

## TECHNICAL PAPER

# INVESTIGATION INTO THE VALIDATION OF THE SHELL FATIGUE TRANSFER FUNCTION

### Authors:

**Anthony Stubbs (Presenter)**, BE(Hons), Masters student, University of Canterbury.  
aps49@student.canterbury.ac.nz.

**Mofreh Saleh**, Ph.D., P.E., Senior Lecturer, University of Canterbury,  
mofreh.saleh@canterbury.ac.nz

**Howard Jeffery-Wright** BSc, Regional Laboratory Manager for Downer EDi Works  
howard.jeffery-wright@downerediworks.co.nz

### ABSTRACT

In the structural design of highways, asphalt concrete layers are designed to withstand fatigue cracking caused by the repetitive flexing under traffic loading. The current Austroads design guidelines for predicting fatigue cracking in the asphalt concrete layers is the Shell Fatigue Transfer Function. However, this model often under or overestimates the asphalt thickness required of New Zealand structural roads. The research investigates the suitability of the Shell model and examines alternatives that would be more appropriate for a typical New Zealand dense graded hot mix asphalt. In addition, the research verifies the discrepancies of the Shell fatigue model. Thirteen beam fatigue tests were carried out using a four point flexural test under a constant strain mode. Different constant strain levels were applied until failure. The measured fatigue life was also compared against the Shell Fatigue Transfer Function and alternative prediction models. The results indicate that the Shell model underestimates the fatigue life of this particular asphalt mix by an average factor of 5.5 (range 3.1—8.9). A new fatigue model is proposed which would provide the New Zealand roading industry with both greater precision and cost savings when predicting fatigue cracking for this asphalt mix.

## **INTRODUCTION**

Fatigue cracking is a common mode of failure on all asphalt roads. This is caused by the repetitive stresses and strains felt by either the top or the bottom of the asphalt layer due to its flexing under traffic loading. In the mechanistic design approach, the tensile strain at the bottom of the asphalt layer governs the required asphalt thickness needed to mitigate fatigue cracking. Currently, the Austroads design guidelines have adopted the Shell Fatigue Transfer Function to predict fatigue cracking for structural asphalt roads. However, within the New Zealand roading industry there is an uncertainty in the appropriateness of this transfer function in predicting asphalt's fatigue life. Moreover, it is commonly observed that this function often over or underestimates the required asphalt thickness in the design of fatigue cracking. Therefore, the objective of this research is to verify this design guideline; using a typical New Zealand densely graded hot mix asphalt (HMA) namely, asphalt concrete (AC) 14 ( i.e the maximum nominal aggregate size is 14 mm) using bitumen class 60/70 penetration grade.

Prediction models for this NZ HMA are presented to provide an alternative fatigue model and two case studies to determine the significance of the alternative models. In addition, the project compares the amount of savings in NZ dollars between the developed fatigue model and the Shell Fatigue Transfer Function for the AC 14 B60/70 HMA.

### **Asphalt Fatigue Characteristics**

Fatigue cracking in flexible pavements is caused by the repetitive stresses and strains due to traffic loading. Cracking occurs when the applied horizontal tensile strain repetitions exceeds the fatigue life capacity of the asphalt layer. Once the asphalt has reached a defined level of cracking it is said to have reached its fatigue life. The fatigue life of asphalt is influenced by a variety of factors, including: the underlying stiffness of the pavement structure; traffic loading spectrum; environmental factors; construction variables; and material characteristics (Austroads, 2004; Baburamani, 1999). While the aim of the project is to compare the laboratory developed Shell fatigue function with experimental testing, the review will only focus on parameters that affect fatigue in the laboratory. An important factor that influences the fatigue life of asphalt is its stiffness. Moreover, the stiffness of asphalt is a product of the bitumen type and content; amount of air voids within the mix; temperature and frequency of loading; and the type of aggregate and its gradation within the mix.

Loading conditions are another factor affecting the laboratory fatigue life of asphalt. Notably, there are several different testing methods used to measure the fatigue life of asphalt that aim to replicate the pavement response in field conditions. These different testing setups include: bending testing (three and four point loading schemes), indirect tensile testing, direct tensile testing. Flexural fatigue testing is the preferred Australian procedure as it is said to reproduce the actual behaviour of an asphalt layer under wheel loading more closely than any another method (Austroads, 2004). Among these various loading setups, there are also two types of controlled loading modes: controlled strain and controlled stress.

Controlled strain (displacement) testing is defined by maintaining a constant deformation during cyclic loading throughout the test. In the strain controlled test, the load decreases during the test to keep a constant deformation. Cracks initiate at the bottom of the asphalt specimen and propagates with the cyclic loading . As the number of cycles increase, the

flexural modulus of the sample beam decreases. In the controlled strain testing there is no clear failure, therefore, failure is often defined when the flexure modulus reduces to a 50 percent of its original value. (Saleh, 2003). On the other hand, controlled stress (force) is achieved by maintaining a constant stress loading throughout the test. In this case, the deformation increases during the test as a result of cracking; hence, failure is defined when the specimen fractures. In the literature, controlled displacement testing is said to be more applicable for relatively thin asphalt pavements (less than 100 mm thick), however, controlled stress testing is more relevant for thick pavements. Usually models developed based on controlled strain testing provides longer fatigue lives compared to those based on controlled stress. However, it is also said that for pavements with a greater flexibility, that is, lower stiffness; controlled strain testing has a more superior fatigue performance with similar initial strain amplitudes (Baburamani, 1999).

Furthermore, laboratory fatigue testing methods do not exactly replicate the same state of stresses that are felt in the field. Baburamani (1999) stated that these discrepancies between the field and the laboratory are due differences in the loading setups (axle loads and traffic setup position); establishing realistic loading times and rest periods between traffic loading; the surrounding temperature during the pavement service life; and the level of compaction of the asphalt. Because of these differences, a calibration factor commonly called shift factor is applied to a laboratory fatigue model to give a better estimate of the fatigue life in the field.

## **Shell Fatigue Transfer Function**

The Austroads pavement design guidelines have adopted the Shell Fatigue Transfer Function as the fatigue life prediction model. The Austroads guidelines require the use of a reliability factor to account for the uncertainty of the different variables in the model. According to the Austroads Pavement Design (2004) 'the probability that the pavement, when constructed to the chosen design, will outlast its Design Traffic before major rehabilitation is required.' The reliability factor is not a shift factor that takes into account in-service field conditions. The Shell fatigue prediction model was developed by Shell researchers in 1978 from 'a broad range of mix types containing conventional binders' (Austroads, 2004). In addition, Baburamani (1999) stated 'this fatigue relationship was based on laboratory-controlled strain sinusoidal loading fatigue testing on several (13) typical asphalt mixes used in various countries.' It can be noted that the exact origins of the types of the mixes involved are unknown in the literature; however, 'various basecourse and wearing course mixes' were used (Baburamani, 1999). Given this model was developed in the laboratory, a shift factor needs to be applied to the Shell Fatigue Transfer Function to account for field conditions. The Shell Fatigue Transfer Function is defined as:

$$N_f = RF \left[ \frac{6918(0.856V_B + 1.08)}{S_{mix}^{0.36} \mu\epsilon} \right]^5 \quad \text{Equation 1}$$

where:  $N_f$  is the allowable number of load repetitions for a given strain level;  $\mu\epsilon$  is the tensile strain produced by the load (micro strains);  $V_B$  is the percentage by volume of effective bitumen in the asphalt;  $S_{mix}$  is the mix stiffness modulus (MPa); and RF is the reliability factor depending on the required level of reliability.

## **Field Data**

Notably, the New Zealand Supplement Pavement Design (2007) states asphalt layer thicknesses using the earlier State Highway Pavement and Rehabilitation Design Manual provide 30 percent thinner pavements than those required using the current Austroads Guide. Furthermore, it states that two thirds of the Wellington and Auckland motorway network are constructed with structural asphalt having being designed using the State Highway Pavement and Rehabilitation Design Manual and are performing well past their design lives with minimal structural maintenance required. This field information clearly demonstrates that a thinner pavement can support a higher number of loadings without fatigue failure. Meaning that that the current Austroads design guidelines for predicting fatigue cracking overestimates the structural design thickness of these New Zealand roads.

## Prediction Models

Empirical relationships are commonly employed to model the fatigue performance of asphalt, characterised by a power law. Conventionally the strain (phenomenological) approach, Equation 2 or the strain and stiffness approach, Equation 3 is used to predict fatigue life and these are expressed in the following form:

$$N_f = a\varepsilon_t^{-b} \quad \text{Equation 2}$$

$$N_f = a\varepsilon_t^{-b} E^{-c} \quad \text{Equation 3}$$

where:  $N_f$  is the allowable number of load repetitions to prevent fatigue cracking;  $\varepsilon_t$  is the tensile strain at the bottom of the asphalt layer;  $E$  is the elastic modulus of asphalt mix; and  $a$ ,  $b$ , and  $c$  are material constants determined by regression analysis. Volumetric properties are also used to characterise fatigue life because binder content and air voids have an influencing role on asphalt performance. However, Baburamani (1999) states the effect of increasing binder content on fatigue performance is dependent on the mode of loading during fatigue testing. For example, in strain-controlled mode, a greater binder content would yield a greater fatigue performance.

Paris' Law is another method to determine the number of loading cycles until fatigue is reached. This is based on fracture mechanics. According to Baburamani (1999) 'Paris's law of crack propagation relates the increase in crack length per load cycle to the stress intensity factor'. However, Pellinen *et la* (2004) suggests that the 'stresses and strains surrounding a crack during propagation are difficult to define precisely'.

## MATERIALS AND METHOD

### Material, mix design and volumetric properties

Thirteen AC 14 B60/70 HMA beams were prepared to certain volumetric properties by Downer EDi Works. The notation AC 14 B60/70 denotes the use of asphalt concrete mix (AC) with a maximum nominal aggregate size of 14 mm for the gradation. B60/70 is the grade of the bitumen in the mix and is determined by the penetration test. Basalt rock was used as the aggregate in this project and was sourced from Bombay Quarry, Auckland, and the bitumen was supplied from Mount Maunganui. The volumetric properties of the prepared samples are similar to one another and are shown in Table 1, below. It can be mentioned that

the small variation of air voids between these samples, proved to have little influence over the fatigue life of this asphaltic concrete mix.

**Table 1 – Beam specimen volumetric properties**

Beam Number	Theoretical Maximum Density	Bulk Density	Air Voids (%)	Bitumen Content by mass (%)	Bitumen Content by Volume (%)
1	2.652	2.513	5.24	5.02	12.37
2	2.652	2.534	4.45	5.02	12.47
3	2.652	2.531	4.56	5.02	12.45
4	2.652	2.547	3.96	5.02	12.53
5	2.652	2.494	5.96	5.02	12.27
6	2.652	2.489	6.15	5.02	12.25
7	2.652	2.495	5.92	5.02	12.28
8	2.652	2.49	6.11	5.02	12.25
9	2.652	2.478	6.56	5.02	12.19
10	2.652	2.477	6.60	5.02	12.19
11	2.652	2.481	6.45	5.02	12.21
12	2.652	2.484	6.33	5.02	12.22
13	2.652	2.493	6.00	5.02	12.27

### Fatigue Testing

Fatigue testing was carried out by a flexural test using a four point loading scheme with the IPC global universal testing machine (UTM) model UTM 21 Stand-Alone Fatigue Apparatus. Testing was carried out in a controlled temperature cabinet at a surrounding temperature of 20°C and with a haversine loading pulse at a frequency of 10 Hz. Tests were conducted using a controlled strain (displacement) mode. Five different constant strain (displacement) levels were applied ranging from 300 micro strains to 600 micro strains. For strain levels at 300, 450 and 600 micro strains three tests were carried out and for strain levels 400 and 500 micro strains two tests were performed; this ensured that there was reliability within the results. Fatigue failure was defined as a 50 percent reduction in the initial flexural stiffness of each beam. The initial stiffness was determined at the end of the fiftieth loading cycle by the UTM.

### Prediction Models

Fatigue prediction models were developed for the AC 14 B60/70 HMA using the fatigue testing results with the non-linear regression analysis package, DATAFIT 9. For each developed model, the measured fatigue life in the laboratory was used as the dependent variable and compared against important independent variables: amplitude of applied constant strain; initial measured flexural stiffness of the mix; initial measured elastic modulus of the mix; percentage of bitumen by volume within the mix; and the percentage of air voids. These prediction models were also plotted against the measured fatigue life juxtaposed with a line of equality.

### Case Studies

The purpose of the case studies was to determine the required asphalt thickness for a typical New Zealand roading cross section, and ultimately to determine the savings difference in dollars between the developed strain approach fatigue model and the Shell Fatigue Transfer

Function. Where CIRCLY 5 was used in the analysis of each case study. Circlly is a software program used to design pavements based on the empirical mechanistic design. To calculate the asphalt concrete thickness, a fatigue performance criterion was defined. For each case study the Shell Fatigue Transfer Function was compared with the different developed fatigue models. The difference in the required asphalt thickness between these two models shows the material savings. Subsequently, a monetary value of the total savings for a one-lane, one-kilometer section of road could be estimated, incorporating direct and indirect costs, material and labour costs, and a markup.

Each case study was constructed with three different layer types: sub-base course, base-course and an asphalt layer. In Case Study One, the base course was 300 mm thick and in Case Study Two, the base course was 500 mm thick. For both studies the modulus of elasticity of the asphalt layer was 3000 MPa with a Poisson ratio of 0.35, the modulus of elasticity for the base course was 300 MPa with a Poisson ratio of 0.35, and the modulus of elasticity of the sub-grade is 30MPa with a Poisson ratio of 0.40. Both case studies are illustrated in Figure 1 below. In the analysis for both case studies, the asphalt was modelled as an isotropic material and both the sub-base and sub-grade were modelled as cross-anisotropic. In addition, 100 million equivalent standard axles (ESA) were applied to each pavement cross-section. To quantify the saving in dollars it was assumed that the pavement system had a bulk density of 2.5 tonne per cubic meter and an indicative cost of \$200 per tonne, which includes production and paving costs, with a realistic gross margin percentage.

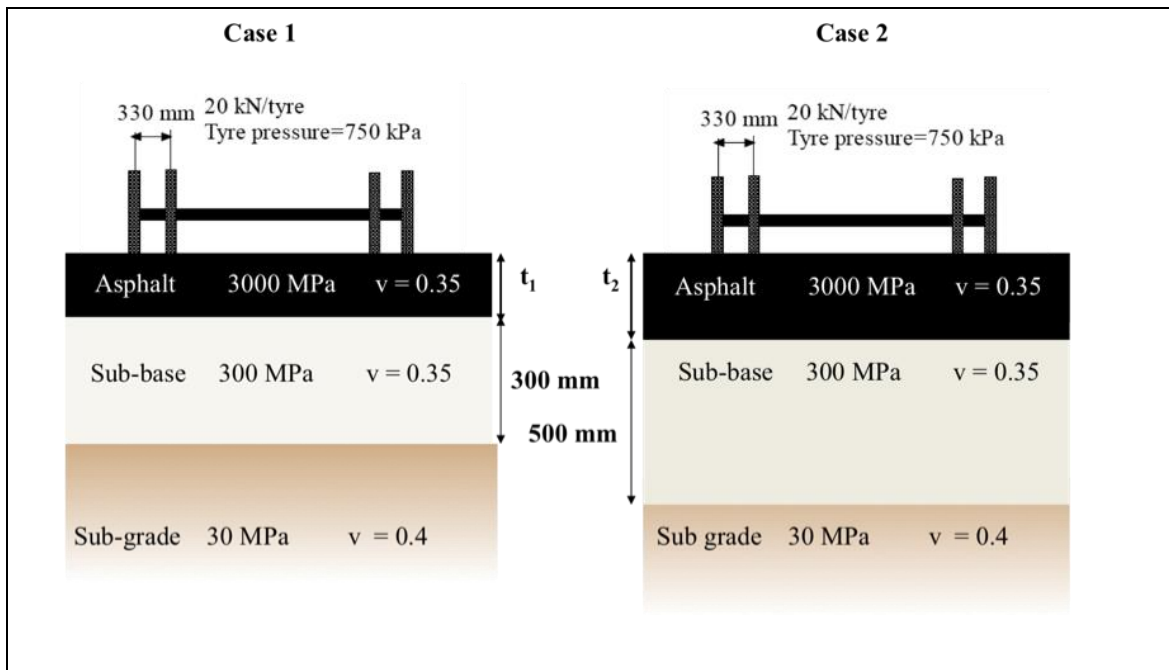


Figure 1 Cross section of a typical roading section of a pavement

## RESULTS AND DISCUSSION

### Fatigue Testing

Findings clearly indicate that the Shell Fatigue Transfer Function under estimates the fatigue life of the AC 14 B60/70 HMA. Figure 2 below, illustrates the relationship of the amplitude of applied strain versus the fatigue life behaviour for both the experimental fatigue testing

results and the laboratory based Shell Fatigue Transfer Function. In addition, the figure also shows the discrepancies between the two. This arises from the different binder types, aggregates, and design volumetrics used in the AC 14 B60/70 and those used in the development of the Shell fatigue prediction model. At 400 micro strains, the Shell function estimates the fatigue life of this mix to be 180 000 cycles. However, the experimental testing suggest that the fatigue life of this mix could be between 466 000–650 000 cycles. Hence the AC 14 B60/70 HMA has a greater life span than the predicted Austroads design guidelines.

Furthermore, table 2 below, compares the calculated number of loading cycles until failure from the Shell function with the measured results for each beam specimen. The table also shows the ratio of the number of loading cycles until fatigue failure to the number of loading cycles predicted by the Shell model for each beam. Overall, for the thirteen beams tested, the fatigue life is on average 5.5 times greater than the predicted fatigue life by the Shell Fatigue Transfer Function. Notably, this ratio is between 3.1 – 8.9 times. Concluding, that the Shell Fatigue Transfer Function underestimates that fatigue life of this HMA.

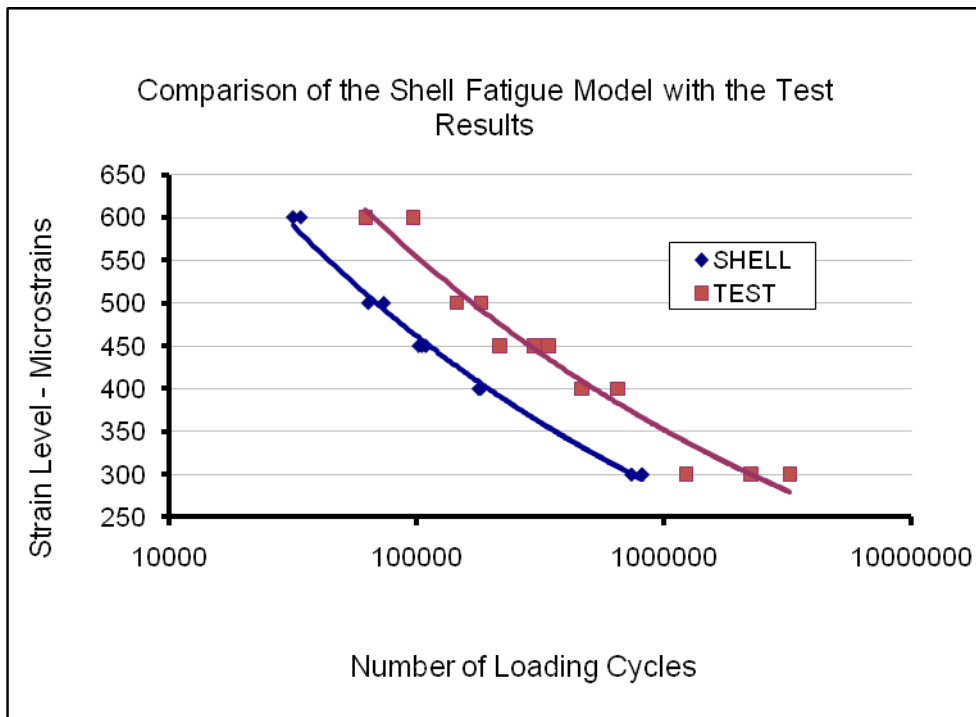


Figure 2 - Comparison of the Shell function with the experimental results

**Table 2 - The number of loading repetitions until fatigue failure of the Shell model and the measure results, and the ratio of the two.**

Beam Number	N <sub>Shell</sub>	N <sub>Test</sub>	Ratio
1	87702	649490	7.41
2	89436	466150	5.21
3	36363	182820	5.03
4	398986	1230550	3.08
5	15664	97300	6.21
6	31487	145420	4.62
7	51804	217710	4.20
8	402209	2248110	5.59
9	16824	62260	3.70
10	363458	3235560	8.90
11	53765	342000	6.36
12	50333	299600	5.95
13	11829	68230	5.77
	<b>Average Ratio</b>		<b>5.54</b>

## Prediction Models

Three different alternative models to predict the fatigue life of the AC 14 B60/70 HMA are presented in this section: strain approach; strain and flexural stiffness; and strain and resilient modulus. The latter is presented because the NZ roading industry will commonly measure the resilient modulus with the in-direct tensile test instead of the flexural stiffness. All these models can be used in the Austroads empirical mechanistic design using CIRCLY 5. However, a field shift factor (FSF) has not yet been calibrated for this asphalt mix.

### Strain Approach (Phenomenological) Model

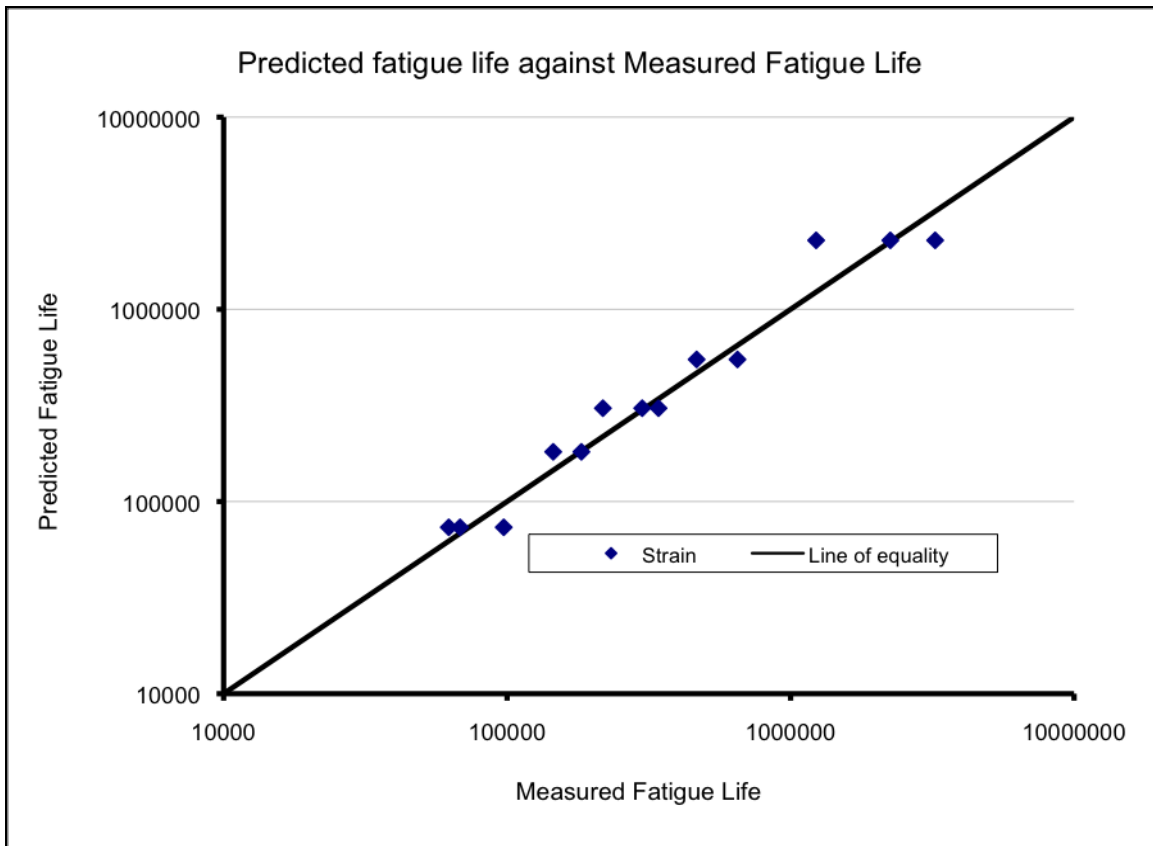
For the AC 14 B60/70 HMA, the strain approach model is given by equation 4:

$$N_f = 4.421 \times 10^{18} \varepsilon_t^{-4.96}$$

**Equation 4**

where  $N_f$  is number of loading cycles to fatigue failure in the laboratory;  $\varepsilon_t$  is the tensile strain in microstrains. This relationship was found to have correlation coefficient of determination,  $R^2$  value of 0.82 for the applied strain in the laboratory and the number of measured loadings cycles until fatigue failure. Figure 3 below plots predicted fatigue life against the measured results and is juxtaposed with a line of equality to assess the goodness of fit of the model; the graph shows that this model is unbiased against the measured fatigue life because the scatter of points is fairly evenly distributed about this line. Hence, indicating that Equation 4 is an excellent model for the data. The goodness of the fit between the predicted and measured

fatigue life is 0.91. It can be noted, that this phenomenological approach is used as the fatigue criterion model for the case studies.



**Figure 3 –The strain approach model against the experimental results**

**Strain–Flexural Stiffness Approach Model**

Equation 5 below, expresses the fatigue failure of this asphalt mix as a function of both the applied laboratory tensile strain and the initial flexural stiffness of the mix.

$$N_f = 3.91852 \times 10^{25} \epsilon_t^{-4.94995} E^{-1.91757} \tag{Equation 5}$$

where,  $N_f$  is the number of cycles to fatigue failure in the laboratory;  $\epsilon_t$  is the tensile strain in microstrains; and  $E$  is the initial flexural stiffness (MPa). This relationship has a value of  $R^2$  of 0.83 between the functional variables: strain and stiffness with the measured fatigue life. Figure 4 shows predicted fatigue life of Equation 5 plotted against the measured fatigue life and this scatter is mapped against a line of equality between the two parameters. Figure 5 shows that this model is unbiased because the scatter of points is evenly distributed about the 45° line. This strain-flexural stiffness approach model has a correlation of 0.91 compared against the experimental fatigue results.

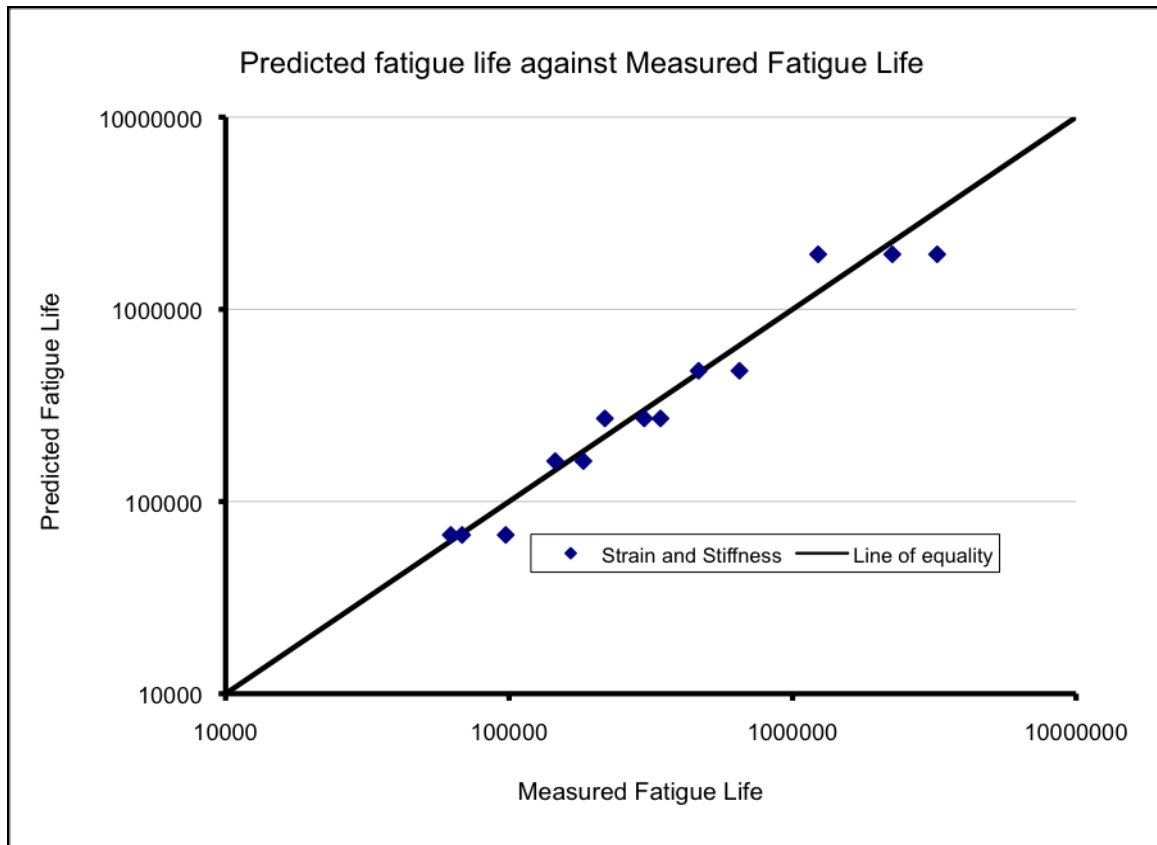


Figure 4 -Strain and flexural stiffness approach model against the measured results

Strain–Elastic Modulus Approach Model

A further function modelling the fatigue behaviour of the asphalt based on the applied strain and the elastic modulus of the asphalt mix was also developed.

$$N_f = 2.2665 \epsilon_i^{2.6} E^{-4.963} \sigma^{-2.10} \tag{Equation 6}$$

Equation 6, provides an alternative to predicting the fatigue of the asphalt mix based on the elastic resilient modulus instead of the flexural stiffness, this was done as often the modulus of the asphalt is given as the resilient modulus and is also an output of the indirect modulus test. The variables within this relationship were found to have a correlation coefficient,  $R^2$  of 0.83. Figure 5 below plots the predicted results of equation 6 against measured fatigue life. This elastic modulus model appears to slightly overestimate the measured fatigue life of the AC 14 B60/70 HMA because the scatter data is slightly above the line of equality as shown in Figure 5. However, this over estimation is reasonable because the measured data is from a flexural fatigue test and the elastic modulus is always greater than the flexural stiffness when conducting a flexural test.

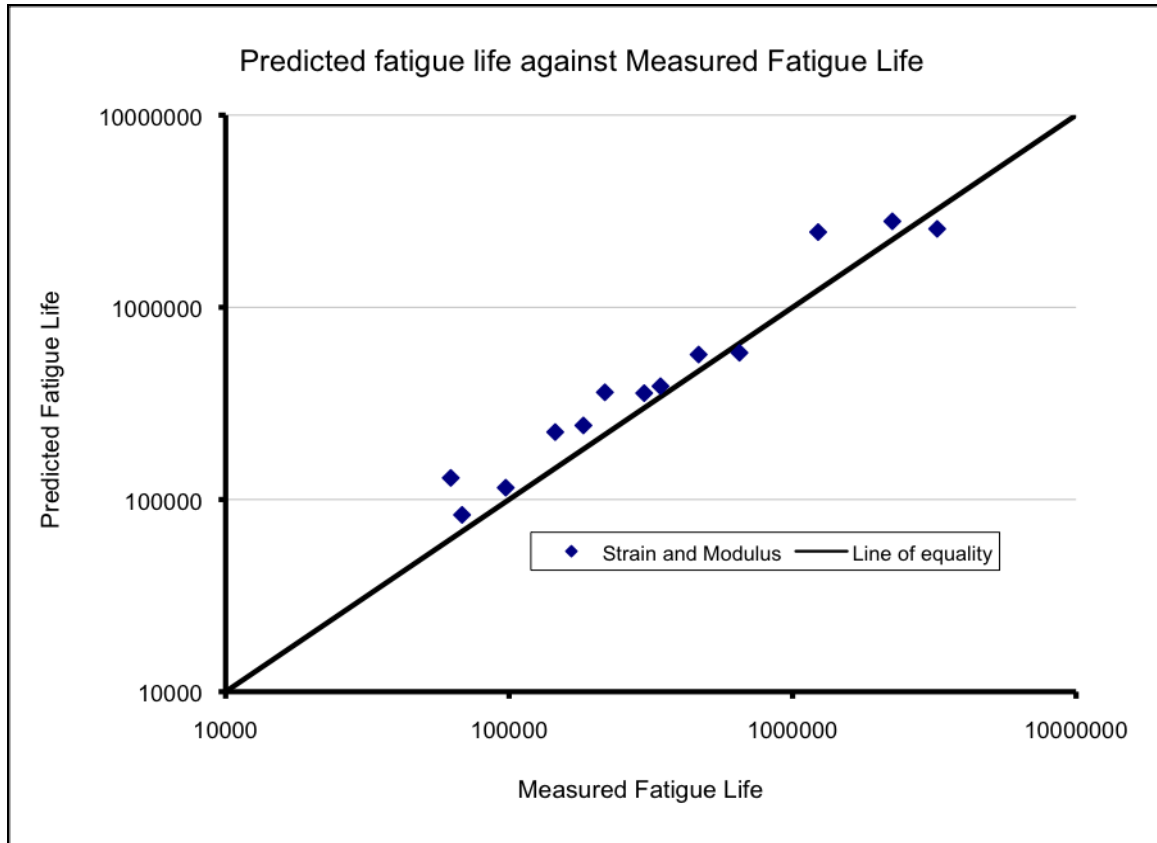


Figure 5 - Strain and elastic modulus approach against the measured fatigue life

### Case Studies

Each case study showed that significant cost savings could be made. This was found when equation 4 was defined as the fatigue cracking performance criterion instead of the Shell Fatigue Transfer Function for the AC 14 B60/70 HMA. The analysis demonstrates that for Case Study One and Case Study Two the estimated savings was \$87,500 and \$96,250 per lane-kilometre respectively. This is a considerable amount. Furthermore, as both of these models were developed under laboratory conditions, which are known to be more conservative than field conditions, there is a further inherent over estimation by both models for field conditions. Consequently, even greater savings might be made after field calibration of these models.

**Table 3 -Results of case study 1**

	Shell Function	Strain Model	Material Savings	Cost Savings
Asphalt Thickness	325 mm	275 mm	50 mm	\$87,500

**Table 4 -Results of case study 1**

	Shell Function	Strain Model	Material Savings	Cost Savings
Asphalt Thickness	290 mm	235 mm	55 mm	\$96,250

In addition, the material savings of Case Study One and Case Study Two are 15 percent thinner and 23 percent thinner respectively. This savings is the amount of material difference between the Shell Fatigue Transfer Function and the strain mode when they are both used as the fatigue failure criterion. As mentioned above, two thirds of the Wellington and Auckland motorway network were designed with the State Highway Pavement and Rehabilitation Design Manual. These roads were 30 percent thinner than the Shell design method (New Zealand Supplement Pavement Design, 2007). Thus, validating that in the field thinner and less expensive roads can be constructed. Therefore these cost savings are realistic.

## **CONCLUSIONS**

These findings demonstrate that the Shell Fatigue Transfer Function underestimates the fatigue life of the common New Zealand roading asphalt mix, AC14 B60/70 HMA. The underestimation is by an average factor of 5.5 (range 3.1—8.9). These results were obtained under a range of specific laboratory conditions with constant strain levels between 300 micro strains and 600 micro strains. In addition, tests were conducted at a surrounding temperature of 20°C and a loading frequency of 10 Hz. The alternative model, using the Austroads mechanistic empirical design method, gives a more accurate prediction of the fatigue life of this particular asphalt mix and proves to be more cost effective. For example, In Case Study One, employing an analysis with CIRCLY 5 for a typical roading section see (Figure 1), the alternative strain approach model would yield savings of \$87,500 per lane-kilometer.

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