

Evaluation of the Linear Damage Assumption in JPCP Bottom-up Fatigue Cracking

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Abstract

Current pavement design methods for Jointed Plain Concrete Pavements assume that bottom-up fatigue cracking occurs at one critical location at the mid-slab edge and that damage accumulates linearly and can be described via S-N curves. More recently, experiments have shown that fatigue does not accumulate linearly. This assumption will lead to an accumulation of prediction error which depends on the sequence of applied loads. In this paper, a modified Paris fatigue equation is used to simulate mid-slab edge fatigue damage. The fatigue equation accounts for the transient and steady state stages of cracking in addition to the peak and stress range effects. An interpolation scheme was used to determine the pavement stresses at the bottom mid-slab edge. The interpolation scheme was constructed from a total of 35,000+ FEM runs in EVERFE. Intra-axle peak and valley stresses were accounted for so load history and R-ratio effects could be assessed simultaneously. Two different climatic regions were considered: Miami, FL and Lansing, MI. The results of this study suggest that the dominant stress and R ratio fall between 0 and 0.1 for both climates. In addition, it seems that although high stress ratios (0.7-0.9) occur infrequently (less than 1% for both cases), the life of the pavement is strongly correlated with high stress ratio events. Finally, it is shown that damage may be under-predicted using a linear damage rule because it does not account for the over-load effect. This under-prediction can be corrected by re-calibrating the percent slabs cracked equation.

Introduction

Concrete pavements are subjected to a complex combination of environmental and traffic loads which produce a unique distribution of peak stress and stress ranges at the critical mid-slab edge. Moreover, the fatigue damage accumulation process is complex because the material is heterogeneous and suffers from a variety of pre-loading flaws such as air voids and shrinkage cracks leading to a substantial variability in fatigue performance.

There are several different ways of predicting fatigue damage in concrete. The classical method is to use a linear damage rule (LDR) accompanied with an experimentally generated log-log S-N curve that relates applied stress ratio (SR) to the number of cycles to failure in fatigue, N_f (Miner, 1945). Note that SR is the ratio between the applied peak stress and the maximum allowable stress ($\sigma/\sigma_{\max}$). Figure 1 displays a series of typical S-N curves which show that N_f will increase as the stress ratio decreases (Chatti et. al., 2008). The solid lines represent a least squares linear regression which is described mathematically using equation (1).

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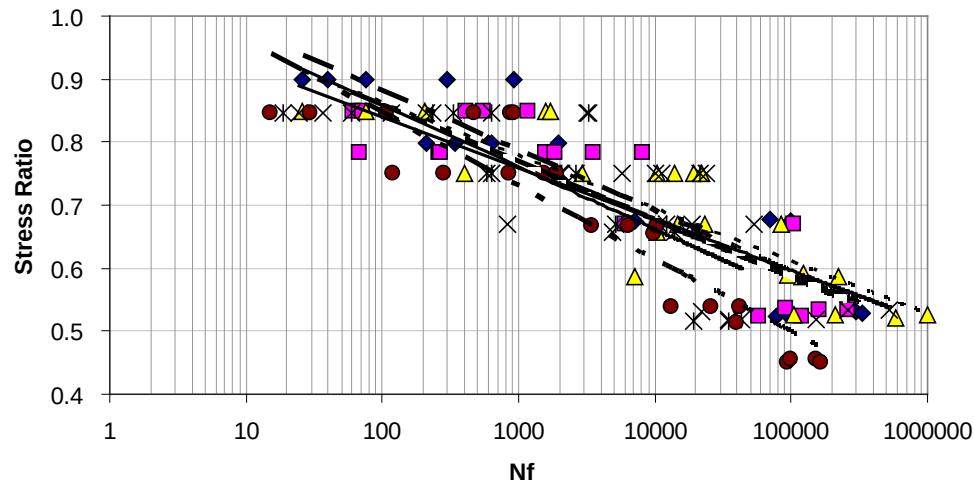


Figure 1: S-N curve

$$\log\left(\frac{\sigma}{\sigma_{max}}\right) = \beta_1 + \beta_2 \log(N_f) \quad (1)$$

An LDR can then be used to determine the cumulative damage by using equation (2), where damage (D) is defined as $1/N_f$, and failure occurs when $D = 1$ and n represents the total number of applied loading cycles.

$$D_i = \sum_{i=1}^n \frac{1}{N_{f_i}} \quad (2)$$

Concrete Fatigue Tests

Over the last few decades, many concrete fatigue studies have been conducted for various concrete mix types and loading scenarios (Murdock and Kesler, 1959; Tepfers and Kutti, 1979; Raithby, 1979; Hsu, 1981; Zhang and Phillips, 1996). In summary, the studies show that fatigue life is dependent on strength, age, drying shrinkage, loading frequency, SR, and R ratio (ratio of applied valley and peak load). In the Metal industry, Goodman diagrams (or constant life diagrams) have been used extensively as a means to establish the relationship between the endurance limit (stress level at which fatigue damage becomes negligible), peak stress and stress range (Sendekyj, 2001). Other researchers, as a means to acknowledge the large variability in fatigue, used a reliability approach to predict fatigue damage accumulation (Park, 1990; Oh, 1991; Singh and Kaushik, 2001). In general, this was accomplished by inserting an experimentally observed N_f probability density function (pdf) into the LDR to express N_f in terms of a probability.

Concrete Pavement Cracking Prediction

The current design methodology (NCHRP, 2004) for transverse cracking uses a hybrid mechanical and empirical analysis. The stresses of an un-cracked pavement are predicted via an artificial neural network that is trained by an extensive Finite Element response database. The stresses output from this network are then inserted into a field inspired calibrated mechanistic-empirical performance model that uses a well-known S-N function. However, the strength reference is established from beam laboratory modulus of rupture (MR) data. A graphic of the methodology is shown in Figure 2. The loaded slab is subjected to both traffic and environmental loads. The

traffic loads are established through user defined hourly truck traffic distributions and axle load spectra. The truck traffic is divided into ten classes, each of which is distributed according to the hourly distribution factor. The axle loads pertaining to each of the trucks is then established via an axle load spectrum (distribution of axle loads for a given axle type) and positioned a certain distance away from the slab edge depending on the user defined wander distribution. The environmental loads are manifested through an equivalent linear temperature gradient acting along the depth of the PCC cross-section. The gradient is derived from a combination of predicted temperatures at various locations throughout the cross-section and from moisture related warping. The design module also accounts for changes in PCC elastic modulus and foundation support. The modulus of subgrade reaction, k , will fluctuate depending on the monthly sub-surface conditions.

Damage is then accounted for by inserting the stresses into the field calibrated S-N performance model and is defined as the inverse of the number of cycles to failure, $1/N_f$ (Miner, 1945). Equation (3) shows the damage algorithm that is used to predict fatigue cracking in concrete pavements. The indices $i-m$, represent each of the possible loading combinations (i = age, j = month, k = axle type, l = load level, m = equivalent temperature gradient, n = traffic path) that the pavement can be subjected to; where σ is the applied stress.

$$D = \sum_{i,j,k,l,m,n} \frac{1}{N_f} = \sum_{i,j,k,l,m,n} \frac{1}{c_1 \left(\frac{MR}{\sigma} \right)^{c_2} + 0.4371} \quad (3)$$

One of the advantages of using this type of damage model is its computational efficiency, since it can rapidly account for, process and convert millions of load repetitions to damage. This allows for multiple designs to be considered within minutes. Some of the disadvantages however, are that it is insufficient in determining the in-situ state of damage because no information is given on the state of the material itself (no information on the stress-strain behavior and the reduction of the Elastic Modulus). In addition, it cannot account for size effect, load history effect, and variable amplitude loading without using some empirical calibration factors. Thus, there is a need for the development of a concrete fatigue model that can account for all three of the aforementioned effects and at the same time, be able to maintain a comparable level of computational efficiency to the S-N approach.

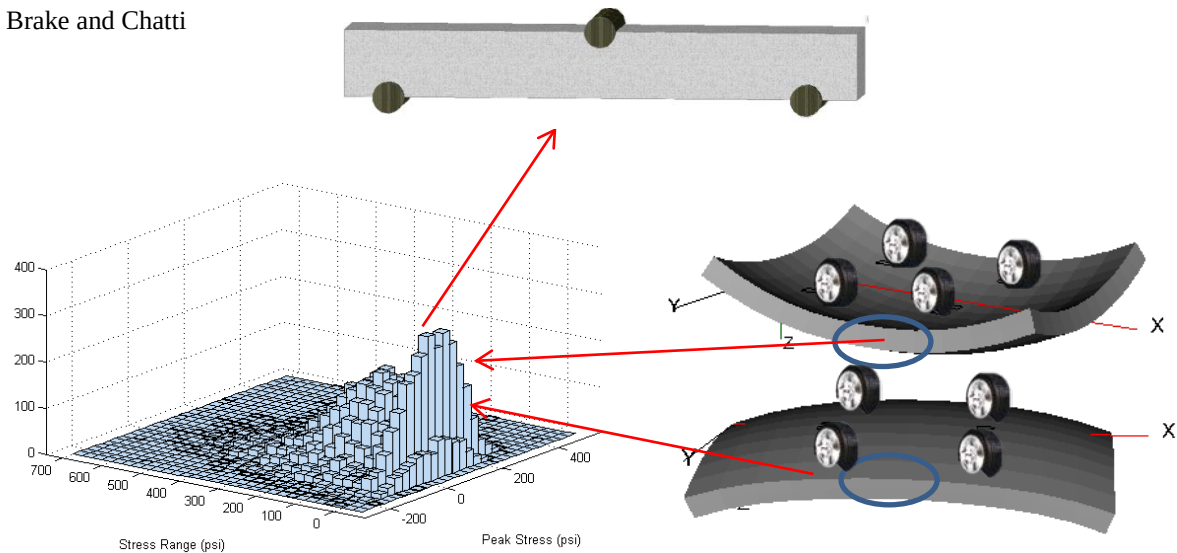


Figure 2: Design strategy for concrete pavement transverse cracking

Variable Amplitude and Load History Effects

In metallic structures, variable amplitude and load history effects have been investigated thoroughly over the last few decades. There is a collection of works that focus on investigating the variable amplitude (or load interaction) and load history effect. The purpose of these studies was to address the inadequacies of a LDR to predict fatigue damage under varying loading conditions. Sufficient experimental evidence suggests that fatigue life predictions using a LDR can both over-predict and under-predict damage depending on the sequence of loading (Fatemi and Yang, 1998). In a low to high (L-H) loading sequence, the LDR's will typically over-predict damage and in a high to low loading sequence (H-L), the LDR's will typically under-predict damage.

Several researchers proposed modifications to the original LDR approach. The double linear damage rule (DLDR) was introduced by Manson (1966) and it separated the S-N curve into two regions to represent the differences in damage accumulation rates within the crack initiation and crack propagation stages. This method was then refined to include a damage curve function (Manson and Halford, 1981) that allowed damage to be expressed as a non-linear power function dependent on load-level. Oh (1991) developed a non-linear damage rule for concrete that expresses the damage (D) as a load-level dependent cubic polynomial, and was able to predict the LDR error for both L-H and H-L loading sequences. More recently, a one dimensional continuum damage mechanics model was used to predict the load sequence effect (Lemaitre, 1992; Xiao, et. al., 1998). These models associate damage directly to elastic modulus degradation and are founded on thermodynamic principles in which the damage induced strain energy release rate, $\dot{Y}$, is the driving force for damage.

Under random amplitude loading, frequency-based techniques are often used to count the level of load repetitions. The rainflow counting method is one example where the random load time history is transformed into an equivalent constant amplitude load histogram allowing classical LDR's to be applicable (Anthes, 1997). A more recent technique uses power spectral density (PSD) functions to transform random stress time histories to the frequency domain with the purpose of correlating Gaussian or non-Gaussian distribution parameters to damage using LDR's (Benasciutti, 2005).

Fracture Mechanics based Fatigue Model

Fracture mechanics based models are also used extensively to predict fatigue fracture. In general, the models can be classified into two types: (1) linear elastic fracture mechanics (LEFM) models, in which stress intensities are related to crack propagation rates; and (2) cohesive zone (or delamination) models that incorporate a constitutive stress softening function that is related to the crack opening displacement (Roe and Siegmund, 2003). For metals, there are a plethora of LEFM fatigue fracture models that attempt to describe the crack propagation rate as a function of mode I stress intensity (K_I), stress range ($\Delta\sigma$), and fatigue threshold (K_{th}). The most well-known equation is the Paris Law which simply relates crack propagation rate to stress intensity (Paris, 1963), as shown in equation (4).

$$\frac{da}{dN} = C(\Delta K_I)^n \quad (4)$$

In concrete, the number of overall fatigue fracture models is limited, especially models that can account for variable amplitude loads. Slowik et. al. (1996) developed a concrete fatigue fracture model similar to the Forman equation in metals (Beden et. al., 2009) that accounts for the effect of stress intensity range (ΔK_I), peak stress intensity, load history, and overloads (F) as shown in equation (5). The model works well in predicting concrete fatigue cracking under variable amplitude in the acceleration cracking stage; however, no information on the crack propagation rates is given for the deceleration stage.

$$\frac{da}{dN} = C \frac{K_I^n (\Delta K_I)^p}{(K_{IC} - K_{I_{max}})^q} + F(\Delta\sigma, a) \quad (5)$$

Subramaniam et. al (2000) showed that fatigue fracture in concrete exhibits two distinct stages: a deceleration stage and an acceleration stage. The authors asserted that the crack resistance curve (R-curve) is directly responsible for the differences in behavior between the two stages. Brake and Chatti (2012a) derived an expression for a fatigue resistance curve (K_{th}) using load and fatigue crack rate data. The study also showed that by inserting K_{th} into the Paris Law, both stages can be predicted continuously under constant amplitude loading.

This research was then extended to random amplitude loading using a similar concept; however, in this work, the Slowik model was used as the governing fatigue equation (Brake and Chatti, 2012b). This in effect replaced the original K_I term with $K_{I_{eff}}$ in equation (5), which is defined as the difference between K_I and K_{th} . The stress range was accounted for the R-ratio which is defined as the ratio between the minimum and maximum applied load over a given cycle. The term Δa was defined as the crack extension; $K_{th,ult}$ was defined as the ultimate fatigue crack resistance and Δa_{ult} was defined as the ultimate crack extension. In total, there were 4 fatigue and 3 crack resistance parameters governing the fatigue process. Equation (6) shows the modified Slowik Model and equation (7) shows the function describing fatigue crack resistance curve.

$$\frac{da}{dN} = C \frac{(K_I - K_{th})^n (1-R)^p}{(K_{IC} - K_{I_{max}})^q} \quad (6)$$

$$K_{th} = \begin{cases} K_{th,ult} \left[1 - \left(1 - \frac{\Delta a}{\Delta a_{ult}} \right)^m \right], & \Delta a \leq \Delta a_{ult} \\ K_{th,ult}, & \Delta a > \Delta a_{ult} \end{cases} \quad (7)$$

A series of eight uniformly and normally distributed random amplitude loading fatigue tests were conducted on three point bending, single edge notch specimens. Two different sizes were tested. The large specimens had a depth and width of 100 mm and a span of 400 mm. The small specimens had a depth and width of 50 mm and a span of 200 mm. Each specimen had a notch to width ratio of 0.15 and the crack lengths were measured via the Jenq-Shah compliance technique (1985). Both the Paris and K_{th} parameters were obtained via a two-tiered non-linear regression routine in MATLAB. The first tier was used to determine the Paris parameters and the second to determine the K_{th} parameters. The Paris parameters were obtained using cumulative crack extension data in lieu of crack rate data, da/dN . This was necessary because in each sub interval i , the R and stress ratios are not constant. Equation (8) describes the cumulative crack extension, Δa_i , at each 0.25 mm crack growth subinterval; where Δa_f is the crack extension at fatigue failure and j is the total number of cycles in each 0.25 mm subinterval.

$$\Delta a_i = \sum_{i'=1}^i \sum_{j=1}^{j(i')} C \frac{K_{Ieff}^{n(1-R)^p}}{(K_{IC} - K_{Imax})^q} \bigg|_{i=\frac{\Delta a_f}{0.25}}^{i=1} \quad (8)$$

The average coefficients in equation (8) were: $\log C = -7.05$; $n = 5.14$; $p = 6.25$ and $q = -0.22$. These parameters were not found to be size dependent or geometry dependent. Figure 3 shows the experimental results and fit from two different size specimens.

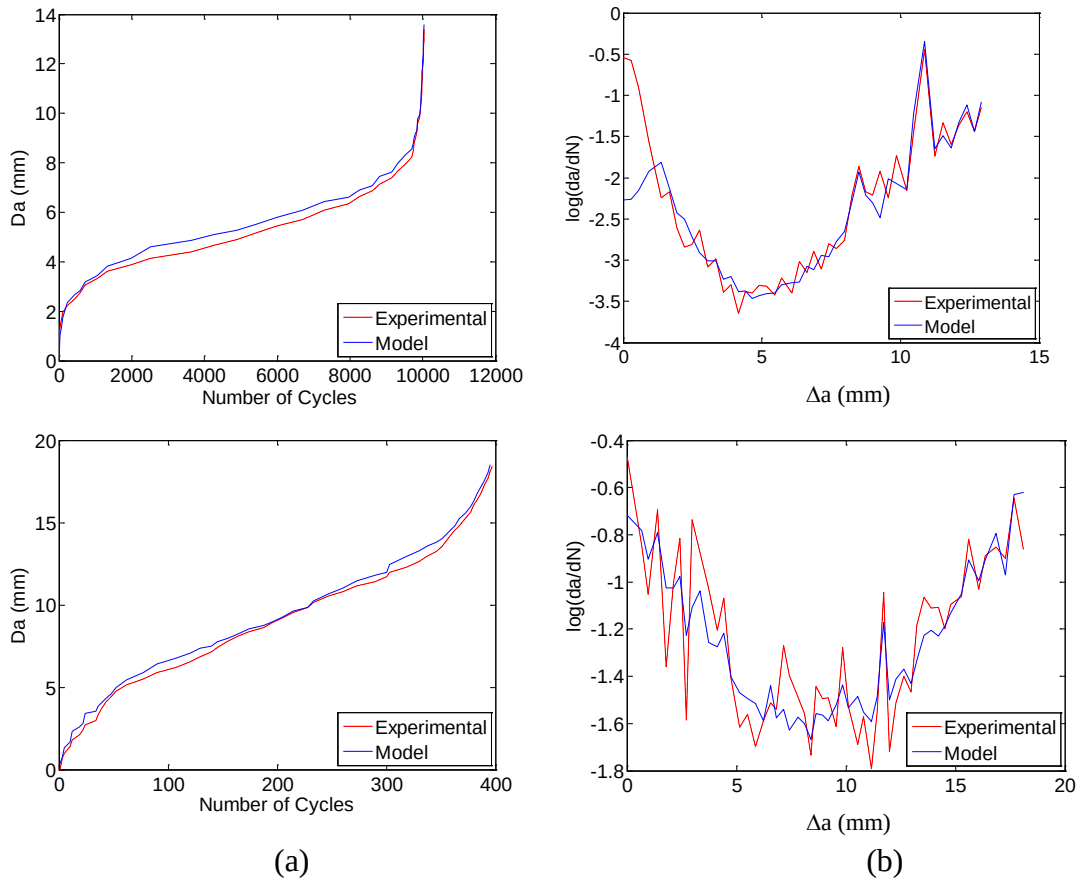


Figure 3: (a) Crack extension and number of cycles,
(b) Crack rate and crack extension

Research Significance

The objective of this paper is to compare the fatigue prediction between a LDR and a fracture mechanics model under a randomly distributed mid-slab edge pavement stress history. Studies have shown that LDR's under-predict damage under H-L loading sequences, but this effect may be lessened when the stress amplitudes have large R ratios. Therefore, it is important to know the dominant R ratio frequency and determine whether it counteracts the load history effect. It is also important to determine the prediction error generated by the LDR under a random stress distribution.

S-N Calibration

In this study, the fatigue fracture model was used to construct the S-N curve by simulating fatigue crack growth under various constant stress ratios. The R ratio was constant (0.1) for all stress ratios. The peak stress under monotonic loading was 5.23 MPa. The width and depth of the beam was 100 mm and the span was 400 mm. The initial notch was 6 mm. This notch length was chosen so that the nominal stress at failure (far field beam stress) would match the modulus of rupture found experimentally. Next, the simply supported fracture model was calibrated to match the S-N curve used in MEPDG to predict pavement slab fracture. This rationale was used because (1) in MEPDG the transverse cracking model also uses beam modulus of rupture as the strength reference, and (2) damage is limited to the edge of the pavement (meaning that only one dimensional crack propagation can exist, similar to a beam). The authors do acknowledge however, that the MEPDG S-N curve is a field calibrated model and would expect a handful of unknowns to influence both the crack propagation rate and direction.

The beam model was calibrated to explain the fracture phenomenon of a fully supported slab that is larger in cross-section (size effect), exhibits out of plane fracture (vertical and transverse direction), is subjected to frictional forces at the concrete/foundation interface, and exhibits mixed mode fracture. The calibration constants were strategically placed onto three terms: $\log C$, K_{th} , and n . The rationale being the following: $\log C$ should be calibrated to account for out of plane fracture, K_{th} should be calibrated to account for size effect (increasing R-curve resistance as a function of size), and n should also be calibrated to account for the effect of the foundation induced frictional resistance (should decrease because friction will reduce the crack propagation rate. Figure 4 shows the S-N curve before and after calibration. The calibration factors β_1 , β_2 , and β_3 were 0.713, 0.977, and 0.879, respectively. Equation (9) shows the calibrated relationship between N_f and stress intensities and was used to construct the calibrated S-N curve.

$$Nf = \int_{a_0}^a \left((10^{\beta_1 \log C}) \frac{(K_I - \beta_2 K_{th})^{\beta_3 n (1-R)^p}}{(K_{IC} - K_{I_{max}})^q} \right)^{-1} da, K_I < K_{IC} \quad (9)$$

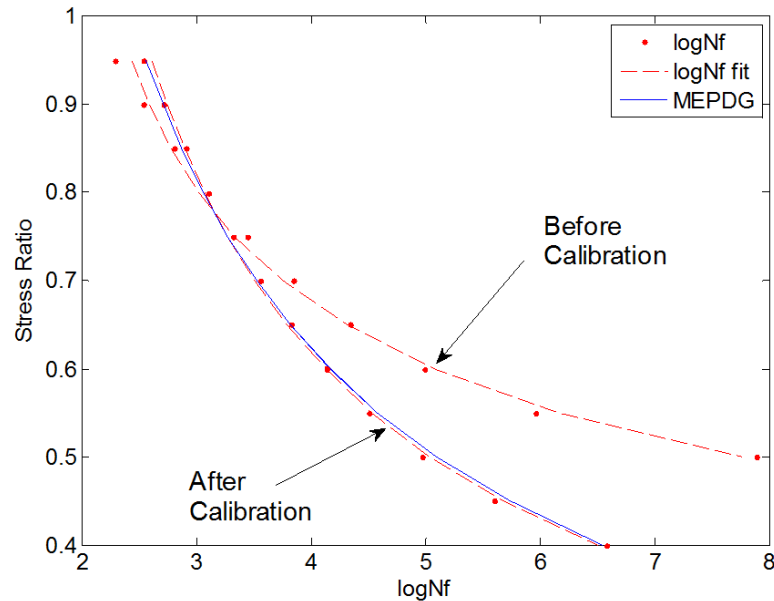


Figure 4: Calibration of S-N curve against MEPDG

MEPDG and Monte Carlo Simulation

Two simulations were conducted to assess the effect of load history. Two different climatic regions were chosen: Lansing, MI and Miami, FL. These regions were selected because of their annual mean temperature difference. The structural geometry and the traffic were identical for both simulations. The pavement cross-section consisted of a 200 mm PCC slab overlying a 150 mm A-1-A granular base and a A-2-7 subgrade. The length of the slab was 4.5 m. The 28 day modulus of rupture was 5.23 MPa. The 28 day elastic modulus was 29.6 GPa. For the purpose of analysis, the strength and elastic modulus were assumed to remain constant because no data for $K_{th,ult}$ and K_{IC} as a function of age was available to the authors. An annual average daily truck traffic (AADTT) of 185 with no growth was used to reduce simulation time. MEPDG default truck distributions and axle spectra's were used. The mean wander was 450 mm with a standard deviation of 250 mm. Medium axle spacing was used. Default axle spacing for tandem and tridem axles was also used. A zero built in temperature curl was used in the simulation to accelerate damage growth.

Monte Carlo Simulation

The randomized truck traffic simulation was created in MATLAB. The random environmental conditions were generated using output from the Enhanced Integrated Climatic Model (EICM) in MEPDG: cross-sectional pavement temperature, monthly modulus of subgrade reaction k , and the elastic modulus of the granular base E_{base} . The foundation properties varied monthly, and the truck traffic varied hourly. The linear temperature gradients were calculated as shown in Appendix QQ in the MEPDG Guide using the output cross-sectional temperatures from EICM. In addition, the non-linear temperature gradients were calculated based on the method proposed by Hiller and Roesler (2010) using the non-linear area (NOLA) concept. The temperature also varied hourly. The critical stresses were interpolated from a stress database (35,000+ runs) that was compiled with EVERFE (Davids et al., 1998). The equivalent layer concept was used to calculate the stresses, as shown in Appendix QQ. The adequacy of the interpolation scheme is shown in Figure 5. The interpolation

scheme output and EVERFE output are compared using 150 random runs using different pavement thickness, elastic modulus, traffic wander, and load for a tandem axle. Reasonable results were obtained.

Truck Traffic Randomization

Every hour, the total truck volume was established by the AADTT and the hourly truck traffic distribution. The hourly volume was multiplied by the truck class distribution and rounded. The result yielded a finite number of 'possible' truck repetitions for a given hour and truck class. A finite truck population was then created corresponding the number of 'possible' truck repetitions for each class, e.g., if three class 9 trucks and two class 13 trucks were 'possible', the truck population vector would be [9 9 9 13 13]. Using the *randperm* function in MATLAB, the vector can be re-arranged randomly to look like [13 9 9 13 9].

The axle population corresponding to each of the truck classes was created in a similar fashion. Each axle load spectra was multiplied by 1000 and rounded to yield a finite number of possible axle repetitions corresponding to each truck class and axle load. The population was then indexed according to the axle load. Thus, for a given random truck, if three 178 kN and two 169 kN single axle loads were 'possible', the load vector would look like [178 178 178 169 169] and after randomization, the re-arranged vector would look like [169 178 178 169 178]. Next, the total number of 'possible' axle load repetitions per truck was determined from the default probability matrix given in MEPDG. The population was arbitrarily chosen to be large (1000). The population was then conditioned by requiring the population average to equal the value shown in the MEPD probability matrix, that when averaged, would yield the number shown in the probability matrix. For example, if a population of five was chosen and the class 4 number of single axles per truck was 1.6, the population vector would look like [1 1 2 2 2] and after randomization would look like [2 1 2 2 1] (The average of the vector is 1.6).

In summary, trucks were selected at random and the number of corresponding 'possible' single, tandem, and tridem, was established and used to select the number of random load repetitions. The axle load was then placed at a random distance away from edge. The distance was dependent on the normal wander distribution. The stresses at the mid-slab edge were then calculated for every random axle load repetition. Note that the stresses were calculated under a hourly linear and non-linear temperature gradient, and monthly foundation stiffness.

Intra-axle (total peak and valley stresses within an axle group, including thermal stresses), and inter-axle contributions were also accounted for so that the contribution of peak stress and R ratio could be assessed. An intra-axle stress cycle for a tandem axle would look like the following: initial thermal stress $\rightarrow$ thermal stress + axle load induced stress (first axle at mid-slab, second axle placed away at a distance equal to the default axle spacing) $\rightarrow$ thermal stress + axle load induced stress (first and second axle placed equidistant from mid-slab), $\rightarrow$ thermal stress + axle load induced stress (second axle over mid-slab) $\rightarrow$ initial thermal stress. Note that the stress repetitions were only counted when there was an axle load present, meaning that isolated thermal cycles were not accounted for. The stresses were then counted using the following procedure: The stress at the i_{th} axle load repetition, σ_i , was counted as 'peak stress' when the condition $\sigma_{i-1} < \sigma_i > \sigma_{i+1}$ was identified. Similarly, a 'valley stress' was counted when the condition $\sigma_{i-1} > \sigma_i < \sigma_{i+1}$ was identified. Figure 6a shows a two dimensional histogram of the peak stresses and stress ranges from the Monte Carlo simulation for the Miami, FL simulation. The results show that both

peak stresses and stress range follow a normal distribution. Figure 6b and 6c shows the one dimensional histogram for the stress ratio and the R ratio, respectively.

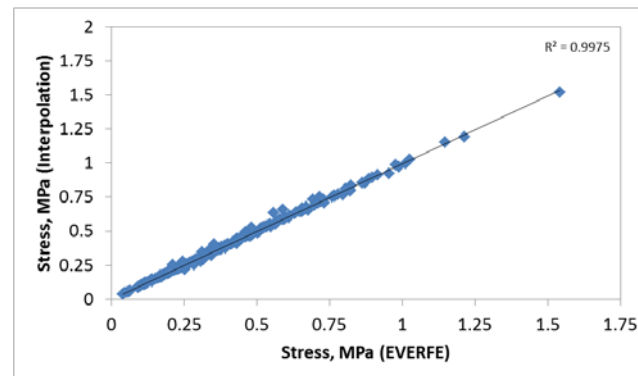


Figure 5: Comparison of bottom-up tensile stresses from EVERFE and Interpolation scheme

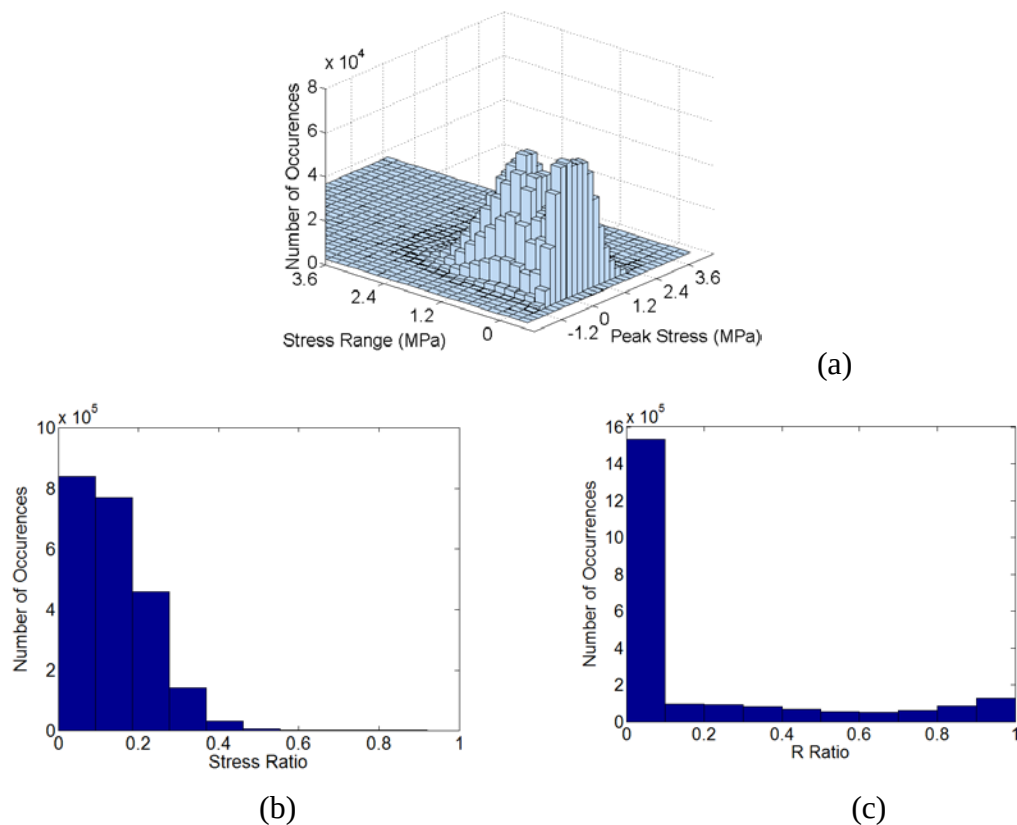


Figure 6: (a) Bi-variate histogram of peak stress and stress range, (b) histogram of stress ratios, and (c) histogram of R – ratios

Figure 6a and 6b show that the dominant R and stress ratio frequency falls between 0 and 0.1 and the dominant stress ratio is also between 0 and 0.1. Note that the R ratios that were negative were changed to zero because (2) was not calibrated to account for stress reversals, which explains why there is a dominant occurrence of zero.

Damage Prediction

The stresses and R ratios calculated in the previous section were used to predict fatigue damage with the fracture mechanics model (equation 6) and the calibrated S-N curve. The Lansing simulation produced more damage because there was a higher stress ratio frequency between 0.7 and 0.9 (1,948 for Lansing compared to 334 for Miami), meaning the pavement should fail faster. Figure 7a shows the comparison between the k histograms for Miami and Lansing. The left tail of the Lansing histogram has smaller k values compared to Miami. This means that during the spring period, the Lansing foundations are significantly more flexible which leads to higher pavement edge stresses. Moreover, the positive mean equivalent thermal gradient is higher for in Lansing which will produce higher bottom up stresses.

Figure 8 shows the damage accumulation from MEPDG and from the Monte Carlo simulation. There seems to be fairly good agreement between LDR predictions in the Monte Carlo and the MEPDG simulations (Lansing: at 29 years, $D_{\text{MEPDG}} = 0.78$ and $D_{\text{Monte Carlo}} = 0.73$, and Miami: at 40 years, $D_{\text{MEPDG}} = 0.61$ and $D_{\text{Monte Carlo}} = 0.62$). Note that the reason why damage is less than one in both cases is because the simulations were stopped when $K_I = K_{IC}$ in the fracture fatigue equation.

Damage was assessed in fracture model by the crack extension, Δa . These values were normalized by the crack extension at failure (Lansing, $\Delta a_f = 19.2$ mm and Miami, $\Delta a_f = 24.1$ mm) so the damage could be compared to the LDR. As shown in Figures 8a and 8c, the fatigue fracture model yields greater damage than its counterpart linear model. The LDR's seem to under-predict damage by approximately 30%, $(1 - D_{\text{MEPDG}})$, on average. This is happening because the linear model does not predict damage well after a significant overload ($SR \sim 0.7-0.9$). This is consistent with the findings of Oh (1991) and the Lemaitre (1992). In addition, it seems that the R ratio does not counteract the load history effect under these specific random stress histories.

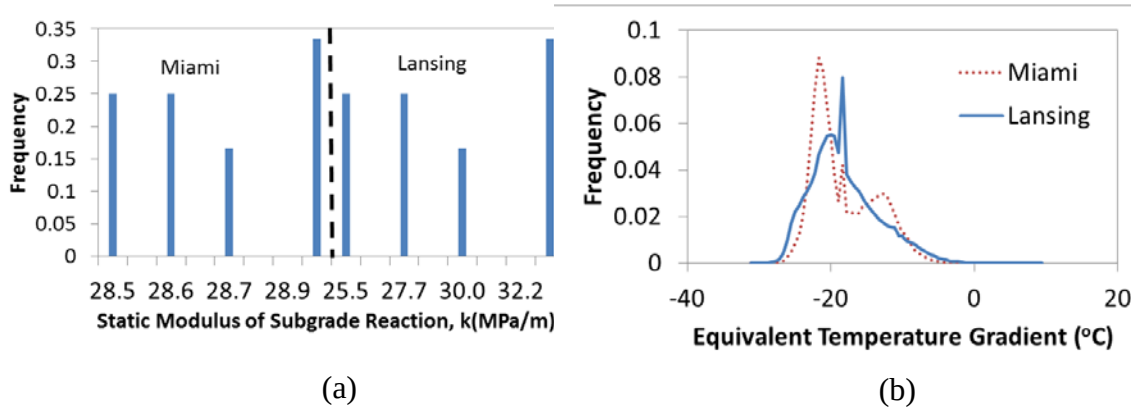


Figure 7: (a) Histogram of modulus of subgrade reaction, k for Miami and Lansing, (b) Histogram of Equivalent Linear Temperature Gradients for Miami and Lansing Simulations

Preliminary Calibration of Percent Slabs Cracked

One way to account for the under-prediction of damage when using the linear damage equation is to re-calibrate the % slabs crack algorithm, shown in equation (10), by inserting a calibration factor, C_1 . The variable FD represents the fatigue damage,

which is defined from 0 to 1. The MEPDG defines $FD = 1$ when the % slabs cracked is equal to 50%.

$$\% Crack = \frac{1}{1+(C_1 FD)^{1.68}} \quad (10)$$

In MEPDG, C_1 is equal to one. For the % slabs cracked to be 50% when FD from the non-linear fatigue model is 1, C_1 must be equal to 1.25 for the Lansing case and 1.6 for the Miami simulation. Note that, for different types of loading history, the discrepancy between non-linear damage and linear damage changes and C_1 will change accordingly. Therefore, a sensitivity analysis would need to be conducted to explain the effect of the fatigue parameters n , p , q , $\log C$, and stress distribution on the prediction error between non-linear and linear damage. In addition, the effect of size and boundary conditions should also be introduced within the fatigue cracking equation.

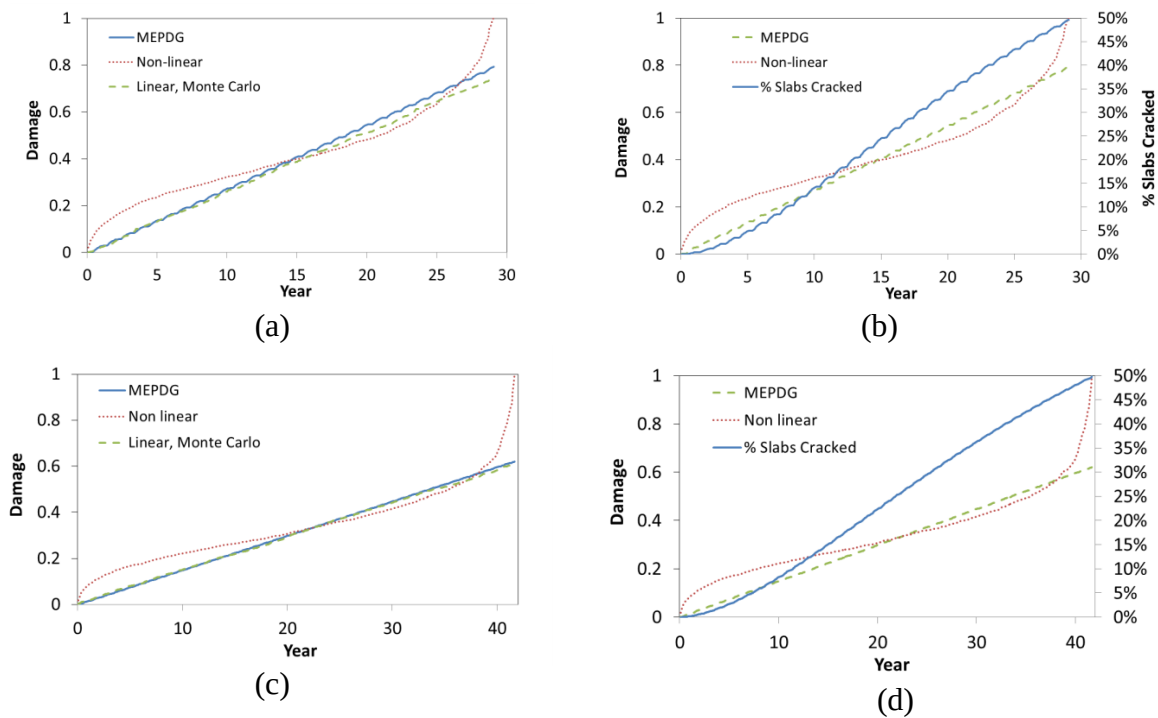


Figure 8: (a) Lansing: linear and non-linear damage, (b) Lansing: % slabs cracked, linear and non-linear damage, (c) Miami: linear and non-linear damage, and (d) Miami: % slabs cracked, linear and non-linear damage

Conclusion

A Monte Carlo truck traffic simulation was conducted for two identical pavement cross-sections in two different climatic regions, Lansing, MI and Miami, FL. Fatigue damage was assessed using a non-linear fracture mechanics fatigue model that accounts for the following effects: peak stress, stress range, and load history. The model parameters were determined by a two tiered calibration procedure based on eight laboratory specimens subjected to both uniform and normal distribution random loading sequences. The fatigue fracture model was used to construct an S-N curve and was calibrated to match the MEPDG S-N curve. Three parameters ($\log C$, K_{th} , and n) were strategically chosen to account for out of plane fracture, size effect, and frictional effects. A Monte Carlo simulation was then conducted to assess the significance of R ratio and load history.

The damage predicted in the Monte Carlo simulation using the calibrated S-N curve compared well with the bottom up damage prediction in MEPDG. However, the results show that damage will accumulate more rapidly in the non-linear fatigue fracture model. On average, the LDR seems to under-predict damage by approximately 30%, although the error seems to vary depending on climatic region. The results also show that the dominant R ratio frequency is between 0 and 0.1 and does not counteract the load history effect.

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