pavement system. The solution found by Schiffman, led to a series of computer programs. All of these programs developed based on Schiffman's solution compute stresses and strains based on the following assumptions.

- Surface load is uniformly distributed over a circular area
- All layers are homogeneous, isotropic (except CIRCLY) and linear elastic
- Upper layers extended horizontally to infinity
- The bottom layer is a semi-infinite half space

An iterative procedure is then used to determine those modulus values that result in the same deflections as measured.

8.3.2 Bedrock

If the subgrade is assumed to be a semi-infinite half space, as in the proposed LE methods, the effect of a stiff layer, or bedrock layer, at a shallow depth can be quite significant. The failure to consider the stiff layer can cause erroneous results in the upper layers. If a stiff layer is to be considered in back-calculation the question must be asked how deep is infinity? Or to put it another way, when a should a stiff layer be considered? Irwin (1994) found that the stiff layer has little to no effect on the back-calculated modulus values when the layer is deeper than 12m, other researchers, such as Ullditz (1987) have found that a level of 5m may be appropriate. Whichever is the case, it is clear that many pavement structures may be affected by the presence of a bedrock layer and the bedrock layers should be considered not only in back calculation but forward calculations as well.

There are two main methods recommended to calculate the depth to the stiff layer in a back-calculation procedure. By far the most widely accepted is the method used in WDOT design manual. In this method the measured deflection is plotted against the reciprocal of the distance from the load ($1/r$). In this analysis if $1/r$ has an intercept that is not equal to zero, it indicates the presence of a stiff layer at shallow depth. This is based on Boussinesq's equations as shown in § 4.3. If the pavement is, in fact, a ½ space the deflection will only be zero when the distance from the load is infinite i.e. $1/r = 0$. Using this approach an assumption is then made, which has been found to be fairly accurate that the radial distance to the point where the deflection is zero is equal to the depth of the stiff layer i.e. the surface deflection is equal to the horizontal deflection at depth. This approach is shown in Figure 77, following.
8.3.3 Assessing the Answers from Back-Calculation

The most popular method at present to assess the accuracy of back-calculation results is to use the Root-Mean-Squared (RMS) of the error between the back-calculated results and that of the forward calculated. What a small RMS error (<2%) indicates is that there is a good match between the measured and calculated deflection bowl. However it does not insure the back-calculated modulus are correct. As indicated by Irwin (2005) The best way to overcome the problems and assess the validity of the back calculation results is to have a thorough understanding of the materials in the pavement. This is where the results for 8.2 become increasing important and the rules determined based on these fundamental material properties. To ensure correct answers are achieved not just close matches to the deflection bowl.

8.3.4 Implications on Forward Calculations

Probably of greater importance in determining the true response of a pavement system to applied loading, than the set of rules developed in the previous sections for the determination of the
pavement's response to loading, is ensuring that the assumptions made in the back-calculation phase match the assumptions made in the forward calculations. It is of no use going to all the effort to determine the non-linear pavement material responses if the forward calculation assumes all materials are linear elastic, as with the depth to bedrock. Therefore for practical implementation of any design procedure, there has to be a link established between the back-calculation procedure and the forward calculation procedure.

8.3.5 Pavement Model for Back-calculation

The model for the back calculation analysis used a 5 layer system with a varying depth to bedrock and is shown following.

- Asphalt thickness was determined from the combined asphalt layers thicknesses, from geotechnical investigation.
- Base layer thickness was determined from the geotechnical investigation; thin base course layers (<150mm) were combined with lower layers.
- Sub-base, if any, was the combined sub-base layers from the geotechnical investigation. Where no sub-base existed, the upper subgrade (500mm) was modelled as a separate layer.
- Subgrade thickness was determined from intercept of inverse deflection plot or limited to a maximum 5000mm layer.
- A bedrock layer existed in all pavements with a stiffness of 5000MPa. The bedrock layer was infinite with depth.
- The approach used for back-calculation was based off a deflection basin fitting approach with the objective function of minimising RMS error.

The pavement model used for each site along with the back-calculated modulus is shown in Table 18, following for both the LLAP sites and the damaged non-LLAP pavement sites.

<table>
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<th>ID</th>
<th>Temp (°C)</th>
<th>Thickness (mm)</th>
<th>Modulus (MPa)</th>
<th>RM S</th>
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**Non-LLAP Sites**

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</tbody>
</table>

FWD results indicate damaged asphalt, therefore section H-S28 was excluded from being a LLAP.

The results show the difference between the measured and calculated deflection bowls was typically less than 1% indicating a high confidence in the back-calculation results. These results are lower than what is typically achieved by back-calculation. This however is due to; the exact material profiles being known at the point of the back-calculation, the incorporation of the bedrock layer, and a higher degree of precession being used in the back-calculation process.

The results show that the subgrade support values found from the back-calculation are somewhat higher than what would be typical on new pavement design with values ranging from 60-150MPa found. However, these values are insitu, which are typically higher what would be expected from soaked ex-situ tests. Therefore, if the model is validated on insitu tests and ex-situ soaked test are used for the purpose of pavement design a degree of conservatism will be automatically built into the design.

Empirical LLAP design procedures such as that proposed by Nunn et al. (2001) place a high importance on determination of subgrade support for LLAP design. However the framework of both the AGPT002 (2012) and the APS-fL project is mechanistic-empirical procedure and as such places a higher emphasis on total support to the asphalt layer through the use of induced strain over subgrade support only. The approach has advantages in that it is not only a measure of the variation in subgrade support but the support additionally offered by any base or sub-base layers. In this way subgrade support and variation in subgrade support is covered in the calculation of strain under the design vehicle. This was shown in the examination of the deflection bowls and confirmed by the NCAT test track results, which clearly showed that strain is directly related to asphalt performance.

### 8.4 Validation of Stiffness-FE L Relationship with Australian Data

To validate the NCAT findings to Australian conditions the APS-fL project examined the sites identified as LLAP and Non-LLAP from the RMS STEP database. The sections identified as LLAP, as shown in Table 19, were sites that were greater than 20 years old, had experienced in excess of $3 \times 10^7$ ESA loading and had no cracking. Additionally, to undertake the validation a number of thick (>140mm) Non-LLAP sites as shown in Table 20, which although had significant traffic loading, were experiencing some form of structural failure.
It was believed that by using both the sites which had undergone failure and the LLAP sites the stiffness-FEL relationship could be validated between real life Australian LLAP and fatigue susceptible pavements.

For each site in the RMS STEP database, FWD data, a full geo-tech investigation, maintenance history and condition assessment was available. The full details of the sites, material type, back calculated modulus and layer thickness can be found in Appendix B and E for the back calculation results.

For each of these sites the stiffness-strain relationship was established by:

1. Determining the supporting layer modulus values from back-calculation of the deflection bowl as described previously and shown in Table 21.
2. The mid-layer asphalt temperature was calculated for six seasons: summer, autumn, winter, and spring. Additionally, the mid layer temperature was calculated for the upper 10th percentile summer temperature and lower 10th percentilie winter temperature using the calibrated Bells equation procedure developed by Roberts (2009).
3. The vehicle speed for the heavy vehicle was taken as 10km lower that of the posted speed limit.
4. The design axle loading was taken as the upper 97.5th percentile axle loading in Australia of 9.0 tonnes.
5. The effective modulus was calculated based on 40mm AC14C320 asphalt and the remaining asphalt being an AC20 C320 layer, typical of asphalt used to construct the pavements.
6. For each season, the horizontal strain at the base of the combined asphalt layer was calculated using Layered Elastic analysis. (NB to be consistent with back-calculation all layers were isotropic)

Once the strains had been calculated, the relationship between stiffness and strain for each of the Australian validation sites was plotted as shown in Figure 78 following, which summarises the results of the analysis for the Australian validation sites. Detailed results can be found in Appendix F. To add to the data and the overall understanding of the performance of LLAP In addition to the Australian sites, the analysis included the results obtained from that of the structural sections on the NCAT test track and three sites from the UK Valmon data, which were the closest to the FEL and Australian LLAP. The results of the FWD analysis on the Colgreen, Brentwood and Thornbury Valmon LLAP can be found in Appendix E of this report. Figure 78 following shows the results obtained from the Australian LLAP and non-LLAP, the structural sections on the NCAT test track and the results obtained from the three Valmon sites, as well as the calibrated relationship obtained from analysis of the Phase II NCAT test track structural sections.
As can be seen in the figure and as was found with the NCAT test sections, a clear differentiation exists in the stiffness-strain relationship between those sections which are; determinate, non-determinate and LLAP. As with the findings on the NCAT test track, the sections which showed signs of failure had strain levels across the whole range of stiffness values higher than the non-cracked sections, clearly showing structural failure again is a function of induced strains, which is a combination of both the subgrade and sub-base support.

The analysis shows that the FEL relationship determined from the NCAT test track appears to hold true for some but not all of the Australian sites. The NCAT level appears to be around a 50% confidence level and while most pavements designed at this FEL will be LLAP, this will not always be the case and some pavement, which may have a long life but will be indeterminate in nature. The results show that a number of sites, which have signs of failure, particularly the Sydney Newcastle Freeway (H-S39) at Mooney-Mooney fall below the proposed NCAT endurance limit. The results also show that all of the UK Valmon LLAP sites fall below that of the NCAT limit.

The analysis shows that the modelling of the six seasons appears not to be required with 3 critical locations possible, summer, winter and a mid-point, if all these points fall below that of the FEL all points throughout the year will also. Given this, it is recommend that the design approach moving forward only use, the upper simmer 95th percentile temperature, winter lower 95th percentile and the midpoint determined as the WMAPT.
The visual inspection of the results shows that a limit appears to exist between the Sydney Newcastle Freeway (H-S 39) site and the Valmon and New England Highway site (H-S 27_18). While the NCAT relationship may be practical to determine a pavement with a long and indeterminate life, it will not ensure that the pavement is a LLAP. Both the Australian and UK field validation data suggest a more conservative FEL is required.

This observation of the data thus suggests that the standard FEL curve needs to be shifted to the deviation between the Sydney Newcastle Freeway section (H-S 39) and the Valmon and the New England Highway site (H-S 27_18), as shown in Figure 79 following.

![Figure 79 Validated Stiffness-Strain Endurance Curve Australian Sites](image)

The results show that the confidence and rest factor, k1, needs an adjustment factor of 0.9 to move from the standard curve to the field observations.

Using the results of the Australian validation sites H22, S15 and H23 and the Valmon sections, a revised validated FEL-stiffness relationship was established from field validation data, as shown in Equation 43 following.

\[ \text{Equation 43} \]

Where;

FEL is the Fatigue Endurance Limit at given modulus
\[ S_{\text{mix}} \] is the equivalent stiffness of the combined asphalt layers

This relationship offers a practical stiffness-FEL, which can be used for LLAP pavement design. At levels below the stiffness-FEL relationship, Australian pavements have not experienced cracking with in excess 8x10^7 ESA loadings and 30 years of service. At strain levels above this relationship, pavements appear to be fatigue susceptible with a majority but not all pavements experiencing damage. As the FEL relationship was developed from Australian mixes and NCAT mixes, the relationship is:

- Valid for mixes with conventional bitumen and low levels of SBS modification.
- Valid over a full range of temperatures experienced by asphalt pavements 0-45°C.
- Valid for both freeway and high volume urban areas.
- Not valid for mixes with high RAP contents (>30%) and its use with higher RAP contents is not recommended.
- Only valid for dense graded mixes and cannot be used with mixes with design air voids of greater than 5%.

8.5 Establishing Confidence Levels

While the relationship found from visual observation of the stiffness-FEL curve shown in Figure 79 will for most cases result in a true LLAP, there is no statistical rational or confidence behind the recommended level. While it may be argued that a FEL design method does not require a statistical confidence level, statistical confidence can be easily inserted into the method by using the \( k_1 \) adjustment constants to move the design curve up or down to cover more points (or a higher or lower confidence), as shown in Equation 44.

\[
FEL = S_{\text{mix}} \times k_1
\]

Equation 44

Where;

- \( FEL \) is the Fatigue Endurance Limit
- \( S_{\text{mix}} \) is the stiffness of the mix, and
- \( k_1 \) is the adjustment constants for differences in rest periods, or confidence levels
- \( k_2 \) is the mix adjustment factor

This approach moves the standard equation up or down to account for changes in potential of the healing of the mix to account for different binders and rest periods or in this case confidence levels incorporating all of these variables.

To establish confidence limits the predicted strain for each of the LLAP section which fell between fell the section with the lowest strain-stiffness relationship, which had shown signs of failure, (the Sydney Newcastle Freeway section H-S 39) and the section with highest stiffness-strain relationship
which showed no signs of failure (H-S29 and N4 as lines cross over) were analysed. The analysis fitted a power relationship to the stiffness strain curve for each of the 14 sections which fell in the indeterminate zone. These relationships were then used to predict the strain at stiffness values near the upper summer temperature (1,500MPa), the WMAPT (3,000MPa) and winter lower (6,500MPa). For each of the stiffness values the strains were then ranked in order of highest to lowest and the strains were then plotted against the inverse standard normal distribution (normal (z) score) as shown in Figure 80 following.

As seen in the figure showing the resulting distributions for the three stiffness levels, the distribution can be seen to closely follow that of a normal distribution and for all practical purposes can be assumed to follow a normal distribution. Because the results follow that of a normal distribution, confidence levels can easily be developed by solving the line of best fit of the normal score against that of the predicted strain for the normal score(z) at a set confidence limit.

The AGPT002 (20120, uses 5 levels of project reliability 80, 85, 90, 95 and 97.5%. these reliability (probability) figures can be equated to a normal (z) score of the standard normal cumulative distribution (mean zero and standard deviation of 1) of 0.842, 1.036, 1.282, 1.644 and 1.960 respectively. This can be seen conceptually in Figure 80 where the 95th percentile value (1.64) can be seen to equate to a FEL of 150m for a stiffness of 1500MPa.

Using these confidence levels the standard FEL equation can then be adjusted by changing the confidence coefficient, $k_1$, to fit the predicted FEL level at the required confidence level. The
resulting fitting parameters and the confidence based FEL are shown in Table 22 following. While the resulting confidence limits for 80 and 95 percentile can be seen on Figure 81 following.

Table 22 FEL and Confidence Limits

<table>
<thead>
<tr>
<th>Confidence Limit</th>
<th>Normal Score (z)</th>
<th>FEL (MPa)</th>
<th>k&lt;sub&gt;2&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1500</td>
<td>3000</td>
</tr>
<tr>
<td>0.8</td>
<td>0.842</td>
<td>174</td>
<td>110</td>
</tr>
<tr>
<td>0.85</td>
<td>1.036</td>
<td>168</td>
<td>107</td>
</tr>
<tr>
<td>0.9</td>
<td>1.282</td>
<td>161</td>
<td>102</td>
</tr>
<tr>
<td>0.95</td>
<td>1.645</td>
<td>150</td>
<td>95</td>
</tr>
<tr>
<td>0.975</td>
<td>1.960</td>
<td>140</td>
<td>88</td>
</tr>
</tbody>
</table>

Figure 81 Confidence Limits and LLAP Sections

Using the recommended relationship, it can be found that on a subgrade of CBR 5 the 50% confidence level will give thicknesses in the range of 260-270mm and at the 95<sup>th</sup> percentile level thicknesses in the range of 330-340mm. These results compare very favourably with the recommendations of Rolt (2001) that a well-designed and constructed asphalt pavements with 270mm of thickness could provide a fatigue resistant pavement structure and Nunn (2001) that 280mm was sufficient to resist fatigue cracking. The additional thicknesses above these figures at higher confidence levels simply allow for increased confidence in design and possible deficiencies in materials and design.
8.6 Laboratory to Field Shift

Given that the confidence based shift factors are all a correction to the standard FEL relationship obtained from US and Australian mixes, the shift factor $k_1$ is simply a laboratory to field conversion. Given that this is a simple shift, confidence levels can easily be applied to mixes which do not fall on the standard FEL relationship, such as EME2 mix and A5E mixes. Simply the same confidence shift factors may be applied to the results of laboratory testing using the determined using the PV approach as found in this study.

8.7 Endurance limits for lower level traffic

While the stiffness-FEL establishes a simple tool for the design of LLAP, pavements which fall above this level can still be LLAP. However, in all cases the traffic loading would have to be less than the common freeway traffic loading experienced in Australia. Because of the lower traffic levels, greater amounts of healing will be possible allowing for thinner pavement sections.

To do this, the database would have to be expanded to cover LLAP without any damage with traffic levels less than $4x10^7$ ESA. The approach would again be to, simply move the confidence equation for the lower traffic levels.

8.8 Conclusions of the Validation of Modulus Based FEL

This finding obtained from the analysis of LLAP on the NCAT test track was extended to actual LLAP in Australia and the UK. The analysis confirmed the same finding as on the NCAT test track that a direct relationship exists between the infield stiffness-strain curve of LLAP and the laboratory determined stiffness-FEL curve. This direct relationship allows the development of a phenomenological stiffness-FEL relationship which can be used for the purpose of pavement design.

Based upon both the calibrated relationship from the NCAT data and validated Australian LLAP, the use of a stiffness-FEL relationship is recommended for the design of long-life asphalt pavements in Australia. The relationship was validated to Australian data and only required a slight modification to the relationship developed from the NCAT, as shown following.

\[
F_{EL} = \frac{S_{mix}}{k_1} + k_2
\]

Where;

- $F_{EL}$ is the Fatigue Endurance Limit
- $S_{mix}$ is the stiffness of the mix, and
- $k_1$ is the adjustment constant for differences in rest periods, or confidence levels
- $k_2$ is the mix adjustment factor
It was found that confidence limits could be incorporated into the equation by the use of the \( k_1 \) adjustment factor and confidence limits could be determined from the Australian validation sites as shown following.

<table>
<thead>
<tr>
<th>Confidence Level</th>
<th>80(^{th})</th>
<th>85(^{th})</th>
<th>90(^{th})</th>
<th>95(^{th})</th>
<th>97.5(^{th})</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_1 )</td>
<td>1</td>
<td>0.97</td>
<td>0.925</td>
<td>0.86</td>
<td>0.8</td>
</tr>
</tbody>
</table>

The analysis of the results showed that the design of LLAP can be supported by a fundamental test using the RDEC and the constant \( PV_L \). However, a laboratory to field conversion is needed to use the test which is equivalent to the confidence values shown previously.

The current design recommendation for Australia LLAP design is to use:

- 9.0 design axle
- Model using 3 seasons; summer, median (WMAPT) and winter.
- Mid layer asphalt temperatures be taken from the ARRB modified Bells equation
- By using this approach the limiting thickness of asphalt pavements can be obtained.
9 Austroads Supplement Recommendations

9.1 Introduction

This chapter sets out the recommended format for a Supplement to the AGPT02 (2012) for LLAP design. The procedure is based on the recommendations found in this report and the design procedure used in the AGPT02. Where possible the procedure has attempted to follow the recommendation of the AGPT and incorporate the required additional steps for LLAP design.

The intent of the AAPA LLAP design procedure is to determine the maximum tensile strain(s) at which, whilst damage might occur, the asphalt will never develop macro-cracking requiring deep structural treatment. As the proposed procedure has been developed and validated for freeways, highways and urban arterial roads, this procedure is relevant to all those pavement types. The recommendation in this chapter should be read in conjunction with AGPT02 (Austroads, 2012).

9.1.1 Granular Material Characterisation

The use of cross anisotropy in the characterisation in the subgrade and working platform results in an increase in strain of 7% over that of the isotopic model. While the use of cross anisotropy is conservative, to ensure the thicknesses requirements remain consistent with the validated approach a correction of 7% may be applied to Equation 40 to account for cross isotropy. This correction has been applied to the recommendations of this chapter.

9.2 (3) Construction and Maintenance Considerations

9.2.1 (3.2.5) Working Platforms

As compaction of asphalt layers on subgrades with stiffness of less than 100MPa (CBR 10%), is difficult, for a subgrade with a stiffness of less than a 100MPa, it is recommended that a working platform be established to achieve a supporting layer of sufficient quality to be assigned a design modulus of 100MPa (or greater) in CIR CLY should be placed directly below the bottom asphalt layer to support this bottom layer and assist with achieving compaction.

9.2.2 (3.7) Pavement Layering Considerations

The LLAP concept utilises a three layered asphalt system;

1) A wearing course and levelling course (surfacing layers), which should be durable and rut resistant
2) intermediate layer(s), which should be rut resistant and be the main structural layers
3) fatigue layer, which should be a fatigue resistant low air void mix

The high rut resistant durable surface layers may consist of dense grade asphalt (It is noted that many SRAs do not utilise DGA as a wearing course on freeways due to texture requirements.) or
a SMA, typically a maximum size of 14 mm. Where an OGA surfacing is proposed for noise and/or drainage considerations, an additional surface layer consisting of a 14mm DG asphalt shall be provided immediately below the OGA surface. For durability and rut resistance, the asphalt in the surfacing layer(s) may incorporate a modified binder, if so an A35P or Multigrade is recommended over elastomeric binders such as A10 or A15E as A35P offer more structural capacity to the pavement.

The intermediate asphalt layers(s) should consist of conventional (C320, 450 or 600) asphalt DG20 mix which is durable and rut resistant suitable for the climatic region.

It is recommended that the LLAP incorporate a low air void (<4%) asphalt fatigue layer, in the bottom 60mm of the pavement. The low void layer is usually achieved by the use of higher binder content in the bottom layer of the DG 20 asphalt mix. This lower level asphalt layer should consist of no more than 30%RAP, if more than 30% RAP it is recommended that the stiffness-FEL relationship of the mix be confirmed to fall on or above the standard stiffness-FEL mix curve.

9.2.3 (3.12) Maintenance Strategy

LLAP are designed to eliminate fatigue cracking therefore no full/partial depth repairs are expected due to structural failure of the pavement for an extended pavement life of greater than 40 years.

In LLAP all maintenance is expected to be top down resulting from oxidation and/or surface initiated cracking, or non-structural environmental cracking, therefore maintenance will be limited to periodic overlays or thin mill and re-sheets.

9.3 (4) Environment

9.3.1 (4.3) Temperature

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

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

The WMAPT is given in Appendix B of the AGPT02 the procedure for calculating the effective layer temperature for the combined asphalt layers is given in Section 9.3.2.
9.3.2 (4.3.1) Calculation of Effective Layer Temperature

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

\[
T_{2} = T_{o} + AAF \times 8.77 + 0.649 \times T_{a} + (2.2 + 0.044 \times T_{a}) \times h_{i} - 1.24 \times h_{i}^{100} \times 17 - 0.503 \times T_{a} + 0.786 \times 4.79 \times h_{i}^{2} - 1.824 \times h_{i}^{3}
\]  
Equation 45

Where;

\[\begin{align*}
MMAT & = \text{Mean Month Air Temperature, for the month of testing (°C)} \\
T_{o} & = \text{surface temperature at time of day (°C)} \\
h_{r} & = \text{time of test on 24 hour (decimal) clock. (e.g.: 14.33 = 2.20 pm)} \\
h_{i} & = \text{Combined asphalt layer thicknesses (mm)} \\
T_{2} & = \text{temperature at mid-point in pavement layer (°C)} \\
AAF & = \text{Australian Adjustment Factor, } = 0.0175 \times \text{MMAT} + 0.6773.
\end{align*}\]

The advantage of this equation is that it uses readily available monthly statistical data from the Bureau of Meteorology, which is available at:


A Visual Basic (VBA) function which can be used in MS Excel can be found in Appendix G, to assist in determining \(T_{2}\).

9.3.3 (4.3.1.1.) Surface Temperature (Summer)

The upper 95\(^{th}\) percentile surface temperature (\(T_{o}\)) of the pavement can be estimated by using the average maximum temperature of the hottest summer month and the simplification of the radiation balance approach, shown in Equation 46 following.

\[
T_{o} = T_{a} + 25.5 \left(10^{-0.1T_{a}} - 1\right)
\]  
Equation 46

Where

\[\begin{align*}
T_{a} & = \text{Air Temperature (°C)} \\
T_{o} & = \text{Surface Temperature (°C)} \\
z & = \text{Zenith angle}
\end{align*}\]

The Zenith angle is given by:

\[z = \text{Latitude}^\circ - 23.3^\circ, \text{below the tropic of Capricorn, and}\]
z = 0 for locations above the tropic of Capricorn.

For calculation of upper the summer temperature ($T_0$) time should be taken as 1pm i.e. 13.0 in the modified Bells equation.

9.3.4 (4.3.1.1.) Lower Surface Temperature (Winter)

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

$$ T_o = T_a $$

For calculation of the lower surface winter temperature ($T_0$) time should be taken as 6am i.e. 6.0 in the modified Bells equation.

9.3.5 (4.3.1.1.) Effective Pavement Temperature

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

9.4 (5) Subgrade Evaluation

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

9.4.1 (5.1) Measures of Subgrade Support

For LLAP design the subgrade support shall be characterised in terms of the stiffness (resilient modulus ($M_r$)). It is recommended that the stiffness of the subgrade be indirectly determined from CBR of the subgrade determined from;

- in-situ DCP Testing, converted to CBR in accordance with AGPT02,
- results of soaked CBR tests, or,
- Appreciation of the subgrade soil type

The CBR shall then be converted to stiffness in accordance with AGPT02 (10xCBR).

Alternatively subgrade support may be directly determined via tri-axial testing or determined from back calculation of FWD data.

For stiffness determined from FWD testing, it is essential to use the same analysis method in back and forward calculation. It is recommended that the back calculated subgrade modulus be validated by comparing the back calculated results to insitu CBR determined indirectly from DCP testing in accordance with Section 5.5 of the AGPT02.
9.4.2  (5.9) Subgrade Failure Criteria

Research by Nunn et al. (2001) has shown that for asphalt pavements with asphalt thickness greater than 150mm, rutting is confined to the asphalt layer(s). However, to ensure adequate cover over the subgrade a minimum number of allowable repetitions of \(1 \times 10^8\) shall be applied, or 650m, for analysis conducted at the WMAPT.

9.5  (6.4) Asphalt

Because the APS-FL project requires the determination of the stiffness of the asphalt mix at the extremes of temperature, the use of section 6 of the AGPT02 is not recommended, as this procedure has only been developed for a limited range of WMAPT temperatures and one asphalt thickness. Additionally, the validation and calibration of the FEL is based on the inter-conversion found in this report and any deviation from this approach will lessen the accuracy of the approach.

9.5.1  (6.4.1.3) Characteristics for Design

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

- Indirect Tension on Cylindrical (IT-CY) samples (AS 2891.13.1)
- 4 Point Bending on Prismatic Beams (4PB-PB) samples (AG:PT/T233)
- Direct Compression on Cylindrical (DC-CY) samples (ASHTO TP 62 or AASHTO TP 79)

All of these test methods have a different definition of time and different stress states.

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

- The (IT-CY) should be considered a haversine pulse load with time equivalent to double the rise time.
- The (4PB-PB) Flexural modulus test should be considered a cyclic frequency load with a haversine load pulse of \(\frac{1}{2}\) the full load pulse width.
- The (DC-CY) dynamic compressive modulus test should be considered as cyclic harmonic frequency with load pulse equal to the radial pulse time.

The mathematical conversion of these three time definitions is given in section 9.5.8.

At extreme temperatures and slow vehicle speeds there can be a difference between the modulus of asphalt in tension and compression due to the stress susceptibility of asphalt. Under normal Australian operating conditions this difference is negligible. However, under extreme conditions (>40°C) stress susceptibility can exhibit an influence on the stiffness of asphalt mixes. In LLAP design there is a net confining stress on the pavement.
Due to this confining effect it is not recommended that asphalt stiffness determined from tension or pure flexural tests be used for modelling at temperatures exceeding 40°C. For these extreme conditions, it is recommended that modulus be determined based on the results of the dynamic compressive modulus test undertaken at a confinement of 200kPa.

9.5.2 (6.4.2) Factors Affecting Modulus and Poisson’s Ratio

While, it is known that Poisson’s ratio varies as a function of asphalt mix type and temperature, the development and calibration of the design procedure used a constant Poisson’s ratio for asphalt of 0.35. Therefore it is recommended that this value be used for all design calculations.

9.5.3 (6.4.2.1) Binder Class and Content

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

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

9.5.4 (6.2.2.2) Air Voids

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

9.5.5 (6.2.2.3) Aggregates

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

9.5.6 (6.2.2.4) Temperature

Temperature has a significant effect on the stiffness of Australian asphalt mixes. Under typical operating temperatures experienced in Australia modulus of Australian mixes can vary between 1,000MPa to 25,000MPa.
Therefore the effective temperature of the asphalt for the design season must be taken into account during pavement design.

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

\[
\frac{1}{e^{a_T (T-25)}}
\]

(Equation 47)

Where:

- \(T\) = pavement design or testing temperature (°C)
- \(a\) and \(b\) = Fitting coefficients of the polynomial equation (refer Table 24)

9.5.7 (6.4.2.2) Rate of Loading (Time)

The effect of traffic speed on the stiffness of the asphalt is significant, especially between urban and freeway conditions. To determine the stiffness at a given loading speed the first step is to convert the loading speed to a time of loading.

The time of loading has been found to be directly related to the strain load pulse time resulting from vehicle loading. For design it should be assumed that the vehicle acts as a haversine pulse with a 1.8m wave length. The equivalent haversine loading time may be estimated using the Equation 48 following.

\[
\frac{t_{hp}}{v} = 6.48
\]

(Equation 48)

Where:

- \(t_{hp}\) = load duration (seconds)
- \(v\) = speed of traffic (km/hr.)

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

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

\[
\frac{t_{hp}}{v} \cdot \frac{1}{e^{a_T (T_{ht}-25)}}
\]

(Equation 49)

Where:

- \(\dot{\varepsilon}\) = temperature shift factor
- \(T_{ht}\) = the reduced load pulse time at the design temperature (seconds)
9.5.8 (6.4.3.2) Laboratory Measurement

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

For consistency and to aid in interpretation it is recommended that master curves only be presented in the equivalent haversine pulse time \( t_{hp} \) space and be represented by a sigmoidal function as shown in Equation 50 following.

\[
\log (|t - t_{ref}|) = \frac{a}{a + b} + \frac{1}{(g + d)(1 + e^{t_{hp(r)}})}
\]

Equation 50

Where:

- \( t_{hp(r)} \): reduced haversine pulse load at the reference temperature (seconds)
- \( a \): the minimum value of the mix stiffness
- \( a + b \): the maximum mix stiffness
- \( g, d \): shape fitting parameters, determined through numerical optimisation of experimental data

As part of the APS-fL project, it was found that there is no difference between the dynamic modulus determined from AASHTO TP62 or AASHTO TP79 and either method can be used to determine the dynamic modulus of the mix.

As the definition of time in other modulus test is different from that in the dynamic modulus tests, a frequency conversion will need to be undertaken if IT-CY, 4PB-PB, or DC-CY testing is proposed to determine the mix modulus.

It has been found that the following frequency conversions give comparable results to the results obtained from dynamic modulus testing (DC-CY).

Resilient Modulus, IT-CY

\[
\frac{f_{Rm}}{f_{Rm'}} = \frac{a}{a + b}
\]

Equation 51

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

\[
\frac{f_{Rm}}{f_{Rm'}} = \frac{1}{2}
\]

Equation 52

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

\[
\frac{f_{Rm}}{f_{Rm'}} = \frac{1}{2}
\]

Equation 53

Where;
9.5.9 (6.4.3.3) Typical Charts

If the exact mix to be used in the pavement is not known at the time of design (which is typically the case) the typical master curves for Australian mixes (Figure 42 to Figure 47) should be used to estimate the dynamic modulus of the mix as a function of binder class and nominal aggregate size. The information required as an input for the master curves is:

- Vehicle speed, where speed is in km/hr.
- Effective temperature (°C) of the asphalt

Alternatively, the stiffness can be determined using the standard temperature shift factors and master curve fitting parameters for the time temperature shift factors shown in Table 24 following and the sigmoidal master curve fitting parameters, shown in Table 25 following. The rate of loading is the equivalent haversine pulse loading time.

### Table 24 Temperature Shift Factors

<table>
<thead>
<tr>
<th></th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Binders</td>
<td>-0.001</td>
<td>0.116</td>
</tr>
</tbody>
</table>

### Table 25 Master Curve Fitting Parameters

<table>
<thead>
<tr>
<th>Mix</th>
<th>a</th>
<th>b</th>
<th>g</th>
<th>d</th>
</tr>
</thead>
<tbody>
<tr>
<td>DG14-C320</td>
<td>2.379</td>
<td>1.878</td>
<td>0.043</td>
<td>0.706</td>
</tr>
<tr>
<td>DG14-C450</td>
<td>2.357</td>
<td>1.860</td>
<td>-0.023</td>
<td>0.735</td>
</tr>
<tr>
<td>DG20-C320</td>
<td>2.569</td>
<td>1.715</td>
<td>0.157</td>
<td>0.818</td>
</tr>
<tr>
<td>DG20-C450</td>
<td>2.005</td>
<td>2.328</td>
<td>-0.454</td>
<td>0.647</td>
</tr>
<tr>
<td>DG20-C600</td>
<td>1.985</td>
<td>2.363</td>
<td>-0.465</td>
<td>0.658</td>
</tr>
</tbody>
</table>

9.5.10 (6.4.5) Suggested Fatigue Endurance Limit

The FEL developed from calibration on full scale test tracks and validated against actual LLAP in Australia is shown by the general relationship shown in Equation 54 following. This relationship determines the maximum tensile strain where damage may occur, but macro cracking will not form, and is given by:

\[
\varepsilon = \varepsilon_0 - C \theta + \frac{5}{2} (1 - C) \frac{D}{L} \theta^2
\]

Equation 54
Where;

\[ F\text{EL} \] is the Fatigue Endurance Limit

\( S_{\text{mix}} \) is the stiffness of the mix, and

\( k_1 \) is the adjustment constants for differences in rest periods, or confidence levels, given in Table 26 following.

\( k_2 \) is the mix adjustment factor (0 for conventional mixes)

**Table 26 Recommended Confidence Limits**

<table>
<thead>
<tr>
<th>Confidence Level</th>
<th>80(^{\text{th}})</th>
<th>85(^{\text{th}})</th>
<th>90(^{\text{th}})</th>
<th>95(^{\text{th}})</th>
<th>97.5(^{\text{th}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_1 )</td>
<td>1</td>
<td>0.97</td>
<td>0.925</td>
<td>0.86</td>
<td>0.8</td>
</tr>
</tbody>
</table>

### 9.6 (7) Design Traffic

The intent of this design procedure is to design a pavement where the healing potential of the asphalt mix becomes greater than the damage inflicted. Under this scenario the cumulative axle loading is immaterial, as the healing potential of the asphalt exceeds the damage caused by vehicle loading.

Research by Thompson et al. (2006) has shown that asphalt can withstand sporadic overloads and return to endurance limit performance. Therefore the critical vehicle used for LLAP design should be taken as the upper 97.5\(^{\text{th}}\) percentile axle load. For the majority of Australian pavements this will equate to a standard axle loaded to 9 tonnes.

If the design axle is taken as the Standard Axle of Section 8 of the AGPT02, then Equation 54 shall be scaled to account for the reduced loading applied in design, with the scaling shown in Equation 55 following.

\[ \text{Equation 55} \]

Where \( U_i \) is the upper 97.5\(^{\text{th}}\) percentile load, usually 9 tonnes

### 9.7 (8) Design of LLAP

#### 9.7.1 Determination of Basic Thickness

Figure 82 LLAP Thickness Requirements following shows and example thicknesses for a LLAP using the described mechanistic procedure. The chart can be used to determine if the thickness determined from fatigue analysis exceeds the threshold thickness level and a LLAP design is warranted also the chart may be used as starting point for LLAP design.
This chart has been developed using the mechanistic procedure and standard asphalt materials for each SRA. The use of alternative materials may result in different thickness requirements. Before using this chart the designers should ensure that materials are appropriate for the situation for which they are used.

9.7.2 (8.1) Mechanistic Procedure

In summary the procedure consists of:

1) Granular materials are considered to be isotropic or cross-isotropic, depending on the characterisation method used.
2) Asphalt materials are considered to be isotropic.
3) The visco-elastic properties of asphalt are considered by using vehicle speed and effective layer temperature.
4) Response to loading is calculated by linear elastic theory.
5) Critical responses are assessed as:
   a. Tensile strain at the bottom of the asphalt layers.
   b. Vertical compressive strain on subgrade.
6) Axle loading consisting of a single axle with dual tyres (SADT)
   a. with a load of 9.0t, with the use of Equation 54.
   b. with a load of 8.2t, with the use of Equation 55.
7) Tyre contact stress is assumed to be 750kPa.
8) 3 Season are modelled
   a. Morning loading in winter
   b. WMAPT
   c. Day time loading in summer

9.7.3 (8.2.3) Combined Asphalt Layer Modulus

It has been found that the use of a single equivalent layer of asphalt provides results as accurate as multiple asphalt layers. The equivalent modulus of multiple asphalt layers can be determined from the use of the conservation of the moment of inertia approach, as shown in Equation 56 following:

\[
\frac{1}{E_{\text{eff}}} = \frac{1}{E_1} h_1 + \frac{1}{E_2} h_2 + \cdots + \frac{1}{E_n} h_n
\]

*Equation 56*

Where:

- \( E_{\text{eff}} \) = is the effective stiffness of the combined layers (MPa)
- \( h_i \) = The thickness of the \( i \)th layer (mm)
- \( E_i \) = The stiffness of the \( i \)th layer (MPa)
- \( n \) = Number of asphalt layers

Where the asphalt surface courses or fatigue layer uses a different binder from that of the base course, it is recommended that the asphalt layer(s) be not combined into a single layer.
Table 27 (8.1) Mechanistic Design Procedure

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

Design Parameters:

a) Location Brisbane Metro

1) Trial Pavement

- 50mm DG 14 C320
- xmm DG 20 C600, trail 215mm
- 60mm DG 20 C600 (low void)
- 200mm Sub-Base Gravel
- Subgrade 60MPa

a) Assume 215mm DG 20 C600 Layer

2) 95th lower Percentile Subgrade stiffness 60MPa.
   a. (Soaked CBR 6) (CHECK)

3) Working platform, 200mm sub-base gravel.
   a. Upper layer design modulus 182MPa

4) Determine the WMAPT for Brisbane = 31.9 AGPT-Part 2 Appendix C

5) From all available climate statistics for BOM site 040214:
   a. Latitude 27.48
   b. MMAT January = (29.4+20.7)/2 = 25°C
   c. MMAT July = (20.4+9.5)/2 = 15°C
   d. Average maximum January = 29.4
   e. Average minimum July = 9.5°C

6) Calculate Surface temperature ($T_0$)
   a. 95th percentile upper temperature 52°C
   b. Average minimum July 9.5°C

7) Calculate the effective pavement temperature using the VBA function in Appendix G
   a. Effective maximum temperature January 43°C
   b. Effective minimum temperature July 16°C

8) From Figure 43 DG14 C320 modulus is:
   a. Summer 1279MPa
   b. WMAPT 2609MPa
   c. Winter 8070MPa

9) From Figure 47 DG20 C600 modulus is:
   a. Summer 1633MPa
   b. WMAPT 3670MPa
c. Winter 10832MPa

10) Calculate the effective layer modulus from Equation 56
   a. Summer 1575MPa
   b. WMAPT 3491MPa
   c. Winter 10371MPa

11) Calculate the FEL at 95% confidence from Equation 54
   a. Summer 1595MPa 155me
   b. WMAPT 3496MPa 93me
   c. Winter 10368MPa 46me

12) Using Linear Elastic Analysis (i.e. CIRCLY) calculate critical strain using a 9 tonne axle
   a. Summer 149me
   b. WMAPT 85me
   c. Winter 38me
   d. Subgrade 224me

13) Compare Calculated strain to FEL
   a. Summer 154<155me  OK
   b. WMAPT 88<92me  OK
   c. Winter 38<46me  OK
10 Conclusions and Recommendations

By reviewing the strategies for designing and maintaining long-life pavements in Australia, the UK, France, Netherlands and several states in the US, it was found that LLAP could be achieved with a maximum thickness of 300-350mm and a minimum of 200mm was required for LLAP performance.

Examination of the gradation and volumetric properties of SRA Australian production mixes shows that despite the variability in the design methods (Marshall, gyratory and Superpave) and the differing compaction efforts, all mixes fit into a very small volumetric window. Additionally, it was found the design gradation of all standard Australian production mixes closely follow the maximum density line, with nearly all mixes being slightly coarse graded, indicating that distinguishing between Australian mixes based on gradation and volumetric properties may be difficult.

The comparison of the results obtained by AAPA and NCAT on two un-aged samples showed that the difference in the compaction method between the AAPA study and US Superpave test method had no influence on the measured dynamic modulus results, with identical results obtained. Based on this finding, it was concluded that the APS-fL project could utilize the results of NCAT testing for modulus and performance data with confidence, to:

- correlate dynamic modulus estimates from back analysis of deflection data,
- validate measured strain and predicted strain using linear elastic analysis, and
- develop fatigue endurance limits.

Examination of the master curves of standard Australian production mixes suggested the minimum modulus value appeared to be the best at distinguishing between binders and nominal aggregate size. As there is little difference in the volumetrics of Australian mixes, no significance could be found in air void levels or binder contents within sub mix types. Unexpectedly, no correlation was found between the minimum modulus and RAP content, indicating that at current RAP levels, RAP has little effect on the minimum modulus value and therefore overall modulus values. The results showed that, most likely because of the small variance in aggregate gradation and volumetric properties, there was no change in the shape of the master curve within groups of the same binder types and nominal aggregate size.

Because of the consistency of the master curves for a given binder type and nominal mix size, it was found for practical implementation that it was not necessary to develop complex master curve equations for routine pavement designs. The results of grouping of Australian mixes showed that Australia can achieve a higher degree of accuracy by grouping common mixes, than from the use of complex model such as the Witzak or the Hirsch models.

It was found that because the variability in the prediction of modulus followed a normal distribution with the standard deviation equal to that of the standard error, confidence could be established from the grouped data by simply varying the minimum modulus data to move the dynamic modulus curve.
down the modulus scale. By doing this it was shown that confidence level master curves could be established for the nominal 14 and 20mm mixes and the three primary binder classes used in Australia, C320, AR450 and C600.

The analysis of dynamic modulus test results from NCAT, MnRoads and WesTrack, against field stiffness determined from FWD testing, found that frequency in the dynamic modulus test should be considered as an angular frequency and that a shift of $1/2\pi$ on the frequency axis will allow the use of dynamic modulus values to determine the modulus resulting from a pulse load in the field. Using this conversion it was found that dynamic modulus results at 5.3Hz ($1/2\pi\times0.03$) could be used to accurately predict the modulus determined from FWD loading with a pulse width of approximately 0.03seconds over a wide range of temperatures. The results of the optimisation found that the use of the mid-layer depth resulted in a slight underestimation of the effective asphalt layer modulus for day time testing and that if mid-layer depth is used a correction of +2°C is required to correct for the average temperature within the asphalt layers. Therefore, if modulus calculations were to be undertaken at times of day other than typically mid-day, more work on modelling the full temperature with depth profile in the pavement structure would be needed to determine the effective temperature of the asphalt layers.

Using the pulse frequency conversion and temperature correction obtained from comparison with FWD testing, the multi-variable optimisation found that dynamic modulus could be used to accurately predict strain under a moving load using layer elastic analysis when time of load is corrected for the effective load length. It was found that when computing strain under a moving load, contrary to some published recommendations, the thickness of the asphalt layer was insignificant in determining strains. It was also found that the time of loading is more related to the length of the deflection response than the current approach of the use of a stress pulse.

The results of the optimisation on the thick asphalt sections of Phase II NCAT support the recommendation of Coffman (1968) that a vehicle acts as a cyclic load with a wave length of six feet, with the optimisation determining the wave length of 1.8m. Based on these findings the following frequencies in the dynamic modulus test are recommended for use in pavement design with an equivalent combined asphalt layer.

<table>
<thead>
<tr>
<th>Speed (km/hr.)</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recommended Frequency Dynamic Modulus $E^*_{\text{Test}}$ (Hz)</td>
<td>1.2</td>
<td>1.5</td>
<td>1.7</td>
<td>2.0</td>
<td>2.2</td>
<td>2.5</td>
<td>2.7</td>
</tr>
</tbody>
</table>

The analysis showed that multi layers were sensitive to the chosen sub layer thicknesses and more work would be required on determining both the appropriate sub layering of multilayer asphalts and the effect of temperature profiles in the sub layering, before a multilayer approach should be recommended over the use of the equivalent asphalt layer approach.
The analysis has shown that there is a direct link between laboratory modulus and strain under a moving vehicle and dynamic modulus can be used in the structural design of LLAP’s. The use of the master curves will enable the determination of either threshold strains or cumulative distribution of asphalt strain in LLAP structures as a function of the climatic conditions and the traffic distribution spectrum. This calculated strain will provide the means to rationally evaluate the compliance of candidate LLAP structures with the limiting threshold strain or cumulative strain distribution empirically derived from the evaluation of international LLAP’s.

The results of the modulus inter-conversion study found that 3 of the 4 common Australian test methods time has a different physical meaning and a frequency conversion is needed to shift between the time and frequency domain. It was found that shift factors could be established from single time/temperature testing and that the time shift factors found form the single time and temperature testing and are valid across the whole time frequency domain.

For the purposes of standardisation, the modulus results need to be converted to a reference stiffness value. Comparable stiffness is obtained, between any of the three test methods by using a constant definition of time (inter-conversion) as shown following.

<table>
<thead>
<tr>
<th>Conversion From</th>
<th>Conversion To</th>
</tr>
</thead>
<tbody>
<tr>
<td>IT-CY(time)</td>
<td>4PB-PR(frequency)</td>
</tr>
</tbody>
</table>
| IT-CY(time)    | □ □□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□□"
limiting value. The research work showed that the basic material property, stiffness, is directly related to the FEL of a mix and can be used to allow for changes in binder, temperature and healing.

The examination of the LLAP sites from the NCAT Phase II study confirmed, in field, the variable nature of the FEL relationship and that LLAP can withstand strains significantly higher than previously recommended (when the asphalt has low stiffness) without undergoing damage. The examination of the two LLAP on the NCAT test track showed a direct relationship exists between the infield stiffness-strain curve of the two undamaged sections and the stiffness-FEL developed by from Australian and US mixes. This finding was then extended to actual LLAP in Australia, which confirmed the same finding as on the NCAT test track that a direct relationship exists between the infield stiffness-strain curve of LLAP and the laboratory determined stiffness-FEL curve. This direct relationship allows the development of a phenomenological stiffness-FEL relationship which can be used for the purpose of pavement design.

Based upon both the calibrated relationship from the NCAT data and validated Australian and UK LLAP, the use of a stiffness-FEL relationship is recommended for the design of long-life asphalt pavements in Australia. The relationship was validated to Australian and UK data and only required a slight modification to the relationship developed from the NCAT, as shown following.

\[ F_{EL} = k_1 S_{mix} + k_2 \]

Where;

- \( F_{EL} \) is the Fatigue Endurance Limit
- \( S_{mix} \) is the stiffness of the mix, and
- \( k_1 \) is the adjustment constant for differences in rest periods, or confidence levels
- \( k_2 \) is the mix adjustment factor

It was found that confidence limits could be incorporated into the modelling approach by the use of the \( k_1 \) adjustment factor and confidence limits could be determined from the Australian validation sites as shown following.

<table>
<thead>
<tr>
<th>Confidence Level</th>
<th>80(^{th})</th>
<th>85(^{th})</th>
<th>90(^{th})</th>
<th>95(^{th})</th>
<th>97.5(^{th})</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k_1 )</td>
<td>1</td>
<td>0.97</td>
<td>0.925</td>
<td>0.86</td>
<td>0.8</td>
</tr>
</tbody>
</table>

The analysis of the results showed that the design of LLAP can be supported by a fundamental test using the Ratio of Dissipated Energy Change (RDEC) and the constant Plateau Value (PV\(_L\)).
However, a laboratory to field conversion is needed to use the test which is equivalent to the confidence values shown previously.

It is recommended the stiffness-FEL design approach be incorporated into a multi-season design approach. The current design recommendation for Australia to use:

- 9.0t design axle (8.2t may be used with a shift in the equation)
- Model using 3 seasons; Summer, Mid temperature (WMAVT), Winter.
- Mid layer asphalt temperatures be taken from the ARRB modified Bells equation

By using this approach the limiting thickness of asphalt pavements can be obtained with confidence for the full spectrum of circumstances encountered on Australian projects.
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Appendices

Appendix A Mix Details and Dynamic Modulus Results

Appendix B STEP Sites

Appendix C Confidence Interval Calculations

Appendix D Laboratory to Field Conversions

1 FWD

2 Strain

Appendix E Backcalculation results

Appendix F FEL Validation

1 LLAP Sites

2 Non LLAP Sites