Interconversion of Laboratory Measured Modulus Results to Field Modulus and Strain

Bevan Sullivan, National Technical Manager, Fulton Hogan Pty Ltd, Australia

Ian Rickards, Pavement Consultant, Australian Asphalt Pavement Association

Saeed Yousefdoost, PhD Candidate, Faculty of Engineering and Industrial Sciences, Swinburne University of Technology

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Abstract

In 2011 AAPA initiated the Asphalt Pavement Solutions for Life (APS-fL) project. This project was initiated to address the concern of industry that current pavement design procedures where producing overly conservative asphalt thicknesses and the concept of long life asphalt pavements where valid. The final objective of the APS-fL project is to develop a long life pavement design procedure.

As part of the APS-fL project a database of a Dynamic Modulus of typical Australian mixes has been established along with frequency-temperature master-curves. It is proposed that these master-curves form a major input for the APS-fL pavement design procedure. However, as with any laboratory measured value, all of the complex states of stress, temperature and load frequencies in the pavement structure cannot be practically assessed and conversion and/or validation between laboratory and the field needs to be undertaken.

As there is still significant debate about the exact conversion between frequency in the dynamic modulus test and loading time resulting from a pulse load in a pavement structures, this study utilises 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 conversion between the dynamic modulus test and field measured modulus. It then uses this conversion to validate strain results obtained at the NCAT test track against strains predicted by the use of layered elastic analysis and the converted modulus.

This paper will present the background of, master-curves, a discussion on the conversion between frequency in the dynamic modulus test and load time under a loading pulse in the field, determine a direct conversion between dynamic modulus and field modulus using real data and provide recommendations on the use of dynamic modulus to predict the strain response under a moving load using layered elastic analysis.

1 Introduction

The APPA APS-fL project is a Mechanistic Empirical Pavement Design procedure and as such requires accurate determination of material properties under field loading conditions to calculate pavement response, in terms of stresses, strains, and/or displacements using mechanistic models. In mechanistic design this calculated response is then used as the pavement response variable in empirical relationships to predict structural damage, (decrease in moduli or cracking) or functional damage (rutting and roughness) or for long life pavement design, ensure pavement response is below tolerable levels to ensure no little to no damage occurs to the pavement.

To accurately predict performance all of these steps must be reasonably correct and validated. If the fundamental characterisation property, modulus, used to calculate the pavement response has no similarity to the actual modulus required to achieve the pavement response, there will be no point in trying to use this response to determine the point at which no damage occurs to the pavement, which is empirical validated. In other words, only if the calculated response is reasonably correct does it make sense to try to relate this response to damage or lack thereof to the pavement.

As part of the APS-fL project a database of the dynamic modulus of typical Australian mixes has been established along with frequency-temperature master-curves. It is anticipated that these master-curves will form a major input for the APS-fL pavement design procedure. However, as with any laboratory measured value, all of the complex states of stress, temperature and load frequencies in the pavement structure cannot be practically assessed and conversion and/or validation between laboratory and the field needs to be undertaken, which is the objective of this research.

Currently, there is no recommended method in the literature which has been validated against field measurements for the conversion of laboratory determined dynamic modulus to the calculation of strains under a moving vehicle. Fortunately, the NCAT track has a significant amount of data which can be used to determine the conversion between dynamic modulus and field modulus determined using the FWD. This conversion can then be used to validate the effect of load speed and thickness of an asphalt layer to obtain an effective frequency in the dynamic modulus test by calibrating and validating strain results obtained at the NCAT test track to strains predicted by the use of Layered Elastic Analysis (LEA).

2 Background

2.1 Conversion between Laboratory and Field Modulus

The AAPA APS-fL procedure proposes the use of the frequency-temperature dependent dynamic master-curves as the method for determine 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 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 against 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 undertaking this 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, as shown by Sullivan et al. (2013). Sullivan established that there was no difference between the 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.

2.1.1 Data for Comparison

As part of 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 which have documented results for field measured modulus determined from FWD testing and laboratory characterisation of asphalt mixes undertaken using the dynamic modulus test.

2.2 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). With researchers such as Dongre et al. (2006) recommend the angular frequency approach, $t= {}^{1}/_{\omega}$, while researchers such as Katicha et al. (2008) recommend the pulse frequency, $t= {}^{1}/_{f}$, approach.

It appears that the earliest use of the angular frequency approach, $t= {}^{1}/_{\omega,}$ 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 subsequently adopted by adopted by Shell in their development of a ME pavement design guide and was subsequently adopted for the basis of the Austroads (1992) asphalt characterisation method. However, the angular frequency approach has not been universally adopted by all design procedures with the US MEPDG following the $t = {}^{1}/_{f}$, approach.

The reason the t= $^1/_\omega$ 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).

$$F^{-1}\left[E'(\omega) = \frac{1}{2\pi}\int_0^{\infty} H(\tau)\frac{\omega^2\tau^2}{1+\omega^2\tau^2}d\tau\right]$$

Where,

ω is angular frequency in rad/s
f is the cyclic frequency in Hz
τ is loading time in seconds
$H(\tau)$ 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= $^1/_{\omega,}$ this is somewhat contrary to the early recommendations of Van der Poel(1954) who suggested the conversion was only approximate. Notwithstanding this, Dongre did not establish that the testing in dynamic modulus test was an angular frequency, which would require the above conversion and the issue is still not resolved amongst researchers.

2.3 Modulus to Field Conversions

Nearly all of the research undertaken to date to correlate laboratory modulus with field modulus has relied upon the use of resilient modulus test. Because the resilient modulus test only measures a single point in the frequency-temperature space, all attempt to convert this single result across the whole frequency-temperature space under field loading where extremely difficult and as such generally resulted in poor agreement.

This not the case for the dynamic modulus test, as the dynamic modulus test produces master-curves, which can be used to determine modulus at any frequency or temperature. The use of dynamic modulus test can and has offered significant benefits in the determination in the conversion between laboratory and field modulus data. More recent attempts to correlate field modulus to dynamic modulus, such as the study by Clyne et al. (2004) using results from the MnRoad test track, providing promising results. Clyne found that dynamic modulus test results correlate rather well with FWD modulus values across the full range of temperatures. While the results correlated rather well, there was a distinct bias in the results and no recommendations were made on the final conversion of between dynamic modulus and field modulus. The results found by Clyne were consistent with a number of other researcher's findings, such as Soe et al. (2012), with nearly all studies finding dynamic modulus results are higher than field measured modulus. However, all these researchers have use the assumption that frequency in the dynamic modulus test was the reciprocal of the time of loading in the FWD test.

The assumption that FWD frequency in the dynamic modulus test was the reciprocal of the time of loading in the FWD test follows the approach recommended by Loulizi *et al.* (2002) who showed that FWD load pulse could be simulated by a haversine wave with a time of 0.03 seconds. Loulizi then used this time to recommend a frequency of 16Hz, using the reciprocal of a sine wave $^1/_{(2x0.03)}$. As previously mentioned there is still currently debate amongst researchers whether this assumption (the reciprocal) is correct.

When examining the relationship between laboratory dynamic modulus and modulus determined from FWD loading, it is important to keep in mind that some approximations need to be undertaken:

- Firstly, back-calculation nearly always considers the entire depth of the asphalt layers, while the dynamic modulus test considers each mix separately.
- Secondly, the dynamic modulus test is conducted at uniform temperatures throughout the specimen, while there are thermal gradients through the layers in the field.
- Thirdly, dynamic modulus test are conducted at a uniform stress state, while varying degrees of stress are present within a pavement structure.
- Finally, the dynamic modulus test is conducted at fixed frequencies throughout the specimen. In the field a gradient may exist due loading, related to the proximity of the asphalt to load.

While these differences are challenging, it is believed that by using a full optimisation process these differences can be taken into account through the determination of calibration or equivalency factors which can be determined to take into account the effects of multiple layers, temperature gradients and frequencies.

2.4 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 to translate laboratory tests to field modulus that the cycle length and the phase shift needed to be known. Coffman based his recommendation on the correlation of field testing and analytical analysis and determined that cycle length was 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 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 of work done by Barksdale (1971). Browns 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 Browns using the average stress loading time is shown following:

$$logt = 0.5h - 0.2 - 0.94logV$$

Where;

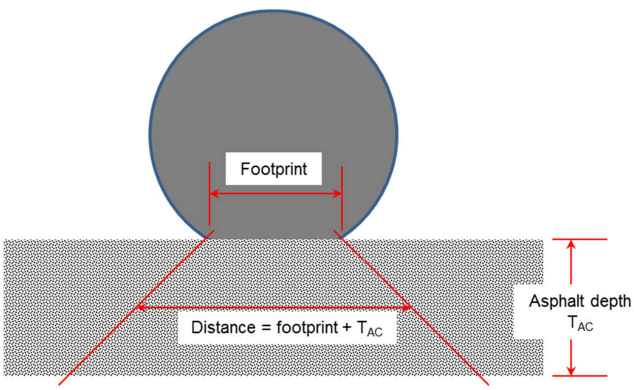
t= loading time (sec)

h = thickness (mm)

V = vehicle velocity (km/hr)

Brown's method has subsequently gained widespread acceptance and forms the basis of various different design process throughout the world, such as the current Austroads deign procedure.

Ulliditz (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^{o} from the tyre radius. The model proposed by Ullditz is shown conceptually in Figure 1 following:



$$t = \frac{200 + 2h}{S}$$

Where;

S = the vehicle speed in (mm/s)

200 = is assumed to be the length of the tyre contact

Figure 1 Model used to calculate load duration

The US Mechanistic Empirical Design Guide (MEPDG) makes uses of 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 the subgrade layer. In theory this should result in significantly longer loading times than the approach adopted by Ullditz and Brown.

2.5 Empirical Relationships to Frequency

Yager (1974) used the early work by Coffman to relate vehicle speed to test frequency using the 6foot load pulse, Yager determined frequencies of 1, 6 and 12Hz related to speeds of 10, 40 and 80km/hr respectably. Jacobs et al. (1996) recommended a loading frequency of 8Hz to correspond to a vehicle speed of 60km/hr. In NCHRP report 465 Witczak 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.

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 would be 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 two 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.

2.6 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 there is currently little published work which has been done 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 is 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.

3 Method and Calibration Plan

Because 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 and are still open for debate. The mathematical solution of this problem was determined to be outside of the scope of this project and does not appear to be easily solved. Due to this complexity, it was decided 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 a calibration coefficient 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 exists.

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 determined to determine strains under a moving vehicle load using the results of dynamic modulus testing.

3.1 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 are relatively, correct to ensure the solver function converges on the correct optimal solution. A three stage sequential optimisation approach was used in the analysis, with the a three stage sequential approach being outlined in Table 1 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 1 Optimisation Approach

	Process	Data Source
Stage 1- Calibrate lab field modulus interconversion	Calibrate Frequency Interconversion	FWD Data NCAT
	Calibrate Temperature Profile	
	Validate findings with WesTrac and MnRoads	FWD WesTrack and MnRoads
	↓	
Stage 2- Calibrate frequency under moving load	Calibrate load pulse width	NCAT |E*| and strain
	Calibrate effect of frequency with depth	
	↓	
Stage 3- Validate multi-layer asphalt	Validate Multi-layer findings	NCAT |E*| and strain

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 and stress path within the pavement using an equivalent single layer of asphalt.

And finally, optimise the model for use with multi-layer asphalt pavements.

4 Field Calibration

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 sources 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 Elmod 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.

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 where used within the pavement structure.

As the initial optimisation process proposed 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 via the following equation.

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

Where;

E_{eff} is the effective modulus of the combined layers

h_i is the thickness of layer i

E_i is the modulus of layer i

n is the number of asphalt layers

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 by Sullivan (2013).

The master-curve fitting parameters for the resulting master-curve for the combined layers are shown in Table 2 following, the sites chosen for the analysis where 6 NCAT test cells where little damage had occurred. Additionally shown in Table 2 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 2 Modulus Calibration Master-curve Fitting Parameters

Test Cell	Temperature Shift Factors (T_{ref} =20°C)		Sigmoidal Fitting Parameters			
	a	B	α	β	γ	δ
N3-Phase III	0.0003	-0.116	1.810	2.774	-0.711	-0.363
N5 Modified-Phase II	0.0004	-0.119	1.688	2.896	-0.770	-0.347
N7-SMA	0.0004	-0.122	1.787	2.897	-0.649	-0.359
S9-Control	0.0004	-0.115	1.868	2.708	-0.709	-0.372
S10 WMA-F	0.0004	-0.115	1.704	2.638	-0.998	-0.457
S11 WMA-A	0.0004	-0.115	2.014	2.316	-0.769	-0.5060
MnRoad C34	0.0007	-0.118	1.564	2.883	-0.500	-0.5231
MnRoad C33	0.0003	-0.109	1.891	2.397	-0.592	-0.6270

WesTrac C2	0.0005	-0.135	1.900	2.281	-0.721	-0.4830
WesTrac C5	0.0001	-0.148	2.138	1.912	-0.640	-0.4840
WesTrac C6	0.0016	-0.155	2.171	1.915	-0.551	-0.6850
WesTrac C7	0.0008	-0.136	2.055	2.151	-0.550	-0.5929
WesTrac C23	0.0017	-0.156	2.172	1.915	-0.550	-0.6850
WesTrac C24	0.0008	-0.135	2.029	2.044	-0.651	-0.4937

4.3 Conversion between FWD and Dynamic Modulus

To initially determine which of the two approaches debated in the literture 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 intial analysis to compare the findings from both approaches. 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, $t = {}^{1}/_{2\pi f}$, and pulse frequency, $t = {}^{1}/_{f}$ 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 where utilised:

- ${}^{1}/_{0.03}$ or 33Hz
- or ${}_{1}/\omega$ 33 rad s-1 or 5.3hz

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 2, for both the angular frequency approach and the cyclic pulse frequency approach.

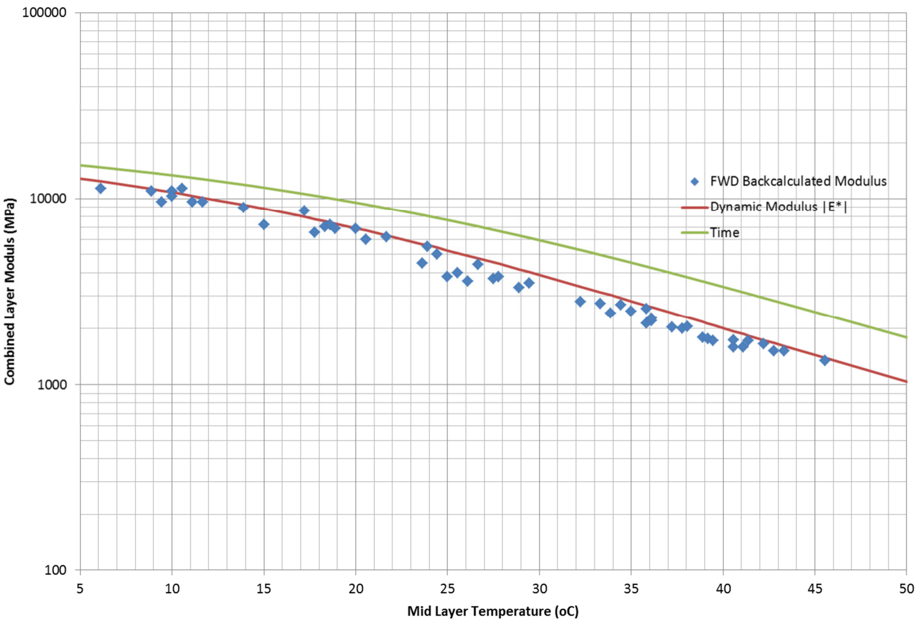


Figure 2 S9 Control Mix Frequency Conversion

The results of the comparison of the Phase IV control section S9, suggest that the angular frequency approach appears to be 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 2π 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.

The 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 with 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.

4.3.1 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 where 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.

$$f_{dm=\frac{1}{kt_p}}$$

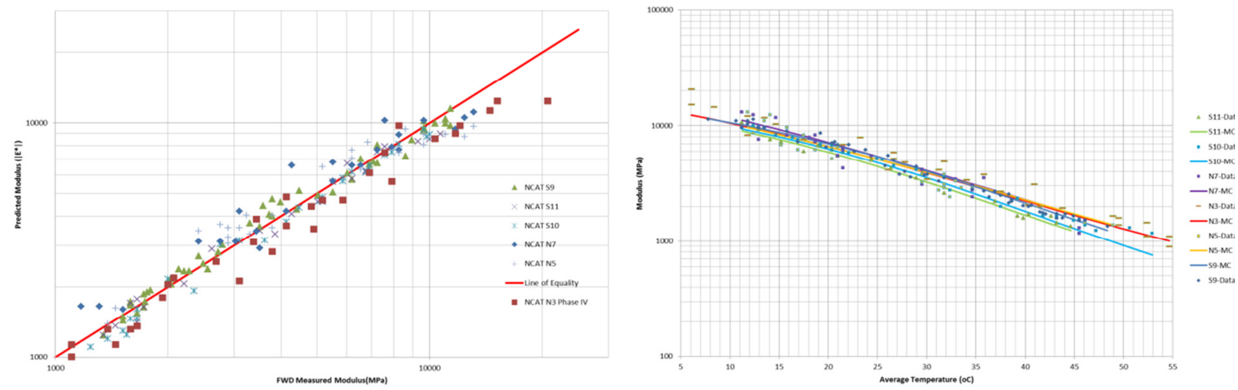
$$T_{ave} = at_{mid} + b$$

Where 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 2π for a, b and k respectively.

The seed and trail calibration coefficients where 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 found that the frequency time conversion constant, k, for all practical purposes was equal the value recommended by Dongre(2006) of 2π. 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 2π.

It was found through the optimisation that the temperature equivalency multiplication factor, a, approached that of 1 and the addition factor, b, approaching 2. For simplicity, the value of a and b was set at 1 and 2 respectively. Figure 3 shows the results of the comparison with the constants of 2π, 1 and 2 applied for (a) the FWD modulus results against that of the Dynamic modulus results, with (b) showing the same results only this time plotted against average layer temperature.



(a) (b)

Figure 3 NCAT Dynamic Modulus Vs. FWD Modulus

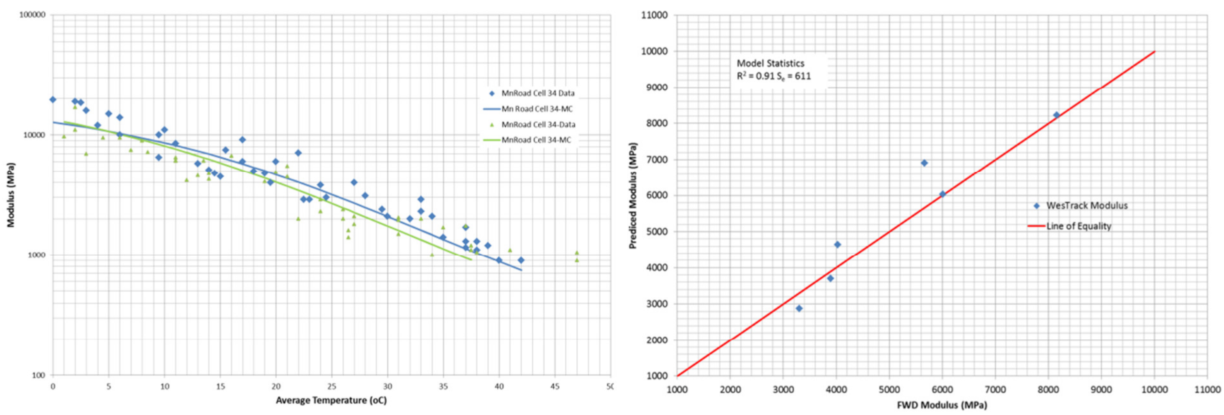
From both figures it is clear that there is a very strong correlation between dynamic modulus and modulus determined from FWD testing. The nonlinear optimisation of frequency and average temperature confirmed the earlier findings of Sullivan (2013) in comparing dynamic modulus results against resilient modulus results 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.

4.3.2 Validation against MnRoads and WesTrack

To validate the findings obtained from the NCAT test sites and determine if the results were transferable to other mixes and climates. The dynamic modulus results and field modulus reported for MnRoads and WesTrack test tracks were analysed using the same calibration factors.

Figure 4 following shows the results of the comparison undertaken on MnRoads (a) and WesTrack (b) test tracks. For MnRoads field modulus data was available across a range of temperatures for Cell 33 and 34. For WesTrack only results were published for Phase 1 testing at a single temperature for a number of sites.



(a) (b)

Figure 4 Westrack Modulus Comparison

While the scatter of results is higher than that found a NCAT, from both figures, it is clear that at these two additional locations with a number of different mixes, that again a very strong correlation is show 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 results found by Sullivan (2013) that at higher temperature and low frequencies modulus is stress susceptible. The results show that the effect of stress suitability 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:

$$f_{dm} = \frac{1}{2\pi t_p}$$

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, when compared to 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 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 Calibration of Strain

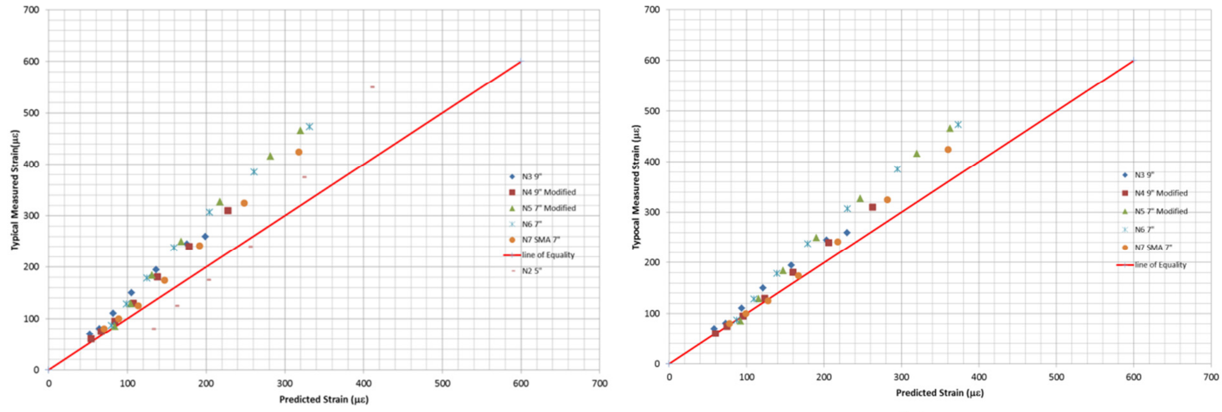
5.1 Validation of Stress Based Approaches

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 undertaken.

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 Sullivan (2013) and taken from Timm et al. (2003), to calculate the strain at the underside of the asphalt layer using Linear Elastic Analysis. These strains where 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 taken for the analysis consisted of those sites from the Phase II study where asphalt thicknesses where greater than 150mm (Constant stress), as the Long life pavement solution method will only be concerned with the thicker asphalt pavement sections.

The results of the analysis can be seen in Figure 5 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.



(a) (b)

Figure 5 Validation of Pulse with Depth Approaches (a) Brown (b) CalMe

As can be seen both of the proposed pulse time approaches, based on the use of a stress pulse, result in an under prediction of strain values. This is due to an underestimate of the time of loading which 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 therefore 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.

5.2 Numerically Optimised Approach

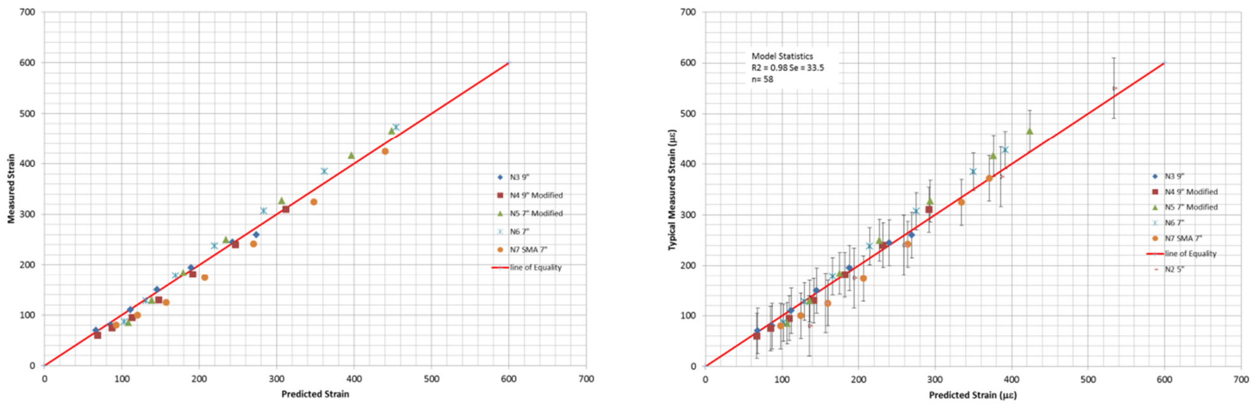
Given both the Brown and CalMe models did not appear to effectively model both the time of load and the effect of depth on frequency with depth, a fully unconstrained optimisation approach was run to determine if better agreement could be achieved between the measured an predicted strain. In the unconstrained 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:

$$t_p = \frac{a + bh}{v}$$

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 6 (a) following.



(a) (b)

Figure 6 unconstrained optimisation (a) equivalent layer (b) multilayer

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 approaching 1. 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 all current 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. These 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 and Brown against that of Coffman, was that Coffman used deflection (strain) against to determine the load pulse with, 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 an full or partial depth asphalt pavement (>125mm) and a constant frequency be used for all thicknesses.

The results of this analysis was then extended to that of a multi-layer asphalt pavement, Figure 6 (b), 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 the both 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 all asphalt layers be used for the purpose of design.

6 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 ($^{1}/_{(2*\pi*0.03)}$) 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 underestimate of the effective asphalt layer modulus for day time testing and that if mid-layer depth is used a correction of +2^{o}C is required to correct for the average temperature within the asphalt layers. Therefore if calculations of modulus are going to be undertaken at times of day other 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.

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 published recommendations, the thickness of the asphalt layer was insignificant in determining strains. It was found 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 1.8m. 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.

Speed (km/hr)	50	60	70	80	90	100	110
Recommended Frequency Dynamic Modulus |E*| Test (Hz)	1.2	1.5	1.7	2.0	2.2	2.5	2.7

The analysis showed that multi layers were sensitive to the chosen sub layer thicknesses and more work is required on determining the both 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.

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