Technical Paper

ISSN 1997-1400 Int. J. Pavement Res. Technol. 4(2):111-117

Implications of Using Calibrated and Validated Performance Transfer Functions in the Mechanistic Empirical Pavement Design Procedure

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Abstract: The Adoption of Mechanistic Empirical pavement design is growing steadfastly in many countries around the world. Australia and New Zealand adopted Mechanistic Empirical more than a decade ago. However, the success of the empirical design method depends on the use of well designed, calibrated and validated transfer functions. The transfer functions are the empirical component of the design procedure that relates pavement response with certain pavement performance indicators. In the Austroads Mechanistic Empirical pavement design, fatigue and rutting are the two performance indicators used in the design. Austroads guidelines adopted the Shell fatigue transfer function to predict the fatigue life of asphalt pavements. However, it was observed by many practitioners and confirmed by this study that Shell Transfer function significantly overestimates the design thickness or in other words underestimates the fatigue life of asphalt mixes. In this paper, calibration and validation of the Shell fatigue transfer function which is currently adopted in the Austroads design guidelines are demonstrated. The calibrated Shell fatigue performance model is integrated with the Austroads design method. A case study from the Christchurch Southern Motorway project has been analyzed using both the current design method and the suggested calibrated Shell performance model. The calibration factor based on the tested beams was found to be in the order of 5.6824. Using the calibrated Shell model produced a 26% to 27% thinner asphalt thickness compared to the current Austroads design guidelines.

Key words: *Calibration; Fatigue; Flexible; Pavement; Validation.*

Introduction

For many years pavement engineers utilized successfully empirical techniques to design highways and airfield cross sections. However, due to the limitations of the empirical techniques several highway authorities and Transportation departments have shifted to mechanistic empirical pavement design methods. However, the success of the empirical design method depends on the use of well designed, calibrated and validated transfer functions. The transfer functions are the empirical component of the design procedure. Transfer functions relate pavement responses with certain pavement performance indicators. In the Austroads Mechanistic Empirical pavement design, fatigue and rutting are the two performance indicators used in the design [1]. The Austroads design guidelines and New Zealand supplement adopted Shell fatigue transfer function to calculate the fatigue damage of the structural asphalt. The Shell fatigue model was developed in 1978. The fatigue relationship was developed in the laboratory on a range of typical asphalt mixes used in different European countries and was done using controlled strain (displacement) with a sinusoidal loading [2]. It was observed by many practitioners that the Shell transfer function underestimated fatigue life. It was also noted in 2007 version of the New Zealand supplement that asphalt pavements designed based on the earlier State Highway Pavement and Rehabilitation design manual are 30% thinner than that required by the current Austroads guidelines [3]. According to the New Zealand supplement, it was also observed that two third of the Wellington and Auckland State highway network were designed and constructed based on the earlier design method and they are all performing well past their design life [3]. It is clearly obvious that the Shell transfer function was developed for different types of binders and using different mix designs than those are currently used on the New Zealand highway system. Consequently, there are differences between the predicted and observed fatigue life. Thus, the Shell model needs to be adjusted to account for these differences. In this research, the Shell fatigue model is only calibrated using laboratory measured fatigue values and field calibration will still be needed to provide the required accuracy of the model prediction. The reason for using laboratory fatigue values to adjust the Shell fatigue model is that first, there is no fatigue field data available for any specific type of mix in New Zealand. Secondly, as mentioned previously, the Shell model is developed for different European mixes which are different from New Zealand mixes; therefore, laboratory calibration will readjust the model prediction to better match the actual NZ mixes.

Calibration of a pavement fatigue model is the process of adjusting the predicted values of pavement fatigue so that the predicted and measured values match as closely as possible for different strain levels and different asphalt mixes. Fatigue models developed in the laboratory always underestimate the fatigue life of asphalt mixes in the field [4, 5]. The main reasons for this are due to many shortcomings in the laboratory testing which do not exactly simulate the field fatigue. For example, traffic wandering and the healing effects that usually happen in the field are not simulated in the laboratory and instead all fatigue testing effects are carried out on one section continuously without a rest period between the loading pulses in the lab. These differences make the fatigue simulation in the lab more severe and quicker compared to the actual fatigue in the field. In addition, there are many other differences between the laboratory and field conditions including

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Note: Submitted April 22, 2010; Revised October 6, 2010; October 7, 2010.

Fig. 1. Downer EDi Works Contractor Roller Compactor.

Table 1. Volumetric Properties of the Loose and Compacted AC14 Asphalt Mix.

Beam Number	G_{mb}	VA (%)	VMA (%)	VFB (%)
1	2.513	5.2	15.6	66.4
2	2.534	4.5	14.9	70.1
3	2.531	4.6	15.0	69.5
4	2.547	4.0	14.4	72.6
5	2.494	6.0	16.2	63.2
6	2.489	6.2	16.4	62.5
7	2.495	5.9	16.2	63.4
8	2.49	6.1	16.3	62.6
9	2.478	6.6	16.8	60.8
10	2.477	6.6	16.8	60.7
11	2.481	6.5	16.7	61.3
12	2.484	6.3	16.5	61.8
13	2.493	6.0	16.2	63.1

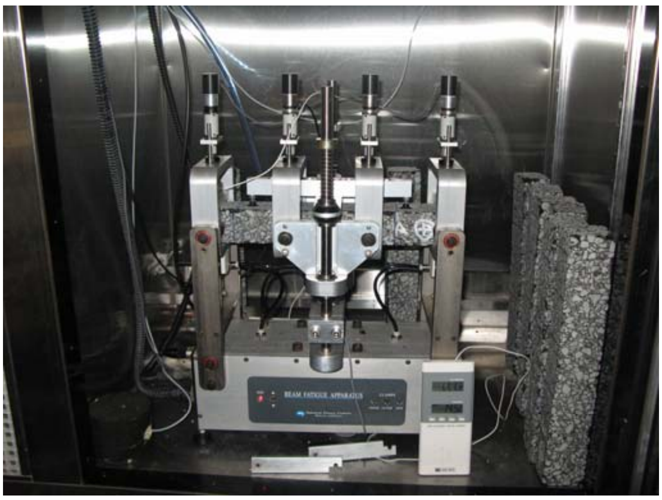


Fig. 2. University of Canterbury Bending Beam Fatigue Apparatus.

temperature variation in the field, traffic loading, and the condition of failure in the field compared to that assumed in the laboratory. Therefore, laboratory models need to be calibrated using shift factors to adjust the laboratory developed model to match the actual fatigue life.

The calibration is performed so that the difference between observed results (e.g., the measured fatigue of a pavement section) and the Shell predicted results is reduced to a minimum value. This fitting of the predicted to the observed results is most often accomplished by minimizing and error function of the residuals [6].

The calibration and validation of the Shell fatigue transfer has been carried out and explained in full details in another publication by the author [7]. In this research, only the summary of the calibration and validation process is explained with a detailed design of two road sections taken from the current Christchurch Motorway project to show the implications of using the calibrated and validated transfer function on the pavement design.

Sample Preparations and Laboratory Testing

The asphaltic concrete samples prepared for the purpose of the fatigue testing were made by a local contractor. The aggregate gradation complies with Austroads AC14 dense graded hot mix asphalt with a maximum nominal aggregate size of 14 mm. The 60/70 penetration grade asphalt binder was selected for this study. Asphalt slabs were compacted using roller compactor similar to that shown in Fig. 1. The asphalt concrete slabs were cut into beams with width, depth and length of 65×50×390 mm, respectively.

The binder content for all slabs (Pb) is 5.02% by the total mass of the mix and the maximum theoretical specific gravity (Gmm) of the uncompacted mix is 2.652. Table 1 shows bulk specific gravity (Gmb), percentage of air voids (VA), percentage of voids in the mineral aggregates (VMA), and percentage of voids filled with Bitumen (VFB).

Laboratory Fatigue Testing

The fatigue test was carried out on the thirteen beams using a constant strain mode in the third point bending test as shown in Fig. 2. Each specimen was subjected to a haversine loading pulse at a frequency 10 Hz at 20℃ until failure. Failure was defined as the number of cycles at which the flexural stiffness of the asphalt beam is reduced to 50% of its initial stiffness. Table 2 shows the measured fatigue for the 13 beams for different strain levels range from 300 $\mu\varepsilon$ to 600 $\mu\varepsilon$. In order to achieve reliable results, each test was replicated at least twice to provide confidence of the test repeatability. In Table 2, comparing the measured fatigue values and the predicted fatigue values, one can clearly see that the Shell fatigue function underestimates the fatigue life by a factor 5.5 times.

Fig. 3 shows the relationship between the applied tensile strain level and the Shell predicted and the actual measured fatigue values. From Fig. 3, the Shell predictions at all strain levels are significantly less than the actual measured fatigue life. The ratio between the actual measured fatigue life and the Shell predicted fatigue life for the same strain level, volumetric properties and flexural stiffness modulus ranges from 3.1 to 8.9 with an average value of 5.5. It is also clear that the measured fatigue values run almost parallel to the Shell model predictions. This suggests that when calibrating the

Table 2. Measured and Predicted Fatigue Lives of the Tested Beams.

Beam Number	Strain Level (με)	Initial Flexural (MPa) Stiffness	Percentage of Bitumen by Volume, V_b (%)	Predicted Fatigue Life by the Shell Equation, N_f	Measured Fatigue Life, N_f	Ratio between Measured and Predicted Fatigue Lives
1	300	4491	12.5	399354	1230550	3.1
2	300	4223	12.3	402579	2248110	5.6
3	300	4409	12.2	363792	3235560	8.9
4	400	4530	12.4	87782	649490	7.4
5	400	4576	12.5	89519	466150	5.2
6	450	4297	12.3	51852	217710	4.2
7	450	4150	12.2	53814	342000	6.4
8	450	4318	12.2	50380	299600	5.9
9	500	4047	12.5	36397	182820	5.0
10	500	4203	12.2	31516	145420	4.6
11	600	3752	12.3	15679	97300	6.2
12	600	3548	12.2	16840	62260	3.7
13	600	4381	12.3	11840	68230	5.8
Average						5.5

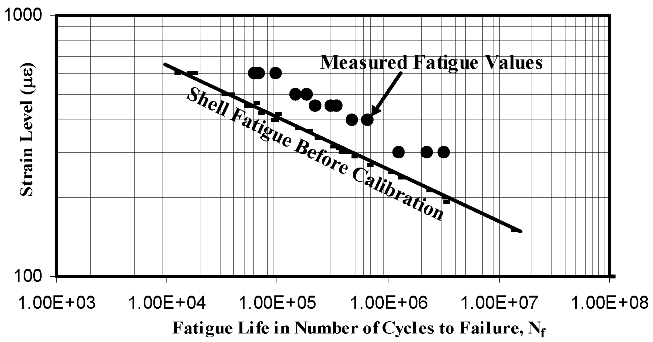


Fig. 3. Fatigue Life as Predicted by Shell Model versus Actually Measured Fatigue Life**.**

Shell model, no shape correction is required and only a shift factor will be needed to adjust the Shell fatigue model.

Determination of Calibration Factors

The calibration factors can adjust either the shape of the model or shift the model parallel to its original direction without any change in the function shape or they can adjust both the shape and the position of the model. In this paper, only adjustment for the position of the Shell fatigue model will be carried out without any change to the shape of the function. The calibration procedure is based on finding a calibration factor that when multiplied by the Shell predicted fatigue life minimizes the total prediction error. The details of the mathematical calculations of the calibration factor were presented somewhere else [7]. Eq. (1) is the final results of the derived calibration factor.

$$k = \frac{\sum_{i=1}^{n}[f(x_i * MFV_i)]}{\sum_{i=1}^{n} f(x_i)^2} \tag{1}$$

$$f(x_i) = N_i = \left[\frac{6918*(0.856*V_b + 1.08)}{E^{0.36} * \mu\varepsilon_i}\right]^5 \tag{2}$$

where:

k = the calibration factor

$f(x_i)$ = the Shell predicted fatigue value at strain level i before any calibration as shown in Eq. (2)

MFV_i = the measured fatigue value at same strain level i

N_i = the number of load cycle to cause fatigue failure at tensile strain level i

V_b = the percentage of binder by volume of the asphalt mix

E = Stiffness modulus of the asphalt mix in MPa

$\mu\varepsilon$ = tensile strain at the bottom of the asphalt in micro-strains

Using Eq. (1), a calibration factor of 5.6824 was derived from the data presented in Table 2. When the calibration factor of 5.6824 is multiplied by the Shell model will provide a minimum total prediction error as previously explained. Therefore, the calibrated Shell fatigue model can be rewritten as shown in Eq. (3) or mathematically simplified as Eq. (4).

$$N_{f(calibrated)} = 5.6824*\left[\frac{6918*(0.856*V_b + 1.08}{E^{0.36} * \mu\varepsilon}\right]^5 \tag{3}$$

$$N_{f(calibrated)} = \left[\frac{9792*(0.856*V_b + 1.08}{E^{0.36} * \mu\varepsilon}\right]^5 \tag{4}$$

Fig. 4 shows that applying an optimum calibration factor to the Shell model significantly adjusts the prediction of the model and minimizes the prediction error of the model. Fig. 5 shows that the calibrated Shell fatigue model provides unbiased predictions with no consistent overestimation or underestimation as most of the data points are clustered around the line of equality.

Validation of the Calibrated Model

In order to check the validity of the model, another set of four beams were tested for fatigue at Downer NZ contractor's laboratory.

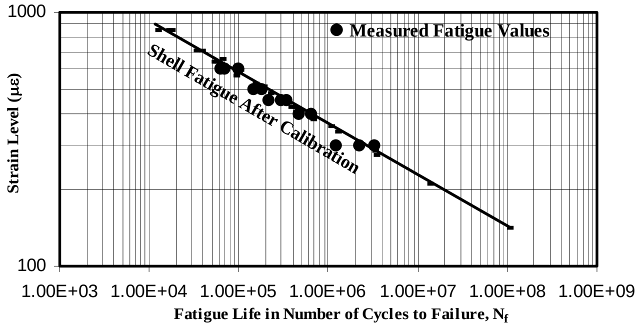


Fig. 4. Fatigue Lives as Predicted by Calibrated Shell Model versus Measured Fatigue Values.

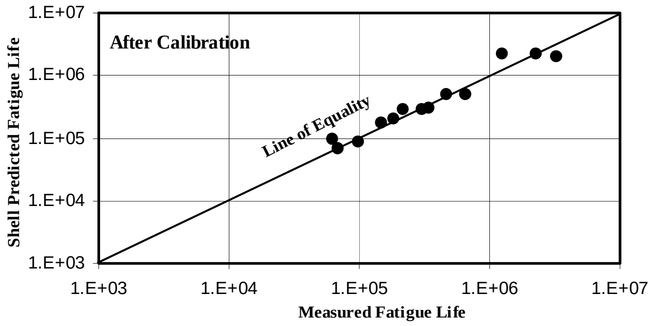


Fig. 5. Calibrated Shell Predicted Fatigue Lives versus Measured Fatigue Values.

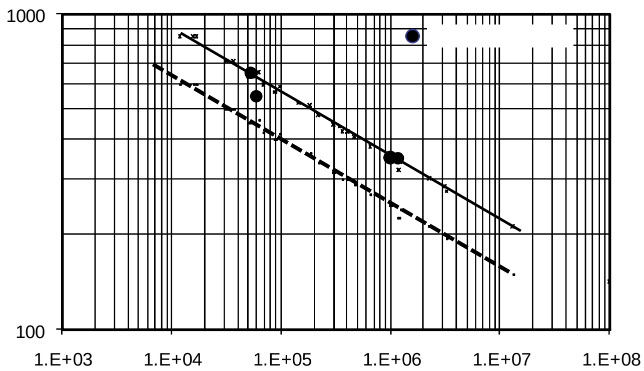


Fig. 6. Validation of the Calibrated Shell Fatigue Model.

Using another laboratory and different operator to test fatigue values at different strain levels that were not originally used in the calibration model will provide a good validation of the calibrated model. Two beams were tested at 350 με and the third beam at was tested 550 με while the fourth beam was tested at 650 με. All beams were tested using constant strain mode at 20℃. The fatigue results of the four beams are shown on Fig. 6 relative to the calibrated and the original Shell fatigue model. From Fig. 6, it can be seen that the calibrated Shell model provides a better match to the measured fatigue values compared to the non-calibrated model. Despite the fact that, only four fatigue measurements are quite small size of data to provide a robust validation for the model, however, the methodology is quite valid. With the collaboration with the Transportation industry, more data can be generated to provide a more rigorous calibration and validation. In addition, it should be noted that only one type of mix, namely AC14 was used in this study. Therefore, the calibration and validation is only applicable for this type of mix. For other types of mixes such as stone mastic asphalts (SMA) or polymer modified asphalts, more testing will be required and separate calibration factors can be determined to better simulate the fatigue behavior of these mixes.

Implications of Using Calibrated Model

Case Study from Christchurch Southern Motorway

The following data are taken from some sections of the Christchurch Southern Motorway, CSM. The Christchurch Southern Motorway is a short motorway that forms part of State Highway 73 in the southwest of Christchurch, New Zealand, see Fig. 7. The CSM is a strategic arterial route and it is expected to provide a key link to the port of Lyttelton in Christchurch. The Austroads mechanistic empirical pavement design was used in the initial design for the structural pavement cross section [1]. In the following paragraphs, only two sections of the CSM will be designed using the current Austroads guidelines and then they will be redesigned with the Austroads after calibrating the Shell fatigue transfer function. The significance of the calibration on the final design will be discussed.

Case Study I: The Motorway Section from Springs Rd to Carrs Rd

Design Inputs

Average Annual Daily Traffic, AADT = 21600 vehicle per day for the two directions; Percentage of Heavy Vehicles in the traffic stream, % HV = 10.0%; The Annual Traffic Growth Rate (r) = 3.0%; Design Period (n) = 25 Years; The average number of heavy vehicles axle groups per heavy vehicle (NHVAG) = 2.4; and Subgrade soil is sandy gravel with a California Bearing Ratio (CBR) of 7.0%.

Asphalt Mix Properties and Project Reliability Level

Reliability level = 95%; Asphalt content by weight of the total mix, Pb = 5.5%; Bulk specific gravity of the compacted mix, Gmb= 2.3; Specific Gravity of Asphalt, Gb = 1.05; Percentage of bitumen by volume, *Vb* = 12.0%; Poisson ratio for asphalt mix, υ = 0.35; and Asphalt resilient modulus is 3000 MPa.

The resilient modulus, *E* and Poisson ratio of base course and subgrade materials are shown in Fig. 8.

Design Traffic Calculations According to Austroads Design

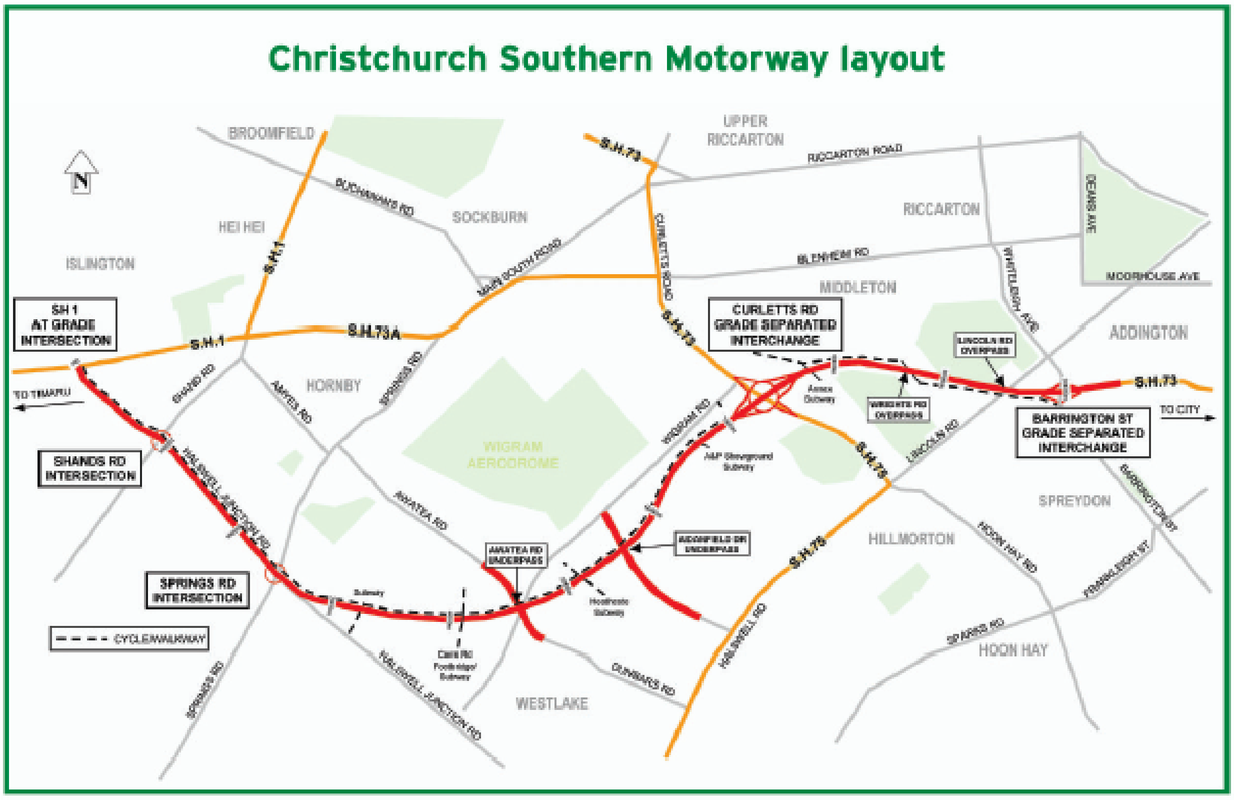
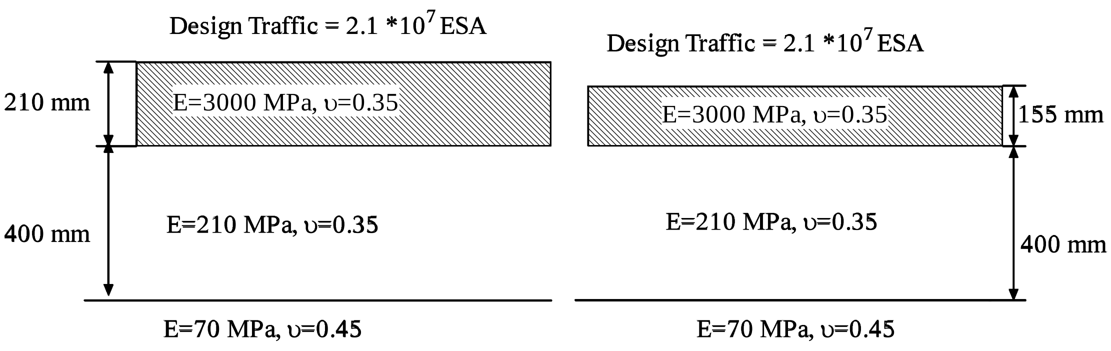


Fig. 7. Christchurch Southern Motorway.



(a) Road Cross Section Design using Current Austroads without Calibration for Shell Fatigue Function

(b) Road Cross Section Design Using Austroads with Shell Fatigue Calibration

Fig. 8. The Motorway Section Design Using: (a) Current None Calibrated Shell Fatigue; (b) Using the Calibrated Shell Fatigue Function.

$$N_{DT} = AADT * \frac{\%HV}{100} * N_{HVAG} * D * L * 365 * CGF \tag{5}$$

N_{DT} = Design traffic expressed in the number of heavy axle groups
$AADT$ = Average Annual Daily Traffic
$\% HV$ = Percentage of Heavy Vehicles
N_{HVAG} = Number of heavy vehicle axle groups per heavy vehicle
D = Directional split factor is assumed to be 0.5
L = Lane distribution factor is assumed to be 1.0
CGF = Cumulative growth factor

In the Austroads design guideline, the traffic growth is assumed to follow a geometric function from the year of construction until the end of the design period or until the road reaches its capacity whichever comes first.

$$CGF = \frac{(1+R)^{N} - 1}{R} \tag{6}$$

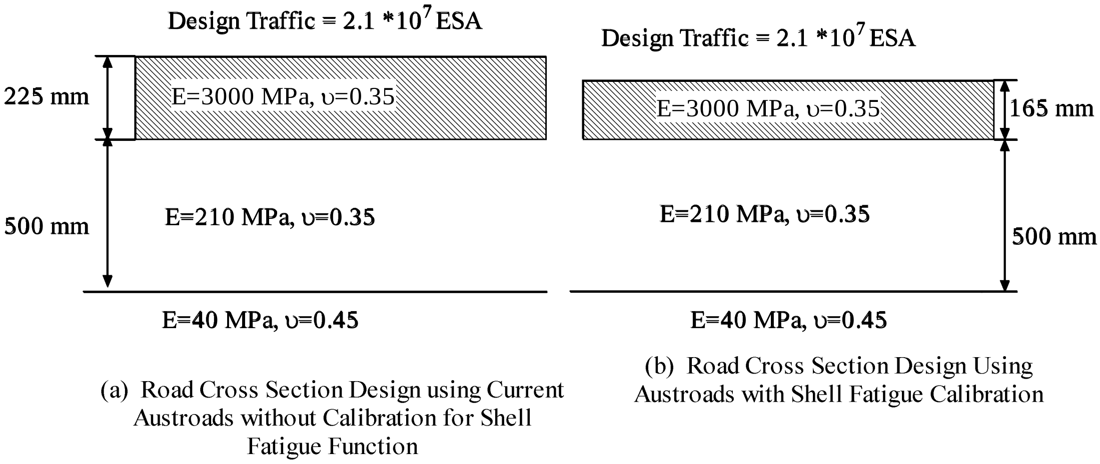


Fig. 9. The Motorway Section Design Using: (a) Current Non-calibrated Shell Fatigue; (b) Using the Calibrated Shell Fatigue Function.

where:
N = Design period
R = Traffic growth rate
CGF = Cumulative Growth factor (CGF=36.46)

$N_{DT} = 21600 * \frac{10}{100} * 2.4 * 0.5 * 1 * 365 * 36.46 = 3.449*10^7$

To convert the number of heavy vehicle axle groups to the equivalent number of standard axles, New Zealand supplement uses an average ratio between the heavy vehicle axle group and the number of standard axle of 0.6. Thus,

$N_{DT} = 3.45*10^7$ HVAG;

Design Number of Equivalent Standard Axles (DESA) = $3.45*10^7*0.6 = 2.1*10^7$ Equivalent Standard Axle (ESA).

Current Austroads Design (No Calibration)

Shell Before Calibration

$$N_f = \left(\frac{6918*(0.856*V_b + 1.08)}{E^{0.36} * \mu\varepsilon} \right)^5 \quad (7)$$

This model is expressed in the Circly software [8], which is used in the Austroads mechanistic empirical design as follows:

$$N_f = \left(\frac{\text{Constant}}{\varepsilon} \right)^5 \quad (8)$$

For asphalt with resilient modulus, E = 3000 MPa, the fatigue constant in Circly is 0.00439835.

Shell After Calibration

$$N = \left[\frac{9792*(0.856*V_b + 1.08)}{E^{0.36} * \mu\varepsilon} \right]^5 \quad (9)$$

For asphalt with resilient modulus E = 3000 MPa, the fatigue constant is 0.00622558.

Fig. 8 below shows the Austroads design for the Motorway section using both the calibrated and non-calibrated Shell fatigue function.

Case Study II: Motorway Section from Carrs Rd. to Aidanfield Dr

This part of the Motorway will be constructed on Greenfield soils which is basically a loose moist Silty Sand with very poor CBR values that ranges from as low as 1.0% to a maximum of 5.0%. Soil stabilization for at least the top 150 mm to improve both the strength and reduce frost heave potential of this soil is highly recommended. Some deep compaction to a depth of at least 1.0 m is required to provide good packing for the soil.

Design Inputs

$AADT$ = 21600; $\%HV$ = 10.0; Traffic Growth Rate = 3.0%; Design Period = 25 Years; N_{HVAG} = 2.4; Subgrade soil CBR = 4.0%; N_{DT} = $3.45*10^7$ HVAG; and DESA = $2.1*10^7$ ESA.

Fig. 9 below shows the Austroads design for the Motorway section using both the calibrated and non-calibrated Shell fatigue function. Resilient modulus and Poisson ratio for the different layers are shown in Fig. 9.

Implications of the Calibrated Model on Materials and Cost Savings

Comparing the two design sections shown in Figs. 8 and 9, it is clear that Shell fatigue before calibration overestimated the design of asphalt concrete thickness by 55 mm in first case study and 60 mm in the second case study. By looking to the ratio between the thickness required using the calibrated Shell model to that of the current Shell without calibration (155 mm / 210 mm = 0.74 and 165 mm / 225 mm = 0.733) which is very much close to the statement by NZ supplement that the thickness design of the asphalt pavements using the old State Highway design manual is 30% thinner than that is required by the current Austroads design method. This is a significant amount of asphalt which can have significant implications of the project cost. By looking to the quantity of asphalt that can be saved in just one lane of 3.5 m width and one kilometer length of the road, one can appreciate the impact of using a well calibrated performance model.

Quantity saving per one lane width and one kilometer length:

- Saved Quantity of Asphalt in case I = 55/1000*3.5*1000*2.3 = 442.75 tonnes;
- Saved Quantity of Asphalt in case II = 60/1000*3.5*1000*2.3 = 483 tonnes;
- For an average of $150 per tonne; and total cost saving/lane-kilometer ranges from $66,413 to $72,450.

This is basically a substantial amount of savings that can affect the choice of pavement alternatives. With the current underestimation of the fatigue life of asphalt mixes by the Shell fatigue model, the use of structural asphalt could be easily rendered as an uneconomical option and some other cheaper options could be selected based on the current Austroads design method.

Conclusions

Fatigue tests were carried out on seventeen asphalt concrete beams made with AC14 dense graded asphalt mix and using 60/70 penetration grade bitumen. The seventeen beams were tested using constant strain mode for different strain levels range from 300 to 650 $\mu\varepsilon$. Thirteen beams were tested in the University of Canterbury, Transportation Laboratory and the other four beams were tested in the Downer NZ contractor laboratory. Comparing the Shell predicted fatigue lives with the actual measured fatigue lives clearly demonstrates that the Shell model consistently underestimates the fatigue lives of the AC14 asphalt mixes. A calibration factor was derived from the measured and predicted fatigue lives for the thirteen beams at different strain levels. The calibration factor was derived in order to minimize the total prediction error. A calibration factor of 5.6824 was computed from the measured and predicted fatigue values. The calibrated model provides unbiased prediction with no consistent overestimation or underestimation. The calibrated model was validated with a limited set of four fatigue tests that were carried out at the Downer NZ laboratory. The calibrated model provides a better match compared to the non-calibrated model. In addition to this laboratory calibration to the Shell model, field calibration will still be needed to provide a more accurate match with the fatigue values in the field. The calibrated model was used to design two sections of the Christchurch Southern Motorway to examine the impact of using a calibrated model on the materials and cost savings. The calibrated Shell fatigue model produced about 26% to 27% thinner asphalt layers compared to the original non-calibrated Shell Model. This means that using a calibrated and validated transfer functions can significantly impact the design values and therefore could impact the decision making in selecting certain design alternatives.

Acknowledgment

I would like to acknowledge and thank the great effort and support from Mr. Howard Jeffery-Wright, Mr. Simeon Hall, Mr. Harrison Jia and Dr. David Hutchison from Downer NZ. I would like also to thank Mr. Anthony Stubbs for his great effort in testing and compiling the fatigue data at the University of Canterbury Transportation laboratory.

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