

# CONSIDERING HOT-MIX-ASPHALT FATIGUE ENDURANCE LIMIT IN FULL-DEPTH MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

## **AUTHORS:**

Marshall R. Thompson, Corresponding Author  
Professor Emeritus – Civil Engineering  
University of Illinois @ U-C  
205 North Mathews  
Urbana, IL 61801-2350  
e-mail: [mrthomps@uiuc.edu](mailto:mrthomps@uiuc.edu)

Samuel H. Carpenter  
Professor of Civil & Environmental Engineering  
University of Illinois @ U-C  
205 North Mathews  
Urbana, IL 61801-2350  
e-mail: [scarpent@uiuc.edu](mailto:scarpent@uiuc.edu)

## **ABSTRACT**

Recent studies (including comprehensive lab testing efforts) have confirmed the existence of a “Fatigue Endurance Limit” (FEL) for HMA. The FEL concept (there is a repeated HMA flexural strain level below which HMA damage is not cumulative) is the basis of “PERPETUAL PAVEMENT” design. The FEL principle is not considered in the NCHRP 1-37A Mechanistic Empirical Pavement Design Guide.

At the 2004 ISAP Conference on Design and Construction of Long Lasting Asphalt Pavements (Auburn, AL), the authors proposed M-E design principles (based on the ILLI-PAVE structural model) for “Long Lasting HMA Pavements.” The proposed critical condition is the HMA flexural strain for the “hottest month” of the year. If the critical HMA flexural strain is < the FEL, the pavement is considered to be PERPETUAL.

For pavements that do not meet the PERPETUAL criterion, there are times during the year (lower HMA temperatures than for the hottest month) when the HMA flexural strains are also less than the FEL. Previous studies by Carpenter suggest that strain repetitions greater than the FEL can be sustained and the HMA will retain FEL behavior. It is not clear how the FEL concept applies in these circumstances.

Limited studies by Thompson for the IL DOT suggest that for pavements that do not meet the PERPETUAL criterion, consideration of the FEL has a limited impact on the predicted HMA fatigue life. The selection of the HMA fatigue algorithm (for considering HMA strains > FEL) appeared to be a more important factor.

In this paper, M-E HMA fatigue concepts are reviewed and summarized and various approaches for considering the HMA FEL in practical M-E HMA FULL-DEPTH thickness design are considered. Procedures are recommended.

**KEY WORDS:** Fatigue Endurance Limit, Perpetual Pavement, Mechanistic-Empirical Pavement Design, HMA Thickness Design

## INTRODUCTION

With the advent of Superpave Hot Mix Asphalt (HMA) mixture design and PG graded asphalt, it is typically assumed that for thicker HMA pavements (i.e. FULL-DEPTH HMA) material selection and mixture design techniques are sufficient for accommodating HMA rutting and HMA thermal cracking concerns. For FULL-DEPTH HMA pavements, subgrade stress/strain levels are generally within acceptable limits and subgrade permanent deformations are not significant. Thus, HMA thickness requirements for high-volume highway traffic are controlled by HMA fatigue cracking considerations.

The HMA FEL (Fatigue Endurance Limit) is the repeated HMA flexural strain level below which HMA damage is not cumulative. Thus, an HMA layer experiencing strain levels less than the FEL should not fail due in fatigue. The term “PERPETUAL PAVEMENT” (PP) was introduced by Huddleston, et al in an Asphalt Pavement Alliance publication (1) to designate such pavements. Terms such as “long-life,” “long-lasting,” and “extended life” have also been utilized to describe PP.

There are two conditions when the FEL concept is applicable.

- Condition 1 - The HMA pavement meets the PP criterion (HMA flexural strain  $\leq$  the FEL) for the “hottest month” of the year.
- Condition 2 - The HMA pavement does not meet the PP criterion for the “hottest month,” but there are months (at some HMA temperatures LOWER than for the hottest month) when the HMA flexural strains are  $<$  the FEL.

Both conditions are considered in this paper.

The initial portion of this paper is devoted to presenting a summary of current HMA fatigue and FEL technology. The following parts present a general framework for considering the HMA FEL and demonstrate how the FEL impacts HMA FULL-DEPTH HMA thickness design in routine/practical Mechanistic-Empirical (M-E) flexible pavement design.

## CONVENTIONAL FATIGUE

The inclusion of fatigue in the structural thickness design of HMA pavements has traditionally involved an expression relating the tensile strain and the number of loads to a predetermined failure level, commonly the 50 percent reduction in initial modulus. This has been expressed as:

$$N_f = K(1/\epsilon)^n \quad (1)$$

Where:

$N_f$  is the number of load cycles to failure,  
 $\epsilon$  is the tensile strain at the outer fiber of the HMA,  
K and n are regression constants from the lab testing.

Because of the phenomenological nature of this relationship, some have proposed applying adjustments to this relation to obtain a “better fit” with observed behavior differences. The most notable adjustment addition being the addition of a modulus term

$$N_f = K(1/\epsilon)^n [(1/E^*)^b] \quad (2)$$

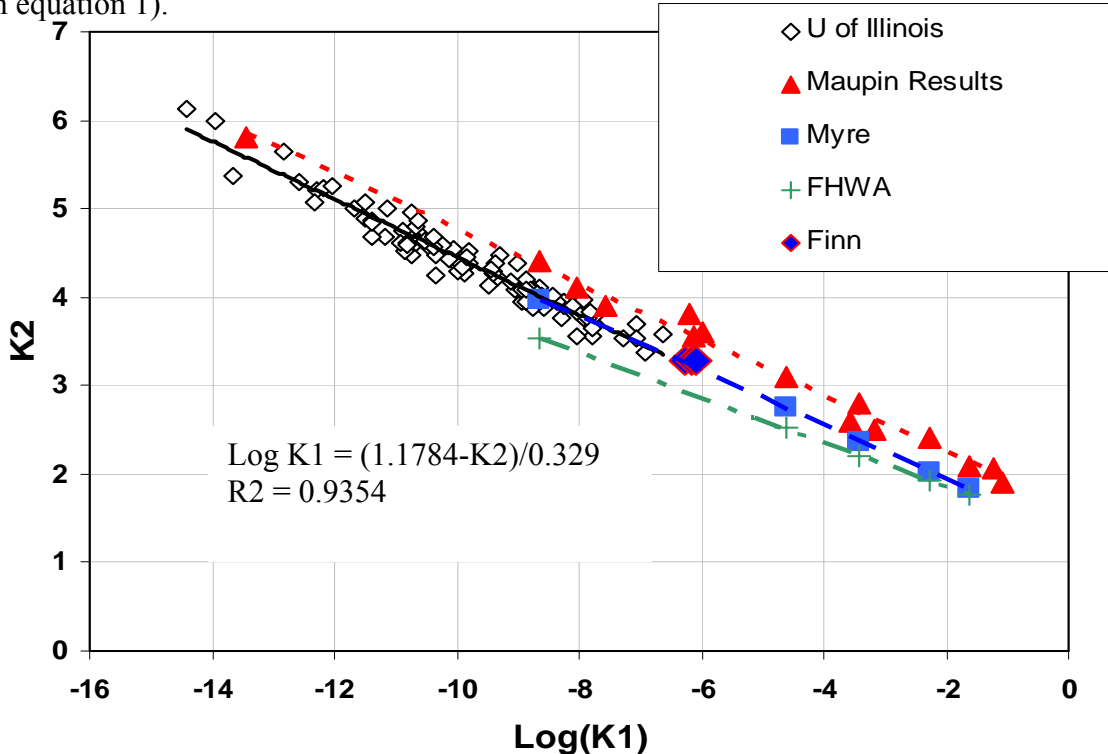
Where:

$E^*$  is the dynamic modulus of the HMA, and  
 $K$ ,  $n$ , and  $b$  are the regression constants from the lab testing.

Various researchers have conducted lab tests and correlated these with observed field performance to generate the regression coefficients. These regression coefficients have in turn been regressed against material properties to generate equations with a relation to mixture properties. The most recent being the work performed to develop a nationally calibrated fatigue model for the NCHRP 1-37A Mechanistic-Empirical Pavement Design Guide (MEPDG).

The fundamental nature of the strain relationship has been shown to account for some 90+ percent of the variability in the data. The addition of the modulus term accounts for the remainder, and for the inadequacies of a non-fundamental relationship. The main failing of current models is in the assumption of a constant  $n$  value for all different HMA materials, with mixture variables being included primarily in the  $K$  term. This form ignores the preponderance of test data which has shown that  $K$  and  $n$  are integrally related in a consistent manner.

Figure 1 illustrates the results of testing over 100 different mixtures at the University of Illinois for the Federal Aviation Administration (FAA) and the Illinois Department of Transportation (IDOT). Air voids varied from 4 to 7 percent. Nominal maximum aggregate size has varied from 9.5 mm to 37.5 mm. Asphalt binders varied from penetration graded to viscosity graded to performance (PG) graded, with both neat and polymer modified (SBS, SBR, EVA) materials being used. The equation shown is for the U of I IDOT mixes (note:  $K_1$  is  $K$  and  $K_2$  is  $n$  in equation 1).



**FIGURE 1 Relation between fatigue algorithm coefficients.**

Also included on this figure are results from Josten Myre of Norway who conducted fully supported beam testing, and results from FHWA (2). The consistent nature of all these different tests clearly establishes a consistent phenomenological relationship between  $K$  and  $n$  ( $K^2$  in the figure). A model that fixes  $n$  will be incorrect. Further, an agencies HMA mixtures will all be expected to follow this relationship. Thus, the  $K - n$  relationship will overshadow a forced constant  $n$  relationship to an extent that attempts at national calibration would not be expected to be indicative of the performance found in a particular state.

## **FATIGUE ENDURANCE LIMIT**

It has long been postulated by Monismith that there appeared to be a strain below which there is no fatigue damage to the HMA (3). This has been investigated by Carpenter (4) starting in 2000, and recently by NCAT. By conducting extremely lengthy tests at low strain levels curves shown in Figure 2 are consistently developed. Of the 24 mixtures tested, none deviate from this trend. There is a definite point at which each mixture's traditional strain- $N_f$  curve deviates from the typical straight line relationship and assumes a relatively flat slope. This flat slope indicates that lower strains produce extraordinarily long fatigue lives, often referred to as "infinite."

Of all the mixtures tested, none required strains below 70 micro-strain to exhibit this plateau value for extended fatigue performance. Depending on the binder, this extended plateau is reached at significantly different strain values. The term chosen for the break point where the traditional fatigue curve begins to flatten out is termed the Fatigue Endurance Limit (FEL in micro-strain units). Strains below the FEL will begin to show extraordinarily long fatigue lives as compared to those that would be predicted by the traditional model. As shown in Figure 2, there are a range of FEL values for the mixtures tested. The difficulty in differentiating the mixture variables and their impact on the FEL derives from the use of the phenomenological relationship for strain and loads to failure. Because this relationship is not fundamental it cannot adequately describe mixture performance under varying inputs.

The dissipated energy approach of Carpenter and Shen (5) clearly provides a unique relationship between damage and loads to failure. The data for all mixtures tested is shown in Figure 3. The plateau value (PV) is the percent of dissipated energy in a load cycle that is causing actual damage relative to the following load cycle. The details for developing this value can be found elsewhere (4, 5). The significant point of this relationship is that there is one unique damage level that relates to fatigue life. Further, there is one unique value where the behavior of the HMA changes from traditional accumulation of damage to one where damage does not accumulate. This point is the FEL, and is a constant PV for all mixtures tested, and is denoted by the term  $PV_L$  on the figure.

The FEL represents the balance point between damage and healing in the HMA. For strain levels above the FEL, the damage done is considerably greater than the healing potential for the HMA (5). When strains are below the FEL, the damage is equal to or less than the healing potential, and the damage is small enough that it is potentially completely repaired during the load cycle which is a haversine at 10 Hz in the flexural fatigue test.

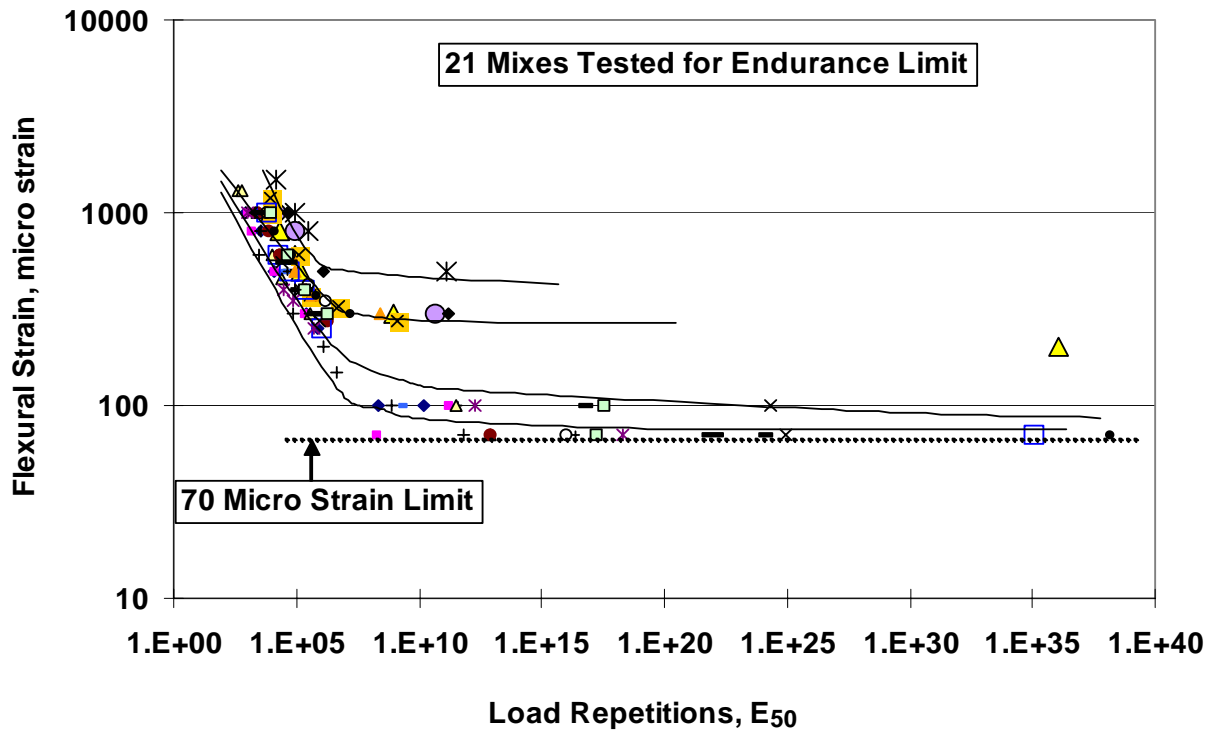


FIGURE 2 Strain – load relationship illustrating the Fatigue Endurance Limit.

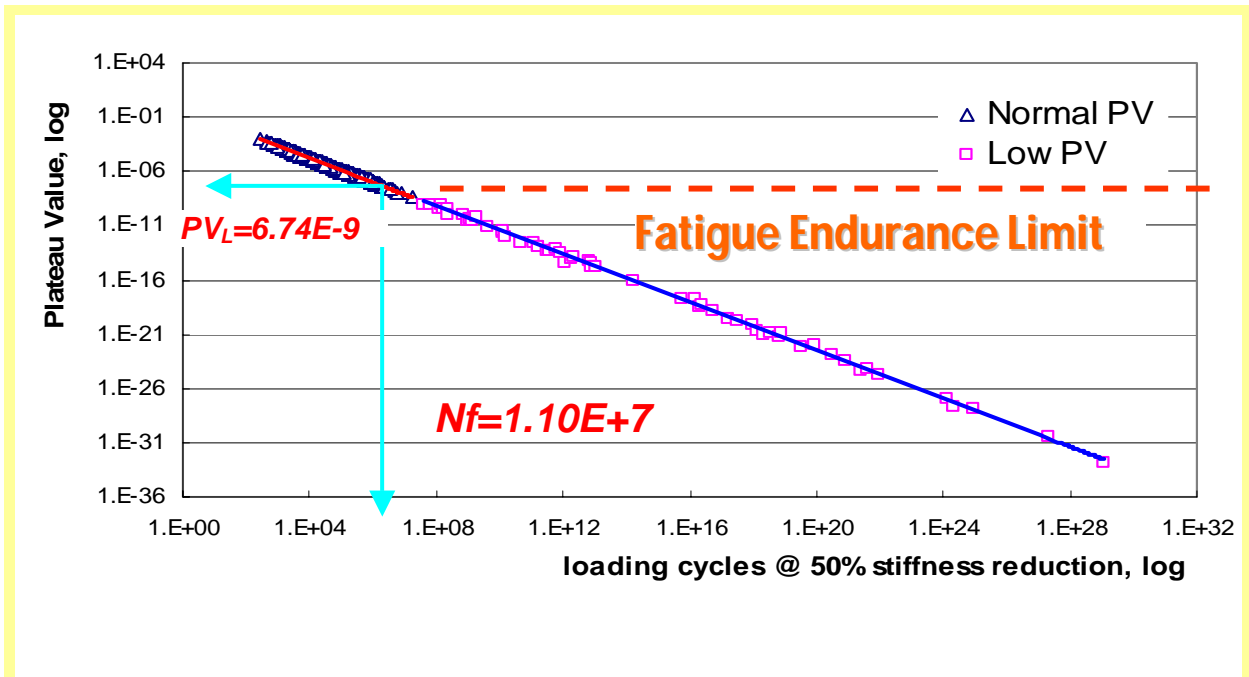


FIGURE 3 PV damage relation with loads to failure in fatigue test.

Table 1 lists the FEL strain values for the mixtures tested at low strain levels. The measured value was obtained from the traditional fatigue curves as the knee in the curve between two straight lines, one representing normal strains and one the low strain tests. The predicted value is obtained from the strain vs. PV relationship presented by Shen and Carpenter (5) to calculate the strain that corresponds to the  $PV_L$  value. It must be noted that in no instance did the FEL value fall below 100 micro-strain. This clearly illustrates the range over which extended fatigue life can be obtained as being somewhere between 100 and 70 micro-strain.

**TABLE 1 FEL Strains and Mixture Information**

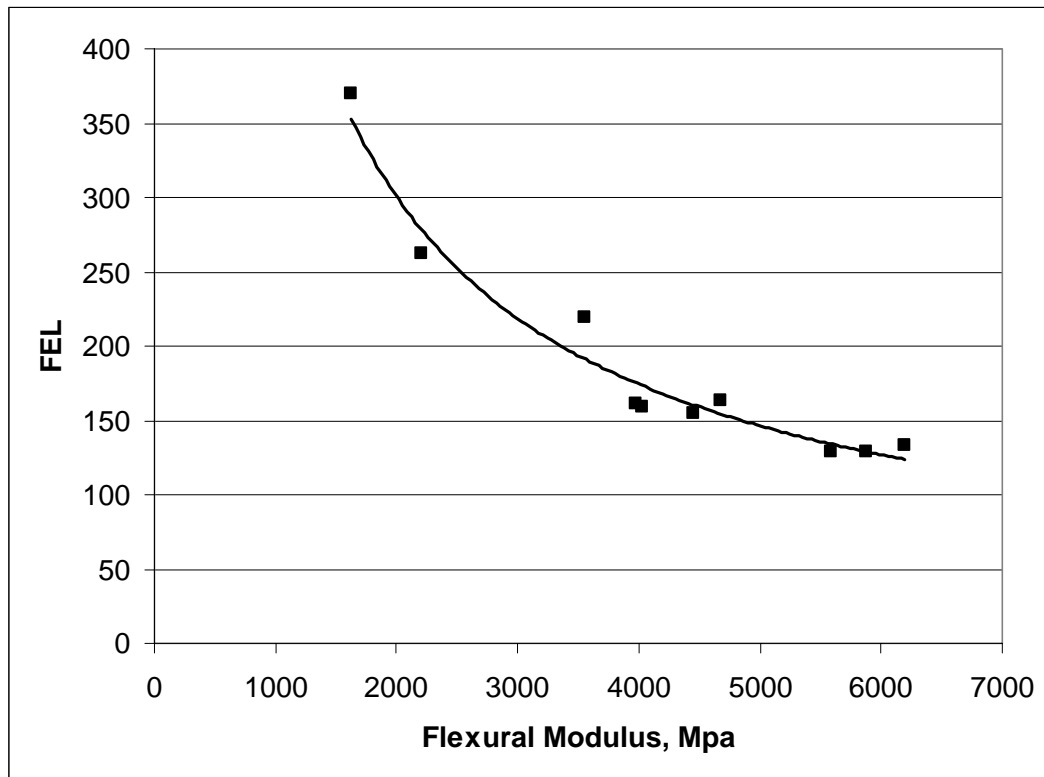
Mix ID	Tested strain Limit ( $\mu\epsilon$ )	Predicted Strain Limit ( $\mu\epsilon$ )	Material information
5N105	500	387	SBS PG76-28, 0.1% lime anti strip, limestone, 4.70%AC
3N90	300	229	SBS PG76-22, 0.5% anti strip, dolomite, 5.06%AC
1N80D	300	273	SMA, SBS PG76-28, 8.04%AC
5N90	100	162	PG64-22, 0.5% anti strip, dolomite, 4.90%AC
2N90	100	170	SBS PG70-22 Dolomite, 5.90%AC
1N105	100	139	SBS PG70-22 dolomite, 6.5%AC
3N70	100	134	PG64-22 limestone, 5.17%AC
6N50	100	168	PG64-22, 0.7% anti strip, limestone, 5.05%AC
3N90T	100	136	PG64-22 limestone, 4.06%AC
8N70	100	167	PG64-22 limestone, 4.93%AC

### VARIATION IN FEL

The FEL data generated thus far indicate that 70 micro-strain is a conservative value that guarantees a structural design will perform in the region of extended fatigue life, providing a “no damage” performance. A design incorporating this 70 micro-strain level under the most extreme conditions can be considered a PP. If the strain remains around 70 - 100 micro-strain during the entire year, there is no accumulation of HMA fatigue damage.

Different mixtures produce different FEL values. While this can be attributed to binder differences, the lack of binder data available to date only allows a comparison with modulus, which for a specific gradation will be controlled primarily by the binder. Figure 4 presents the FEL and the flexural stiffness obtained during the initial 50 load cycles of the fatigue test for 11 IDOT Superpave mixtures tested at low strain levels at the University of Illinois. Table 1 presents the mixture and binder information. These data clearly indicate that for these mixtures of similar Superpave design, alteration of the modulus, essentially through binder differences, produces a strong relationship between modulus and the FEL. What is important for these IDOT Superpave mixtures is that there is a strong indication that as modulus increases, the FEL decreases asymptotically and most likely reaches a limit around 85 micro strain, which would correspond to an extremely high dynamic modulus (near 12,000, ksi when converted) at 20 C,

with a 10 Hz haversine load. This modulus value is certainly not attainable in any normal mixture. This is extremely strong evidence that IDOT mixtures may have a conservative FEL of 70 micro-strain.



**FIGURE 4 Relationship between FEL and flexural modulus in fatigue test.**

The relationship between the FEL and the flexural modulus also clearly indicates that there is a lower limit to the FEL that appears to be well above the 70 micro strain level. Further, because healing potential increases as temperature increases, it can be expected that the FEL will change with temperature, which may be indirectly indicated by this modulus relationship.

It is recognized that some designers propose conducting a “traditional fatigue design” using the entire shape of the fatigue curve from normal to low strain. When the loading and/or environmental conditions produce a strain below the FEL range there is no damage. This is a legitimate design approach, but this is not a PP design, and is extremely more complicated than a traditional design approach if it is to be performed properly. Utilizing FEL concepts with a traditional fatigue curve is not consistent as one incorporates healing while the other ignores it even though it is present. Load damage levels change with environment, and even amount of traffic. The FEL changes with temperature and binder types. Healing rates will change with season. Unless these are accounted for, the desire to perform a traditional fatigue design is misplaced and will not provide a consistent relationship between load levels, damage, and load repetitions to failure.

The first element that must be considered is that the FEL likely changes with temperature. Because the FEL is tied closely to the healing potential of the binder, at higher temperatures healing occurs more rapidly and damage is recovered more quickly, and the strain level that can be tolerated with no damage accumulation is increased (6). To be correctly

included in a traditional fatigue design the FEL must vary with season, just as the modulus and resulting strain vary with season. If the variation in FEL is included, the impact of healing in the HMA between load pulses must be considered. Rest periods reduce the damage of any one load application, and are a major factor in the lab to field shift values of 40 to 400 that have been applied to make existing lab fatigue models applicable to field conditions.

The selection of the conservative 70 micro-strain during the hottest period of the year eliminates the need to consider the intricacies of fatigue damage accumulation and truly provides a PP. The axle load selected for the design should be representative of the maximum axle load in the traffic stream. The occurrence of extreme events will not invalidate the FEL behavior as was shown by Thompson and Carpenter (7). In this paper, it was shown that overloads in the range of 7 times the design axle load for approximately 15 to 20 percent of the time did not negate the “no damage” performance when strains returned to a value below the FEL. Healing studies underway at Illinois may provide insight into the ability of the HMA to completely heal intermittent overload damage during the period when loads are below the FEL.

### **PAVEMENT LOADING**

Current truck load limits are 20-kip single-axle-load (SAL) and 34-kip tandem-axle-load (TAL). The 20-kip SAL is more critical (higher HMA flexural strain). The NCHRP 1-37A report (8) indicates that 80-90% of the traffic loadings on the interstate system are generated by Class 9 vehicles (18-wheelers). The 1-37A default truck traffic table for Class 9 indicates that about 2.5% of the SALs are greater than 18 kips and about 1% are greater than 20 kips.

Representative ILDOT truck weigh station data show Class 9 vehicles constitute about 85% of truck traffic. Typically (for Vehicle Classes 4-9), less than 1% of the SALs exceed 18 kips and less than 1% of the TALs exceed 36-kips.

Even though SALs loads greater than the critical SAL and pavement temperatures in excess of the maximum MMPT may be experienced, their occurrences are very limited. If it is assumed that 1/12<sup>th</sup> of the “overloads” occur during the hottest month (the critical month for PP design), practically all of the truck SALs are 18 kips or less. Thus, HMA strain repetitions greater than the FEL will consume a practically insignificant amount of the HMA fatigue life.

Information previously presented (7) clearly indicates that periodic overloads will not damage the HMA to the extent that the FEL limit effect is negated. It is recommended that the 18-kip SAL be utilized as the PP design load.

### **STRUCTURAL ANALYSIS**

HMA strains can be estimated from various Elastic Layer Programs and stress dependent finite element programs such as ILLI-PAVE. In this paper, ILLI-PAVE is used as the structural model.

#### **ILLI-PAVE Program**

ILLI-PAVE is an iterative finite element flexible pavement analysis model. Asphalt concrete is modeled as a linear elastic material. Nonlinear, stress dependent resilient modulus material models and failure criteria for granular materials and fine-grained soils are incorporated into the model. Granular materials are considered stress hardening (modulus increases as stress increases) and fine-grained soils are “stress-softening” (modulus decreases as stress increases). The principal stresses in the granular material and fine-grained soil layers are modified at the end

of each iteration so that they do not exceed their shear strength as defined by the Mohr-Coulomb theory of failure.

HMA flexural strains at the bottom of the HMA layer are calculated from the following algorithm (developed at the University of Illinois and used in the Illinois DOT FULL-DEPTH HMA pavement design procedure):

$$\text{Log } \epsilon_{HMA} = 5.746 - 1.589 \text{ Log } T_{HMA} - 0.774 \text{ Log } E_{HMA} - 0.097 \text{ Log } E_{Ri} \quad (3)$$

Where:

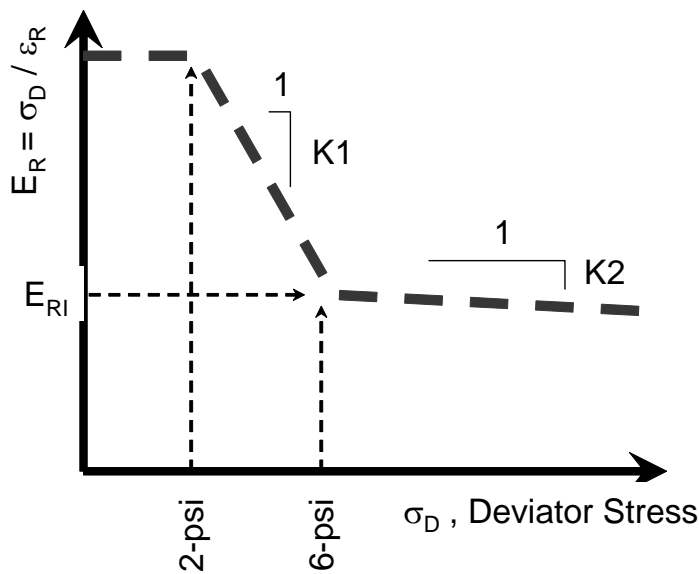
$\epsilon_{HMA}$  - HMA flexural strain (micro-strain) for a 9-kip wheel load

$T_{HMA}$  - HMA thickness (inches)

$E_{HMA}$  - HMA modulus (ksi)

$E_{Ri}$  - Subgrade modulus (ksi)

NOTE: Subgrade modulus ( $E_{Ri}$ ) is approximated as the resilient modulus at a repeated deviator stress of 6 psi. The arithmetic resilient modulus – deviator stress model is shown in Figure 5. As the subgrade deviator stress decreases, the resilient modulus increases significantly.



**FIGURE 5 Arithmetic Resilient Modulus – Repeated Deviator Stress Relation for Fine-grained Soils.**

The dominant factors affecting  $\epsilon_{HMA}$  are  $T_{HMA}$  and  $E_{HMA}$ . Table 2 presents HMA thicknesses required to achieve FELs of 70 and 85 micro-strain for a range of HMA moduli and subgrade  $E_{Ri}$ s.

### ILLI-PAVE Inputs

ILLI-PAVE inputs (for a given loading) are HMA modulus and subgrade modulus ( $E_{Ri}$ ). The HMA modulus (see Table 2) has the most impact and is influenced by many factors. Asphalt grade and content, temperature, void content, and rate of loading are the dominant factors.

“Dynamic” modulus  $|E^*|$  has been widely used to characterize such conditions. An early Asphalt Institute (9) equation for predicting  $|E^*|$  uses the following inputs: HMA temperature,

asphalt cement viscosity, % asphalt cement, % air voids, % passing #200 sieve, and loading frequency. All inputs, except for HMA temperature and loading frequency, are specific to the HMA mix design. A similar, but more comprehensive equation has been developed by Fonesca and Witzak (10). A further modified version was developed in the NCHRP 1-37A project (8). The most recent “Witzak Model” (to be included in the “final version” of the AASHTO MEPDG) is presented in a recent AAPT paper (11).

Christensen et al (12) have developed a procedure based on the Hirsch Model. The model is particularly appealing since the major inputs are the HMA mixture design properties of VMA and VFA and the “modulus” of the asphalt cement. They state:

“Although not as good as actual measurements, the accuracy of the model is probably suitable for many practical design and analysis applications. Estimated modulus values in fact are probably similar in reliability to measured values when only limited replicate tests can be performed, or when the laboratory personnel are not experienced in making modulus measurements on asphalt concrete specimens.”

**TABLE 2 HMA Modulus and Subgrade E<sub>Ri</sub> Effects on HMA Thickness for Achieving FEL**

HMA MODULUS (ksi)	E <sub>Ri</sub> (2 ksi)	E <sub>Ri</sub> (5 ksi)	E <sub>Ri</sub> (7.5 ksi)	E <sub>Ri</sub> (10 ksi)
300	17.1*/15.1**	16.1/14.3	15.7/13.9	15.5/13.7
400	14.8/13.1	14.0/12.4	13.7/12.1	13.4/11.9
500	13.3/11.8	12.6/11.1	12.3/10.9	12.0/10.7
600	12.2/10.8	11.5/10.2	11.2/9.9	11.1/9.8
700	11.3/10.0	10.7/9.5	10.4/9.2	10.2/9.1

\* First entry is HMA thickness (inches) to achieve 70 micro-strain.

\*\* Second entry is HMA thickness (inches) required to achieve 85 micro-strain.

HMA temperature depends on pavement location. Witzak (13) proposed the following algorithm to estimate Mean Monthly Pavement Temperature for design purposes:

$$\text{MMPT} = \text{MMAT} [1 + (1 / \{Z+4\})] - [34 / (Z + 4)] + 6 \quad (4)$$

where:

MMPT - Mean Monthly Pavement Temperature (°F)

Z - Depth from pavement surface (inches)

MMAT - Mean Monthly Air Temperature (°F)

Witzak (13) recommends the depth Z to be taken at the upper third point of the HMA layer. Hill and Thompson’s data (14) for FULL-DEPTH HMA pavements indicate that the HMA temperature at a depth from the surface of about 0.4 of the HMA thickness is appropriate for estimating an “effective modulus” for the HMA layer. The 0.4 value is used in this paper.

Thompson et al (15) found “excellent agreement” between the Climatic-Materials-Structural computer model (the predecessor to the Integrated Climatic Model) and the Asphalt Institute procedure. Recently, similar comparisons at the University of Illinois confirmed the

agreement between the Integrated Climatic Model (utilized in NCHRP 1-37A) and the Asphalt Institute procedure. The Asphalt Institute procedure is particularly appealing because of its simplicity and the ready availability of Mean Monthly Air Temperature data (weather.com website). The Asphalt Institute procedure is considered quite adequate for routine flexible pavement analysis and design activities.

## DESIGN OF LONG LASTING PAVEMENTS

In this section, a general design framework is presented. Design concepts are illustrated for typical conditions.

HMA Modulus – Temperature relations (as presented in Ref. 7 and utilized in this paper) for representative 19-mm Superpave mixes (5% -#200 / 4% air / 10 Hz frequency) with 4.8% of various PG asphalts are:

### PG 58-22

$$\text{Log } E_{\text{HMA}} = 4.062 - 0.018 T \quad (5)$$

### PG 64-22

$$\text{Log } E_{\text{HMA}} = 4.330 - 0.0198 T \quad (6)$$

### PG 70-22

$$\text{Log } E_{\text{HMA}} = 4.48 - 0.0198 T \quad (7)$$

Where:

$E_{\text{HMA}}$  - HMA Dynamic Modulus (ksi)

T - HMA temperature (°F)

### Condition 1

Condition 1 is achieved when the HMA pavement meets the PP criterion. The criterion is the estimated HMA strain for the maximum MMPT is  $\leq$  to the HMA FEL endurance limit.

The maximum HMA flexural strain is experienced when the HMA modulus is at its minimum value. The minimum HMA modulus occurs in the month with the highest MMAT. MMATs and MMPTs for some cities (Minneapolis, MN/ Springfield, IL / Orlando, FL) are shown in Table 3. For IL and MN, the maximum MMAT and MMPT occur in July. In FL, the MMAT and MMPT occur in August. The significant effect of location and PG grade are apparent. The importance of selecting the appropriate PG grade is obvious.

**TABLE 3 HMA Moduli for Select Locations and Different PG Asphalts**

City	MAX MMAT (°F)	MMPT* (°F)	PG 58-22 (E-ksi)	PG 64-22 (E-ksi)	PG 70-22 (E-ksi)
Minneapolis MN	73 (July)	83	370	485	685
Springfield IL	76 (July)	87	315	405	570
Orlando FL	83 (August)	94	235	295	415

\* Calculated for a 5-inch depth

PP HMA thickness design is a “direct calculation.” Establish the HMA modulus for the maximum MMPT ( “hottest month of the year”) and determine the HMA thickness (18-kip SAL) required to achieve a HMA flexural strain  $\leq$  the FEL. HMA PP thicknesses for the cities mentioned above are shown in Table 4 for various HMA moduli. The moduli for Minneapolis and Springfield are for the 19-mm Superpave mix with PG 64-22 asphalt. The Orlando HMA is a 19-mm Superpave mix with PG 70-22 asphalt. Note the small difference among the required PP thicknesses when the appropriate PG grade adjustment is applied.

**TABLE 4 HMA Thickness Requirements to Achieve FEL for Different Locations**

City	MAX MMAT (°F)	MMPT* (°F)	HMA E (ksi)	HMA FEL*** Thickness (inches)
Minneapolis MN	73 (July)	83	485*	12.8
Springfield IL	76 (July)	87	405*	13.9
Orlando FL	83 (August)	94	415**	13.8

\* PG 64-22 \*\* Pg 70-22 \*\*\* FEL = 70 micro-strain

NOTE: For subgrade  $E_{Ri} = 5$  ksi

**Condition 2**

In Condition 2, the HMA pavement does not meet PP criterion for the maximum MMPT, but there are months (at some MMPTs lower than the maximum MMPT) when the HMA flexural strains are less than the FEL. To consider this condition, the HMA fatigue life is estimated for each MMPT. The combined effects are considered based on Miner’s Cumulative Damage theory.

$$\sum_{i=1}^n N / N_i \quad (8)$$

Where:

n – Number of strain levels considered

N – Number of strain repetitions applied at a particular strain level

$N_i$  – Number of strain repetitions to failure at a particular strain level.

Failure occurs when the summation is equal to one. For strain repetitions < than the HMA FEL,  $N_i$  is “infinite” and those strain repetitions do not contribute to the accumulation of fatigue damage.

For a given pavement structure, the following equation can be used to predict the number of strain repetitions to failure,  $N_F$ .

$$N_F = 12 / \left( \sum_{i=1}^{12} [1 / N_a] \right) \quad (9)$$

Where:

$N_a$  – Number of strain repetitions to failure for the conditions of month “i”

$N_F$  – Number of strain repetitions to failure when traffic is applied throughout the 12 months of the year.

Condition 2 analyses were conducted for Minneapolis, MN, Springfield, IL and Orlando, FL. The following conservative HMA fatigue algorithm was utilized:

$$N = 2.65E-09 (1 / \epsilon_{HMA})^4 \quad (10)$$

A range of HMA thicknesses less than required to achieve a PP were evaluated. The results are shown in Table 5.

The FEL effect varies and the differences are limited. However, it appears the patterns are “location sensitive.” The greatest impact is noted for the highest MMAT location. The FEL effect for Condition 2 should be considered for specific inputs (including the selection of the HMA fatigue algorithm).

Table 5 data are for a specific fatigue algorithm. An equally, and perhaps more important, consideration for Condition 2 analyses is the assignment of the HMA fatigue algorithm. Recall the algorithm is of the form shown in equation 1:

$$N = K (1/\epsilon_{HMA})^n$$

As the exponent “n” in the algorithm increases, the relation “flattens” and the fatigue life becomes very sensitive to a small HMA strain difference.

Based on Carpenter’s extensive HMA fatigue testing at the University of Illinois, the following K-n relation shown in Figure 1 was established:

$$\text{Log } K = (1.1784 - n)/0.329 \quad (11)$$

**TABLE 5 Effect of the FEL on Estimated HMA Fatigue Failure**

CITY	HMA Thickness (inches)	N <sub>F</sub> – With FEL (ESALs*10 <sup>6</sup> )	N <sub>F</sub> – W/O FEL (ESALs*10 <sup>6</sup> )	Impact** (%)
Minneapolis	12.8*			
	11	22.3	19.7	13.2
	9	52.2	51.3	1.8
	7	9.68	9.61	0.7
Springfield	13.9*			
	12	231	193	19.7
	10	58.3	56.8	2.6
	8	12.9	12.7	1.6
	6	1.87	1.86	0.5
Orlando	13.8*			
	12	159	118	34.7
	10	36.9	34.4	7.3
	8	7.61	7.61	0

N<sub>F</sub> – Estimated HMA fatigue life for , HMA > FEL per:

$$N_F = 2.65E-09 (1 / \epsilon_{HMA})^4$$

\* - HMA thickness to achieve FEL = 70 micro-strain (Subgrade E<sub>Ri</sub> = 5 ksi)

\*\* N<sub>F</sub> increase (%) achieved by considering HMA FEL.

To illustrate the impact of the HMA fatigue algorithm selection, three “K-n” combinations were utilized to analyze a 10-inch FULL-DEPTH HMA for Springfield, IL. The results are shown in Table 6. The very large impact of the HMA fatigue algorithm selection is apparent! A small change in the K-n factors produces huge changes in the estimated HMA fatigue life. For Condition 2, the FEL impact is small compared to the fatigue algorithm effect.

**TABLE 6 Effect of HMA Fatigue Algorithm on Estimated HMA Fatigue Failure (10-inch HMA / Subgrade E<sub>Ri</sub> = 5 ksi)**

CITY	K/n	N <sub>F</sub> * – W/O FEL (ESALs*10 <sup>6</sup> )	N <sub>F</sub> * – With FEL (ESALs*10 <sup>6</sup> )	Impact** (%)
Springfield	8.78E-8/3.5	19.1	19.9	4.2
	2.65E-9/4	56.8	58.3	2.6
	8.02E-11/4.5	167	170	1.8

\* N<sub>F</sub> – Estimated HMA fatigue life for , HMA > FEL per:

$$N_F = K (1 / \epsilon_{HMA})^n$$

\*\* N<sub>F</sub> increase (%)

FEL = 70 micro-strain

## PAVEMENT CROSS-SECTION ADJUSTMENTS

The previous analyses were conducted for the condition where the same HMA constitutes the “FULL-DEPTH HMA” pavement. Many HMA designs include different HMA layers. Thus, the “multi-layer” pavement must be converted to an EQUIVALENT thickness of the bottommost HMA layer (where “bottom-up” fatigue damage accumulates).

The various HMA layer thicknesses can be converted to “equivalent thicknesses” of the bottom/fatigue resistant HMA layer using the Odemark Transformation. The Odemark Transformation is:

$$T_{EQ} = T_1 (E_1 / E_2)^{0.33} \quad (12)$$

Where

$T_{EQ}$  - Equivalent Thickness of layer 1

$T_1$  - Thickness of layer with  $E_1$  modulus

$E_1$  - Modulus of layer to be transformed

$E_2$  - Modulus of "reference" layer (the bottom HMA layer)

In many instances, the modular ratio for the HMA is near one and  $T_{EQ}$  is approximately equal to  $T_1$ . It is not recommended to use a modular ratio greater than 1. Any thickness deficiencies accumulated from the transformed thicknesses are accommodated by increasing the bottom HMA lift thickness a similar amount.

## SUMMARY AND RECOMMENDATIONS

- Current HMA fatigue and FEL technology are presented. The veracity of the HMA FEL is established. The FEL range is shown to vary from 70 to 100 micro-strain for typical neat binders and mixtures.
- HMA fatigue characterization for M-E pavement design should include a consideration of the HMA FEL.
- It is recommended that the 18-kip SAL be utilized for considering the FEL in HMA thickness design.
- HMA moduli for the “hottest month of the year” (maximum MMPT) should be utilized in PP thickness design.
- The impact of incorporating the FEL in HMA FULL-DEPTH HMA thickness design is demonstrated for two conditions. Condition 1 is achieved when the HMA pavement meets the PP criterion (HMA flexural strain is  $\leq$  FEL) for the maximum MMPT. Condition 2 applies when the HMA pavement **does not** meet the PP criterion for the maximum MMPT but there are months (at some HMA temperatures LOWER than for the hottest month) when the HMA flexural strains are less than the FEL.

- A general framework for considering the HMA FEL in routine/practical M-E flexible pavement design is presented and typical Condition 1 and Condition 2 applications are shown for Minneapolis, MN, Springfield, IL, and Orlando, FL.
- HMA PP thicknesses only varied from 12.8 to 13.9 inches. The Minneapolis and Springfield PP thicknesses were for PG 64-22 asphalt. The 13.8-inch thickness for Orlando, FL was achieved with a PG 70-22 asphalt.
- Asphalt PG grade selection is a very important factor in PP thickness design.
- The effect of considering the FEL for Condition 2 varies. The FEL effect is most pronounced for HMA thicknesses slightly less than the HMA PP thickness and is accentuated for higher MMATs. The maximum effect (34.7%) was obtained for the Orlando, FL location at the 12-inch HMA thickness..
- The large impact of selecting an HMA fatigue algorithm (K and n) is illustrated for a 10-inch FULL-DEPTH HMA pavement in Springfield, IL. For Condition 2, the HMA algorithm selection is more critical than considering the FEL effect.
- The selection of the HMA fatigue algorithm and the consideration of the HMA FEL are significant factors that definitely should be considered in HMA thickness design.
- The direct design of a HMA PP (as presented in this paper) is simple and straightforward. It eliminates the need to estimate traffic loadings.
- If HMA FEL data are not available, the use of a 70 micro-strain value is conservative.

## REFERENCES

1. Huddelston, J.; Buncher, M. and Newcomb, D., “Perpetual Pavements,” Asphalt Pavement Alliance (National Asphalt Pavement Association, Asphalt Institute, State Asphalt Pavement Association Executives), 2000.
2. Myre, J., “Fatigue of Asphalt Materials for Norwegian Conditions”, Seventh International Conference on Asphalt Pavements, Vol. 3 Proc., U.K., 1992.
3. Monismith, C. L. and D. B. McLean, “Technology of Thick Lift Construction: Structural Design Considerations, *Proceedings, Association of Asphalt Paving Technologists, Vol 41, pp. 258-304* 1972.
4. Carpenter S. H. K. Ghuzlan, and S. Shen, “Fatigue Endurance Limit for Highway and Airport Pavements,” Transportation Research Record: Journal of the Transportation Research Board, No. 1832, TRB, National Research Council, Washington, D.C., 2003, pp. 131-138.
5. Shen, S. and S. H. Carpenter, “Application of Dissipated Energy Concept in Fatigue Endurance Limit Testing,” Transportation Research Record, Journal of Transportation Research Board, No. 1929, pp. 165 – 173, 2005
6. Carpenter, S. H., and S. Shen, “A Dissipated Energy Approach to Study HMA Healing in Fatigue,” Accepted for Publication in the Journal of the Transportation Research Board, 2006.
7. Thompson, M. R., and S. H. Carpenter, “Fatigue Design Principles for Long Lasting HMA Pavements,” ISAP International Symposium on Long-Lasting Asphalt Pavements, Auburn University, June 7-8, 2004
8. ERES Consultants, Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures. NCHRP 1-37A Project Report, March, 2004.
9. The Asphalt Institute, Research and Development of The Asphalt Institute’s Thickness Design Manual (MS-01). 9<sup>th</sup> Edition. Research Report 82-2. 1982.
10. Fonesca, O.A., and M. W. Witzak, “A Prediction Methodology for the Dynamic Modulus of In-Place Asphalt Mixtures,” Journal of the Association of Asphalt Paving Technologists, 1996.
11. Bari, J. and M. W. Witzak, “Development of a New revised Version of the Witzak E\* Predictive Model for Hot Mix Asphalt Mixtures”, Paper presented at the 2006 Annual Meeting of AAPT.

12. Christensen, D.W., T. Pellien, and R. F Bonaquist, "Hirsch Model for Estimating the Modulus of Asphalt Concrete," Journal of the Association of Asphalt Paving Technologists, 2003.
13. Witczak, M.W., "Design of Full Depth Asphalt Airfield Pavements," Proceedings - Vol. I, Third International Conference on the Structural Design of Asphalt Pavements, 1972.
14. Hill, H. and M. Thompson, "Early Life Study of the FA409 Full-Depth Asphalt Concrete Pavement Sections," Unpublished Research Report, University of Illinois at Urbana-Champaign, Illinois Department of Transportation, Illinois Cooperative Highway and Transportation Research program, June, 1988
15. Thompson, M.R., B.J. Dempsey, H. Hill, and J. Vogel, "Characterizing Temperature Effects for Pavement Analysis and Design," Transportation Research Record 1121, Transportation Research Board, 1987.