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FATIGUE PERFORMANCE OF IDOT MIXTURES

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16. Abstract
The fatigue characteristics of an HMA mixture are an integral element of the structural design of a pavement containing asphalt concrete. There are a number of models that have been developed to express the fatigue behavior of an HMA as a function of mixture composition. These models are presented and the shortcomings are discussed relative to the IDOT tensile strain model of the form $K1(1/\varepsilon)^{K2}$. Two sets of IDOT mixtures were investigated for this study:

- The first was a general, pre-Superpave selection of IDOT mixtures.
- The second was a set from the Extended Life Hot Mix Asphalt Pavement ELHMAP project, representing Superpave mixtures and rich bottom binder (RBB) mixtures.

The fatigue model coefficients of these tests are examined and suggestions relative to acceptable coefficients for IDOT use are made, specifically:

- K2 = 3.5 to include all mixtures tested.
- K2 = 4.0 to include nearly ninety percent of mixtures tested.
- K2 = 4.5 for the average value of mixtures tested.

The test results clearly indicate the interrelationship between K1 and K2 that must be observed to avoid significant errors in fatigue life prediction given that all models in current use have a constant K2 regardless of the K1 value calculated from mixture variables.

The testing also examined the Fatigue Endurance Limit (FEL) of these two data sets which shows that binder type may influence the limit, but in no cases was a strain below 70 micro-strain required to get an extraordinarily long fatigue life. The effect of the RBB mixtures on fatigue resistance was positive but marginal.

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EXECUTIVE SUMMARY

The fatigue characteristics of an HMA mixture are an integral element of the structural design of a pavement containing asphalt concrete. There are a number of models that have been developed to express the fatigue behavior of an HMA as a function of mixture composition. These models are presented and the shortcomings are discussed relative to the IDOT tensile strain model of the form $K1(1/\varepsilon)^{K2}$. Two sets of IDOT mixtures were investigated for this study:

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FATIGUE PERFORMANCE OF IDOT MIXTURES

CHAPTER 1 INTRODUCTION

Fatigue is a phenomenon in which a pavement is subjected to repeated stress levels less than the ultimate failure stress. Hveem 1955 was one of the first researchers who reported fatigue failure caused by repeated loading on asphalt pavement over highly resilient soils. He concluded that there was a correlation among observations of cracking, fatigue-type failures in bituminous pavements, and the measured deflections which the pavement must undergo with each passing wheel. Hveem suggested a well-designed pavement must be capable of withstanding deflections or have sufficient stiffness to reduce the deflections to permissible levels.

Beyond the characterization for a well-designed pavement that resists fatigue damage over its useful life, the newer consideration is the validation of the long-proposed mechanism of a fatigue endurance limit first put forth by Monismith and Deacon, 1969. This concept postulates that there is a strain limit below which a hot mix asphalt (HMA) pavement will not experience any damage from the traffic loadings. It was postulated that from examinations of existing data at that time that there appeared to be a limit of around 70 micro-strain, but there has been no previous laboratory confirmation of this for an HMA.

Thus, there are two considerations in fatigue design for an HMA pavement. First, the use of fatigue algorithms or models representing fatigue resistance for an HMA mixture that are used to select HMA thicknesses to resist the applied traffic. Secondly, a validation of the fatigue endurance limit as an existing principle in HMA performance.

This report presents and discusses current models utilized for fatigue design and provides current Illinois Department of Transportation (IDOT) mixture data to support appropriate model use. Fatigue endurance limit testing will also be presented to illustrate current mixture behavior under low strains.

CHAPTER 2 FATIGUE MODELS

There are several fatigue models that have been developed around the world to predict fatigue cracking. Fatigue models are typically divided into two main types, the strain-based models and the strain-modulus based models. These models are termed phenomenological models as they present observed relationships that have not been derived from a theoretical analysis of pavement mechanics.

Longitudinal cracking observed on the surface of a flexible pavement is commonly considered as being related to traditional fatigue except for temperature cracks and long-term weathering cracks, and the newly proposed mechanism of top-down cracking. Laboratory testing clearly shows there is a relation between the tensile strain at the outer fiber in a flexural beam test and the fatigue life. This observed relationship has been extended to the pavement, proposing that the tensile strain at the bottom of the asphalt concrete layer (ε_o) is related to the number of load applications to crack appearance in the pavement (N_f). This relation has been expressed in the following form, Pell 1987:

$$N_f = K1\ (1/\ \varepsilon_o)^{K2} \qquad (1)$$

Bonnaure et al. 1980 and Finn et al. 1977 noted differences in the coefficients of this equation for different temperatures. They proposed a fatigue formula using modulus as follows:

$$N_f = K1(\ 1/\varepsilon_o)^{K2}\ \ (E^*)^{\ b} \qquad (2)$$

Where:

E* is the dynamic stiffness modulus of the HMA,
ε_o is the tensile strain,
K1, K2, and b are fitting coefficients to the data.

Introducing the dynamic stiffness modulus into the fatigue relation is felt to quantify some of the differences in coefficients seen in lab testing for temperature variations.

To account for strain variations, the Miner's hypothesis of damage has been used in conjunction with these phenomenological fatigue relationships, Miner 1945. Miner's hypothesis is represented as a relative damage factor where the crack will occur when the sum of the damage factors equals one. Miner's damage hypothesis is given as follows:

$$Di = ni/\ Ni \qquad (3)$$

Where:

Di = relative damage during some period i,
ni = number of load applications during a period i, and
Ni = the ultimate number of load applications the pavement could carry.

To use Miner's approach, Ni is determined from the fatigue equation.

There are three coefficients that need to be determined: K1, K2, and b as shown in equations (1) and (2). Different values are found in literature for the (b) coefficient. Finn et al. 1977 found the (b) coefficient to be -0.854 while Bonnaure et al. 1980 found the (b) coefficient could have two values –1.4 and –1.8 based on mode of loading. The K2 coefficient has values generally more than 3, while the K1 values varied significantly between different agencies as shown in the next section.

Surrogate models of fatigue behavior were developed in the SHRP A-404 study, SHRP 1994. Both strain dependent or energy dependent models could be used for surrogate mix analysis. In the case of the strain-dependent model, the initial strain, the initial loss modulus, and the voids filled with asphalt in the mix are required to use the model. When the energy-based model is considered, the initial dissipated energy is needed in addition to the voids filled with asphalt in the mixture. Furthermore, viscoelastic analysis is needed for such types of models. These models are more suited to research investigations and do not lend themselves as a readily usable methodology to select pavement thickness.

These basic models have served as the framework for various agencies in calibrating these models to their specific pavements and mixtures. Given below are the fatigue models for several agencies with some details about the coefficients developed and the assumptions used.

2.1 ILLINOIS DOT / UNIVERSITY OF ILLINOIS MODEL

Elliott and Thompson, 1986 used the deflection-based performance equations from the AASHO road test to estimate the value of the (b) coefficient. There are two performance deflection equations in the AASHO Road Test. One was for the number of load repetitions to a Present Serviceability Index of 2.5 (the point at which most of the major highways are rehabilitated) and the other equation for Present Serviceability Index of 1.5 (the point at which the road test pavements were removed from test). These two equations are based on the spring normal Benkelman beam deflection. They developed an algorithm that relates load repetitions to failure with the surface deflection:

$$N_{18} = 5.6\times10^{11} / \Delta^{4.6} \qquad (4)$$

Where:

N_{18} = Number of 18-kip loads to fatigue failure, and

Δ = Surface deflection (mils) for 18-kip axle load (Benkelman Beam).

This relation was substituted in the fatigue equation and the final estimate of the K2 coefficient was found to be 2.92 and 3.27 for each equation. Values of the K2 and (b) coefficients were used with the road test data and the design algorithm for asphalt strain to calculate an average K1 value. The design algorithm was developed based on statistical analysis of extensive literature data. In the design algorithm, the AC strain was based on material thickness and the modulus value. The final estimate of K1 was found to be 2.234. Based on this analysis, the following fatigue model was developed by Elliot and Thompson 1986:

$$\text{Log } N = 2.4136 - 3.16 \times \text{Log } \varepsilon - 1.4 \times \text{Log } Eac \qquad (5)$$

Where:

N = number of load repetitions to cracking,

ε = predicted AC strain (in/in), and

Eac = AC dynamic stiffness modulus (psi).

The typical fatigue relation used by the Illinois Department of Transportation, from Thompson 1987 is :

$$N_f = K\left(\frac{1}{\varepsilon_{AC}}\right)^n \qquad (6)$$

Where:

K, n = factors depending on the composition and properties of the AC mixture, which have been replaced in the nomenclature by K1 and K2, respectively, in current equations.

For typical Illinois Department of Transportation dense-graded class I type mixtures the values of K1 (K in Equation 6) and K2 (n in Equation 6) were initially set at 5×10^{-6} and 3.0 respectively. These were established after a thorough study of the literature and utilization of the Asphalt Institute equations to examine Illinois binder mixtures being used at the time. It may be possible to establish K1 and K2 values for other types of mixtures based on mixture composition as shown by Bonnaure et al. 1980 and split tensile strength characteristics as shown by Maupin and Freeman, 1976 although this has not been achieved to date but is under study for the final report. This fatigue relation was utilized in the Illinois DOT thickness design procedures from Thompson and Cation, 1986, and recently the exponent K2 has been raised to 3.5, and K1 correspondingly set at 8.2 E-08 for rubblized sections based on data from this study.

2.2 SHRP A-404 MODEL

An accelerated performance test for defining the fatigue response of asphalt-aggregate mixes and their use in mix analysis and design systems was developed in SHRP A-404, SHRP 1994. A model was developed to predict fatigue life of asphalt concrete mixtures. The effect of the following variables on fatigue performance of asphalt concrete mixtures was investigated in this study: asphalt type (8 types), aggregate type (2 types), asphalt content, air-void content (2 levels), strain levels (2 levels), replicates (2 replicates), frequency (10 Hz), and test temperature (20° C). The experiment is 8×2 where it includes 8 different asphalts and 2 different aggregates. Based on the experiment, the following model is obtained:

$$N_f = 466.4 e^{0.052VFB} (\varepsilon_0)^{-3.948} (S_0^{"})^{-2.270} \qquad (7)$$

Or

$$N = 6.72 e^{0.049VFB} (w_o)^{-2.047} \qquad (8)$$

Where:

N_f = fatigue life,
ε_o = initial strain (in/in),
$S_o^{"}$ = initial loss stiffness (psi),
W_o = initial dissipated energy per cycle, and
VFB = percentage of voids filled with bitumen.

2.3 THE ASPHALT INSTITUTE MODEL

The fatigue relation for the Asphalt Institute (AI) was developed based on laboratory fatigue data for selected sections of the AASHO road test by Asphalt Institute 1982, and Finn et al. 1977. The following fatigue relation was developed by the Asphalt Institute 1982:

$$N_f = 18.4 \times (C)(4.325\text{ E-3})(\varepsilon_t)^{-3.291} \left| E^* \right|^{0.854} \qquad (9)$$

Where:

N_f = number of 18,000 lb equivalent single axle loads,
ε_t= tensile strain in asphalt layer, (in/in),
$\left| E^* \right|$ = asphalt mixture dynamic modulus (psi), and
C = function of volume of voids and volume of asphalt.

The fatigue relationship in Equation (9) was modified to reflect the effect of the air voids and asphalt content in the asphalt mixture. This was done by introducing the correction factor (C), equal to:

$$C = 10^{M}$$

Where:

$$M = 4.84\left(\frac{Vb}{Vv + Vb} - 0.69\right)$$

Where:

Vb = volume of asphalt, percent, and
Vv = volume of air voids, percent.

The M value was obtained from laboratory fatigue data developed by Pell and Cooper 1975. The value of C was set to be 1 when the volume of binder equals 11 and the volume of air voids equal 5. It can be noted that the fatigue life is reduced by increasing the air voids content or reducing the asphalt content in the asphalt mixture.

In Japan, a fatigue equation similar to the AI relation was used with some changes in the voids and asphalt factor, Nishizawa, et al. 1997.

2.4 SHELL PAVEMENT DESIGN MANUAL MODEL

The SHELL fatigue criterion is based on strain and modulus. The following formula is used to predict fatigue life from Shell 1978:

$$N = 4.91 \times 10^{-13} \, (0.86\, V_b + 1.08)^{5.0} \, (1/\varepsilon)^{5.0} \, (1/S_{mix})^{1.8} \qquad (10)$$

Where:

N = number of load cycles to failure,
V_b = volume of asphalt in the mixture (%),
ε = maximum tensile asphalt concrete strain, (in/in), and
S_{mix} = dynamic modulus of the asphalt mixture, (ksi).

This fatigue relation was developed based on the fatigue formula given by Van Dijk and Visser, 1977, which was obtained using a wheel tracking machine on asphalt slabs. Controlled strain testing was used, and it was concluded that the crack patterns observed were similar to those obtained in practice. Therefore, results from controlled strain testing were used as the fatigue criterion in the SHELL procedure.

2.5 TRANSPORT AND ROAD RESEARCH LABORATORY – TRRL UNITED KINGDOM MODEL

The TRRL fatigue criterion was developed after TRRL report 1132, Powell et al. 1984, and is based on the field performance of several experimental flexible pavements. A multi-layer elastic model was used to calculate the dynamic strains.

The performance for 34 sections of experimental road with well-compacted, dense road base macadam pavements and 29 sections of experimental rolled asphalt road base were monitored. The fatigue design curve was produced based on laboratory fatigue testing over a range of temperatures and levels of dynamic strain. It is widely known that laboratory fatigue tests underestimate the fatigue life of pavements; therefore, a shift factor is routinely needed to correlate to field performance. Mixed traffic loading and pavement temperature conditions were considered. The accumulation of fatigue damage was calculated based on Miner's hypothesis. Considerable adjustment was needed to correlate between laboratory fatigue relations and field performance.

The design life could be calculated using the following relationships:
For 85% probability of survival and an equivalent temperature of 20° C (68° F):

Dense bitumen macadam (100 pen):

$$N_f = (4.169 \times 10^{-10})(1/\varepsilon_r)^{4.16} \qquad (11)$$

Hot rolled asphalt (50 pen):

$$N_f = (1.660 \times 10^{-10})(1/\varepsilon_r)^{4.32} \qquad (12)$$

Where:

N_f = the road life in standard axles, and
ε_r = the horizontal tensile strain at the underside of the bound layer under a standard wheel load.

2.6 THE MOBIL PAVEMENT DESIGN MANUAL (U.K.) MODEL

The maximum tensile strain at the bottom of the asphalt base was used in the Asphalt design manual for the United Kingdom to design against fatigue cracking, Mobil Oil, 1985. The fatigue relation used in the manual is based on the Nottingham procedure, Brown et al. 1977 and Brown 1980. The traditional fatigue relation $N_f = K1(1/\varepsilon)^{K2}$ was used. The coefficients in this equation were based on two main mix parameters, the volumetric proportion of binder and its initial softening point.

The following equation relates the number of load cycles to failure to the tensile strain from Brown 1980:

$$\text{Log } N_f = 15.8 \text{ Log } \varepsilon_t - k - (5.13 \text{ Log } \varepsilon_t - 14.39) \text{ Log } V_B - (8.63 \text{ Log } \varepsilon_t - 24.2) \text{ Log } SP_i \qquad (13)$$

Where:

k = 46.82 for life to critical conditions,
k = 46.06 for life to failure conditions (pavement life) ,
N_f = number of load cycles to failure (pavement life),
ε_t = tensile strain (micro-strain),
V_B = volumetric proportion of binder (%), and
SP_i = initial softening point of binder.

Failure conditions were defined as a 20 mm rut or extensive cracking in the wheel tracks, while critical conditions were characterized by a 10 mm rut or the first appearance of wheel path cracks. The above relation was developed based on laboratory fatigue testing of a wide range of mixes. Pell and Cooper, 1975 noted that there was a linear relation between the two fatigue relation constants K1 and K2. This relation is given as:

$$K2 = 0.5 - 0.313 \log K1 \qquad (14)$$

To account for difference in conditions between laboratory and field, a shift factor was used. The shift factor was 77 for critical conditions and 440 for failure conditions by Brunton et al. 1987.

2.7 THE BELGIAN ROAD RESEARCH CENTER MODEL

A comprehensive design procedure for asphalt pavements was developed for Belgium by Verstraeten et al 1977 and 1982. This procedure considers both fatigue cracking and rutting. The fatigue relation obtained from laboratory fatigue testing using sinusoidal stress without rest period is the traditional strain model shown in equation (1).

The K1 constant was found to be dependent on the mix composition but independent of the temperature and frequency. The K2 coefficient was found to be almost constant and equal to 4.76. This analysis was based on 42 different mixtures. Based on mix composition, the following fatigue relation can be used, Francken and Verstraeten, 1974:

$$N_f = 10^6 \{ \varepsilon/(A^*G) [(V_B+V_v)/V_B]\}^{3.85} \quad (15)$$

Where:

N_f = number of load applications to failure,
ε = initial strain,
A = coefficient that depends on the asphaltene content in the bitumen,
G = empirical factor that depends on the gradation of the aggregate; for most mixes used in road construction G = 1,
V_B = bitumen content by volume (%), and
V_v = voids content (%).

For practical design purposes the following fatigue equation is used in Belgium:

$$N_f = 4.92 \times 10^{-14} (1/\varepsilon)^{4.76} \quad (16)$$

2.8 THE NATIONAL ROAD DIRECTORATE OF DENMARK MODEL

Ullidtz 1977 proposed the fatigue criterion for major roads in Denmark. The elastic theory was used to calculate critical stresses and strains using the falling weight deflectometer test. The proposed equation is:

$$N_f = 2.94 \times 10^{-20} \times V_B^{\ 5.62} \times (1/\varepsilon)^{5.62} \quad (17)$$

Where:

N_f = number of load cycles to failure,
V_B = percentage of bitumen by volume, and
ε = tensile strain at the bottom of the asphalt layer.

The above equation was determined from laboratory fatigue tests, corrected for the influence of rest periods, and the allowable normal stress on the subgrade from the analysis of the WASHO Road Test, the AASHO Road Test, and the CBR-curves. The failure criterion for these allowable values is fatigue cracking of the HMA layer and the minimum acceptable PSI value.

2.9 NORWEGIAN FATIGUE CRITERIA

Myre 1992 used three different test methodologies to conduct fatigue testing on several mixes in Norway. Ten different mixtures were included in this study, and 464 samples were tested. The apparatus used included: 4-point repeated bending apparatus, center-point loaded beam (CPB) on a rubber base, and the indirect tensile test apparatus. The test temperature was generally 5° C; however, some fatigue curves were obtained at 15° C and 25° C. The Norwegian fatigue criterion was developed based on results from the CPB apparatus. The analysis of 336 specimens produced the following fatigue equation:

$$\text{Log } N_f = 34.5326 - 6.1447 \times \text{Log } \varepsilon - 3.395 \times \text{Log } E + 0.3864 \times \text{Log } VB \times MF - 0.0788 \times Vv \quad (18)$$

(Coefficient of correlation = 0.92, and SEE = 0.445)

Where:

N_f = number of load cycles to failure,

ε = tensile strain at the bottom of the asphalt concrete layer,
E = elastic modulus,
VB = binder content,
V_V = void content, and
MF = mode factor, -1< MF < 1
= 1.99 – 3.37 × A/B – 0.00342 × B + 0.004 × A + 0.00153 × Esg

Where:

$$A = \sum_{i=1}^{i=n} (h_i . \sqrt[3]{E_i})$$

$$B = \sum_{i=1}^{i=m} (h_i . \sqrt[3]{E_i})$$

h_i = thickness of layer number i (cm)
E_i = E-modulus layer number i (Mpa)
Esg = Subgrade E-modulus (Mpa)
n = the lowest asphalt layer below the surface
m = subbase

The parameter A is the sum of the layer thickness multiplied by the third root of the corresponding E-modulus of each of the bituminous layers, whereas B is the sum of the layer thicknesses multiplied by the third root of the corresponding E-modulus for all layers in the pavement structure.

2.10 FRENCH FATIGUE CRITERIA

The fatigue criterion in France is based on the strain amplitude, which induces half a rigidity (50% reduction in the complex modulus E*) at one million cycles considering a constant strain amplitude. The test protocol is a two points bending, cyclic test, on prismatic samples as shown in Di Benedetto et al. 1996, De La Roche and Riviere 1997, and Odeon and Caroff 1997.

The following fatigue relation is used:

$$N_f = 10^6 \times \left[\frac{\varepsilon_{cal}}{k \times \varepsilon_6(\theta)} \right]^{1/b} \qquad (19)$$

Where:

ε_{cal} = strain calculated in the structure equivalent to circuit of one load,
$\varepsilon_6(\theta)$ = strain causing the failure of the sample after 10^6 applications of the loading stress/strain at temperature (θ ° C),
b = slope of the fatigue curve,
N_f = theoretical life duration of the structure, and
k = shift factor translating laboratory and test track.

According to the French standard, fatigue testing is conducted under controlled strain, two-point bending on trapezoidal specimen, with no rest period, De La Roche and Riviere, 1997. The elastic layer theory is used to calculate the stresses and strains in the pavement. The shift factor (k) was obtained based on laboratory fatigue data and observations from the LCPC track, and it varied between 0.8 and 3.87. The slope of the fatigue curves (1/b) in this study varied from 3.4 to 9. Fatigue failure in the test track was defined at 50% cracking. Based on extensive laboratory testing and comparison with the test track, it seems the controlled strain fatigue testing gives more reasonable fatigue lives than the controlled stress testing.

2.11 PDMAP – NCHRP PROJECT 1-10B

The PDMAP program (Probabilistic Distress Models for Asphalt Pavements) was developed to enable the highway engineers to predict distress conditions of given pavement sections. The two main distresses considered are fatigue cracking and rut depth. Exact prediction of fatigue and rut depth is not possible because of uncertainties in the measurements of material properties, in the estimation of traffic, and in the damage model itself. Therefore, the PDMAP program employs probabilistic analysis, which computes the expected amount of damage with specified reliability factor at any time during the analysis period, Finn et al. 1977.

The prediction model for fatigue cracking used in PDMAP is based on the fatigue testing done by Monismith et al. 1970 as follows:

$$\text{Log } N_f = 14.82 - 3.291 \text{ Log } (\varepsilon/10^{-6}) - 0.854 \text{ Log } (|E^*|/10^3) \quad (20)$$

Where:

N_f = load applications of constant stress to cause fatigue failure,
ε = initial strain on the bottom of the asphalt concrete, and
$|E^*|$ = complex modulus (psi).

Cracking and rutting observations from the AASHO Road Test were used to calibrate the above equation. A shift factor of 13.0 was used for the 10 percent cracking and 18.4 for the 45 percent cracking. Cracking is defined as the percentage of the wheel path area exhibiting class 2 cracking. The following fatigue prediction models were obtained using materials similar to those used at the AASHO Road Test by Finn et al. 1986:

$$\text{Log } N_f (10\%) = 15.947 - 3.29 \text{ Log } (\varepsilon/10^{-6}) - 0.854 \text{ Log } (|E^*|/10^3) \quad (21)$$

$$\text{Log } N_f (45\%) = 16.086 - 3.29 \text{ Log } (\varepsilon/10^{-6}) - 0.854 \text{ Log } (|E^*|/10^3) \quad (22)$$

2.12 NCHRP 1-37A CALIBRATED FATIGUE MODEL

This model contains significant modifications to the standard form of the fatigue equation, but still relies on the basic strain-modulus form. Because thick and thin pavements exhibit different behavior when analyzed with the standard phenomenological model, changing from constant strain in a thin pavement to constant stress in a thick HMA layer, the 1-37A research team elected to add a variable to change coefficients as the HMA layer becomes thicker. This model takes its basic form from the Asphalt Institute equation. An extensive calibration process using field data and LTPP sections was conducted to establish the coefficients for different mixtures and different parts of the United States. The final form of the model from El-Basyouny and Witzcak 2005 is:

$$N_f = \beta_{f1} \, k_1 (\varepsilon_t)^{-\beta_{f2} k_2} (E)^{-\beta_{f3} k_3} \quad (23)$$

Where:

Nf = Number of load repetitions to fatigue failure
εt = tensile strain at the critical location
E = the dynamic modulus of the HMA
k_1, k_2, k_3 = Laboratory regression coefficients
β_{f1}, β_{f2}, β_{f3} = Calibration parameters

Basically, the exponents, k_2 and k_3, are constants, and the coefficient k_1 contains the mixture variables. Other coefficients are included for constant stress to constant strain considerations. The calibration parameters are designed to reduce the bias and scatter in the prediction.

2.13 Summary of Asphalt Concrete Fatigue Relations

As mentioned earlier, two main forms of fatigue relation have been used in mechanistic-pavement design procedures. These two forms are: strain-based fatigue relation and strain/modulus fatigue relation. As shown in the following summary list, many agencies have adopted the strain-based fatigue criteria:

Illinois DOT / University of Illinois, Thompson 1987,
The Transportation and Road Research Center in the U.K, Powell et al. 1984,
The Mobil Pavement Design Manual for the U.K , Mobil Oil 1985,
The National Road Directorate of Denmark, Ullidtz 1977,
The Belgian Road Research Center, Verstraeten et al. 1977, Francken and Verstraeten, 1974, and
The French Procedures, LCPC, Di Benedetto et al. 1996, De La Roche and Riviere, 1997, and Odeon and Caroff 1997.

The strain/modulus-based fatigue algorithms are utilized in the Asphalt Institute 1982, Shell Pavement Design Manual 1978, the Norwegian Pavement Design Procedures from Myre 1992, and the NCHRP 1-37A mechanistic empirical pavement design guide from El-Basyouny and Witczak 2005. The relative impacts of the “strain” versus “modulus” factors on fatigue life were discussed in NCHRP 1-26 1992, and it was shown that for both the AI and SHELL algorithms the “strain” is dominant. The net effect of the AC modulus term is less than 5 percent. However, it is not appropriate to conclude that one relation is right and the other is wrong. As shown in NCHRP 1-26 1992, any decision of which form to use ultimately rests on decisions regarding data collection concerns and considerations of relative accuracy in prediction levels. Until standardized modulus testing is more universally available, the strain model presents a comprehensive model with the best widespread applicability to state agencies.

2.14 Discussion of Fatigue Model Coefficients

The fatigue models presented here all derive from the same basic form relating tensile strain to the number of load repetitions to a pre-defined failure state. The main differences in these forms are in the coefficients. The K1 coefficient has consistently shown a strong relationship with mixture variables and composition. Air voids and asphalt content appeared most frequently in the different agencies’ formulations, clearly reflecting the differences in mixture composition for the different agencies. Of significant importance in the examination of these models is that every model assigned a constant value to the K2 coefficient. While different agencies assigned different values, it is interesting to note that everyone felt that a constant K2 value, not related to K1, was appropriate.

The assignment of a constant K2 value is significant since none of the databases developed or used in the analyses support a constant value for K2. In fact, all previous data support a consistent phenomenological relationship between K1 and K2 as shown by Ghuzlan and Carpenter, 2000. As will be presented later, all fatigue data clearly support a single relationship that is not dependent on testing mode, sample shape, and so on.

These historical data clearly indicate that different mixtures will not only have different K1 coefficients, but they **must** have different K2 exponents for any mix that is assigned a different K1 value. These values are highly correlated and one cannot change without the other changing appropriately. To arbitrarily assign coefficients invites a disconnect between different mixtures and would, on the surface at least, appear to nullify any calibration efforts.

The next section will present fatigue testing of various IDOT mixtures to illustrate the level of conformity with accepted models and to illustrate mixture variability in coefficients found in typical mixtures to provide support for selecting typical values to use in pavement design.

CHAPTER 3 IDOT MIXTURES

The current fatigue model used by IDOT in its mechanistic design for full-depth and rubblized HMA construction was presented previously and uses the standard strain format. The testing program for IDOT mixtures is contained in two data sets. The first set of data was collected on 25 different mixtures obtained from the field during paving from different districts in the State of Illinois during 1996 and 1997 as part of a gyration-level study being conducted for IDOT's Superpave implementation studies. The fatigue testing was conducted on another project examining damage characteristics of fatigue testing. These mixtures were designed with either the standard Marshall mix design procedure or the Superpave mix design procedure. The binders used were combinations of viscosity grading and the new Superpave PG grading system. Two air void levels were used for the testing on Set One, 4 percent and 7 percent.

Table 1 shows the physical properties including asphalt type, asphalt content, maximum theoretical specific gravity of asphalt mixture, and the bulk specific gravity of aggregate.

The second set of mixtures was sampled from the trucks at the plant during the 2003-2004 construction season as part of the ELHMAP study being conducted for IDOT. The 21 mixtures sampled included 10 binder mixtures. These mixtures were all Superpave mix designs using a variety of gyration levels. All mixtures used the PG graded binders. Table 2 presents the physical properties of these mixtures. The air void level selected for these tests was 7 percent.

These mixtures were also altered by adding a nominal extra 0.5 percent asphalt cement to produce the rich bottom binder (RBB) mixture being proposed for the Extended Life Hot Mix Asphalt Pavements (ELHMAP). The binder used for this alteration was the binder sampled from the plant during the production cycle when the mixtures were sampled. These RBB mixtures were compacted to a nominal 3 percent air voids. The mix properties produced with this extra asphalt cement are given in Table 3. Nine of the mixtures were used for RBB fatigue testing. The mix identification for these mixtures has an additional P added, indicating plus asphalt.

Table 1. IDOT Mixture Properties – Set One

Mixture ID	Gmm	Opt. AC%	Gsb	NMS	AC Type	BIT No.	Year
1	2.510	4.6	2.617	19	PG 70-28	85BIT3400	1997
2	2.522	5.6	2.648	9.5	PG 70-28	87BIT7105	1997
3	2.517	5.5	2.637	9.5	AC-20	85BIT0731	1997
4	2.499	4.6	2.642	19	PG 70-28	85BIT3402	1997
5	2.489	4.6	2.609	19	AC-20	85BIT0729	1997
6	2.453	5.4	2.588	9.5	PG 64-22		1997
7	2.526	4.4	2.652	12.5	PG 58-28	84BIT000S	1997
8	2.515	5.3	2.646	9.5		84BIT001S	1997
9	2.488	5.4	2.608	9.5		83BIT6232	1997
10	2.508	4.5	2.624	19	PG 64-22	83BIT6230	1997
11	2.468	5.7	2.587	12.5		83BIT6226	1997
12	2.518	5.6	2.648	9.5		87BIT7105	1997
13	2.505	4.3	2.618	19	PG 64-22	87BIT0493	1997
14	2.496	4.8	2.611	19	PG 58-28	82BIT2158	1997
15	2.477	5.9	2.616	9.5	PG 58-28	82BIT2159	1997
16	2.478	5.5	2.601	9.5	AC-10	82BIT1379	1997
17	2.527	5.7	2.707	9.5	AC-10		1997
18	2.536	5.1	2.708	12.5	AC-10		1997
19	2.493	5.6	2.628	9.5	AC-20		1997
20	2.734	5.0	2.784	12.5			1997
21	2.466	4.2	2.628	19			1997
D6-B-96	2.427	5.3	2.58	19		64BIT1349	1996
D6-S-96	2.479	5.3	2.635	9.5		64BIT1350	1996
D7-B-96	2.518	5.1	2.661	19		87BIT7155	1996
D5-B-96	2.5	4.7		19		85BIT0726	1996

Table 2. Mix Properties for the ELHMAP Binder Mixtures for Fatigue Testing

Mix ID	Asphalt Type	Asphalt Grade	Anti Strip	AC content	Agg. Type	Mix #	Ndesign	Gmm
1N105	polymer	SBS PG70-22	No	6.50	Dolomite	81bit 054H	N105	2.473
1N80D	SMA, polymer	SBS 76-28	No	8.04	crushed dolomite	81bitGA10	N80	2.483
2N90	polymer	SBS PG70-22	No	5.90	Dolomite	82bit 2491	N90	2.527
3N70	Neat	PG64-22	No	5.17	Limestone	83bit 016R	N70	2.484
3N90	polymer	SBS PG76-22	0.5% AD-HERE LOF 65-00 LS**	5.06	Dolomite	83bit 018Z	N90	2.484
3N90T	Neat	PG64-22	No	4.06	limestone and RAP	83bit 002T	N90	2.51
5N105	polymer	SBS PG76-28	0.1% lime by weight of Agg.	4.70	Limestone	85bit 3430	N105	2.483
5N90	Neat	PG64-22	0.5% Pave Bond Lite	4.90	Dolomite	85bit 3603	N90	2.540
6N50	Neat	PG64-22	0.7% Pave Bond Lite	5.05	Limestone	86bit 3312	N50	2.45
8N70	Neat	PG64-22	No	4.93	Limestone	88bit 1928	N70	2.496

**No anti strip indicated by District; mix design and plant paper indicated use.

Table 3. Mix Properties for RBB Mixtures

Mix ID	AC	Air Voids	Gmm
1N105P	6.94	2.4	2.439
1N80DP	8.97	2.6	2.473
2N90P	6.78	3.1	2.509
3N70P	5.62	2.5	2.485
3N90P	5.45	3.8	2.492
3N90TP	4.72	3.2	2.5
5N90P	5.53	2.9	2.525
6N50P	5.55	3.1	2.447
8N70P	5.23	3.6	2.481

CHAPTER 4 EXPERIMENTAL SETUP AND SAMPLE PREPARATION

Fatigue testing is time consuming, since it may take several days or weeks to establish a fatigue curve for a specific mix depending on the desired fatigue characteristics. There are several methodologies for measuring the fatigue behavior of HMA. AASHTO T321-03 adopts the simple flexural setup in constant strain to evaluate fatigue performance. Therefore, flexural beam fatigue equipment was used in this study. The first step in fatigue testing is compacting the asphalt concrete beams to get the desired voids and a desirable aggregate structure. Specimens have to be fabricated in such a way that duplicates field conditions in as many aspects as possible such as composition, density, and engineering properties through proper compaction. Several compaction methods might be used such as static compaction, impact compaction, rolling beam compaction (full scale), and rolling beam compaction (small beams).

4.1 BEAM COMPACTION

Based on the research comparing different compaction methods, the Asphalt Institute 1982 concluded that the rolling wheel and the kneading and/or gyratory methods of compaction produce specimens more like the in-situ materials compared to the impact or static load compaction. Of these, the rolling wheel compaction most closely simulated field conditions as exhibited in performance testing. Therefore, a small scale rolling wheel was used to compact the asphalt concrete samples in this study. Mixes were heated to 135° C and aged for less than two hours, then compacted using a Rolling Wheel Compactor (RWC).

The RWC consists of a solid steel wheel to apply a vertical pressure and movable table as shown in Figure 1. The movable table moves back and forth with adjustable speed. A steel mold with inside dimensions of 375 mm × 125 mm × 75 mm was used in this study. Both the asphalt mixture and the mold are heated to the compaction temperature (135° C), then the hot mix is placed in the mold, the sample is rodded, and the cover of the mold is put in place. The mold is fixed to the movable table in the RWC, and the steel wheel starts to compact through the vertical load while the mold is moved backward and forward beneath the steel wheel. The vertical load is increased gradually during the compaction. During compaction, the cover moves down slowly, compacting the mix until the cover sits on the top of the mold. Voids are controlled through using the proper weight of mixture inside the constant-volume mold. Different weights are used for different mixtures to get a specific level of air voids. After compaction, volumetrics are checked for each compacted brick to check air voids, and corrections are made if necessary to achieve the desired voids in the next brick. The asphalt concrete bricks are cut to obtain two beams from each brick using a diamond masonry saw. According to the AASHTO standards, the dimensions of the beam fatigue specimen are 380 $\pm$6 mm in length, 63 $\pm$6 mm in width, and 50 $\pm$6 mm in height. At least 6 mm have to be cut from both sides of the specimen to provide smooth surfaces for mounting the measurement gages.

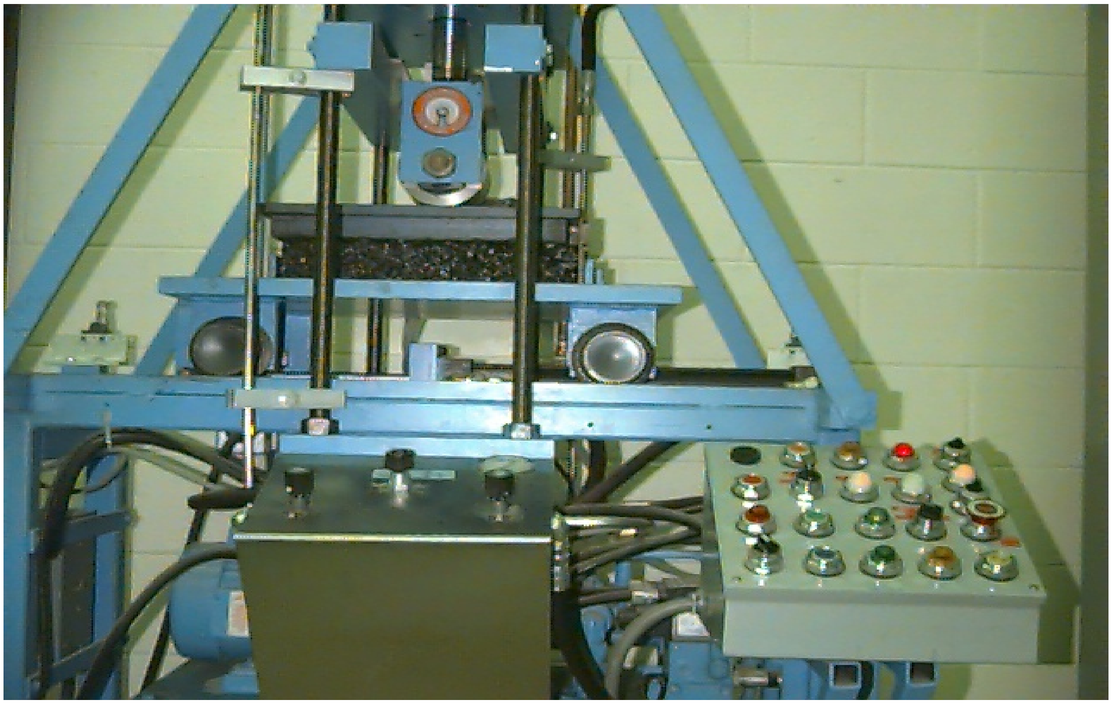


Figure 1. The rolling wheel compactor**.**

4.2 BEAM FATIGUE DEVICE

A digitally controlled, servo-pneumatic closed loop beam fatigue apparatus was used to test the asphalt concrete beams. The equipment consists of three main components: the testing frame, the environmental chamber, and the control data and acquisition system (CDAS). Figure 2 shows a picture of the Beam Fatigue Apparatus. This same testing setup can perform both the controlled strain or controlled stress mode of loading. In addition, different wave shapes can be used. Setting the loading mode and the wave shape is done through the software provided with the system. This software is user friendly with required inputs to perform the test being input under control by the software interface.

4.3 THE FATIGUE FRAME

The testing frame is a self-contained, digital closed loop servo-pneumatic controlled third point loading frame that satisfies the AASHTO TP8-94 for sample positioning. The loading system operates under position feedback control. This control system automatically adjusts the output waveform to match the input waveform, producing very precise control. A load cell is used to measure the force applied to the specimen. The maximum force the machine can apply is 5 KN ($\pm$2.5 KN). A 1 mm ($\pm$ 0.5 mm) stroke Linear Variable Displacement Transformer (LVDT) is used to measure the deflection of the specimen. The LVDT measures the deflection at the center of the asphalt specimen. The machine can run up to 100 million load cycles.

4.4 THE ENVIRONMENTAL CHAMBER

The environmental chamber contains both the testing frame and specimens. The chamber can maintain temperatures between 2° C and 60° C. All tests were conducted as specified in SHRP standards at 20 $\pm$ 0.5 °C (AASHTO TP8-94). Temperature transducers measure the temperature both on the skin and in a hole drilled into the core of a sample beam in the cabinet.

4.5 THE CONTROL DATA AND ACQUISITION SYSTEM (CDAS)

The CDAS is a compact, self-contained unit that provides all critical control, timing, and data acquisition functions for the testing frame and the transducers. The CDAS automatically controls the operation of the beam cradle during the test; also, it directly controls the servo valve to apply the requested loading rate. This control system automatically adjusts the output waveform to match the input waveform, producing very precise control. The standard acquisition module has eight normalized (+/- 10v range) transducer input channels

Figure 2. Beam fatigue equipment.

that are digitized by accurate, high-speed 12-bit Analog to Digital (A/D) converters for data analysis and presentation. The normalized input means that any transducer with +/- 10v output range can be plugged into any channel, which enhances the flexibility of the data acquisition module. The control module has three normalized input channels for feedback control. One is dedicated to the actuator position, the second to actuator force, and the third is a general purpose input for on-specimen transducers. The CDAS in concert with the personal computer controls the load deformation during testing and collects the data at the same time.

4.6 TESTING CONDITIONS

The HMA specimens were stored in the chamber for at least two hours to reach the required test temperature as verified by the embedded thermocouples. The controlled strain mode of loading was used on all samples in this study. The following parameters were used in the IPC fatigue equipment:

Mode of loading: constant-strain

Wave shape: haversine
Load pulse width: 100 ms (10 Hz)
No rest period
Temperature: 20° C

The strain levels used in controlled strain testing are varied between 250 and 1000 micro-strain as a general range. It is worth noting that in the haversine loading, the sample is loaded then unloaded. To return the sample back to its original position, a small force is applied by the computer to return the specimen to the undeformed position. At least four specimens were tested to establish a representative fatigue curve. Testing was conducted at varying strain levels to generate a fatigue curve for the material.

CHAPTER 5 TEST RESULTS

The following properties for each sample can be determined during the test: maximum tensile stress, maximum tensile strain, flexural stiffness, modulus of elasticity, phase angle, dissipated energy, and the cumulative dissipated energy. The dissipated energy terms are used in research studies and are not included in this analysis. The following formulas are used to calculate the different properties during the test:

Maximum Tensile Stress (kPa):

$$\sigma_t = \frac{3000aP}{wh^2} \tag{24}$$

Where:

a = distance between reaction and load clamps (118.5 mm),
P = peak force (N),
w = beam width (mm), and
h = beam height (mm).

Maximum Tensile Strain (mm/mm):

$$\varepsilon_t = \frac{12\delta h}{23a^2} \tag{25}$$

Where:

δ = peak deflection at center of beam (mm).

Flexural Stiffness (MPa):

$$S = \sigma_t / 1000\ \varepsilon_t \tag{26}$$

Modulus of Elasticity (Mpa):

$$E = \frac{Pa}{\delta Wh}\left[\frac{23a^2}{4h^2} + k(1+\nu)\right] \tag{27}$$

Where:

k = actual shear stress divided by average shear stress (assumed 1.5),
ν = Poisson's ratio.

Phase Angle (degrees):

$$\phi = 360\ fs \tag{28}$$

Where:

s = time lag between P and δ, in seconds, and
f = load frequency (Hz).

The main information presented here from this testing focuses on the fatigue curve which is composed of the K1 and K2 coefficients from the strain model, and not material properties. The strain and the number of load repetitions to reduce the modulus by 50 percent are plotted on a Log-Log plot and the best fit straight line provides the coefficients. The 50 percent modulus reduction is the accepted deterioration level for fatigue life comparisons and model development. Figure 3 is a typical fatigue curve for an IDOT mixture. The data scatter represented and the goodness of fit of the equation are typical of all tests with this equipment. For this mixture, the K1 is 4.393×10^{-11} and K2 is 4.46 when the x term is $(1/\varepsilon)$.

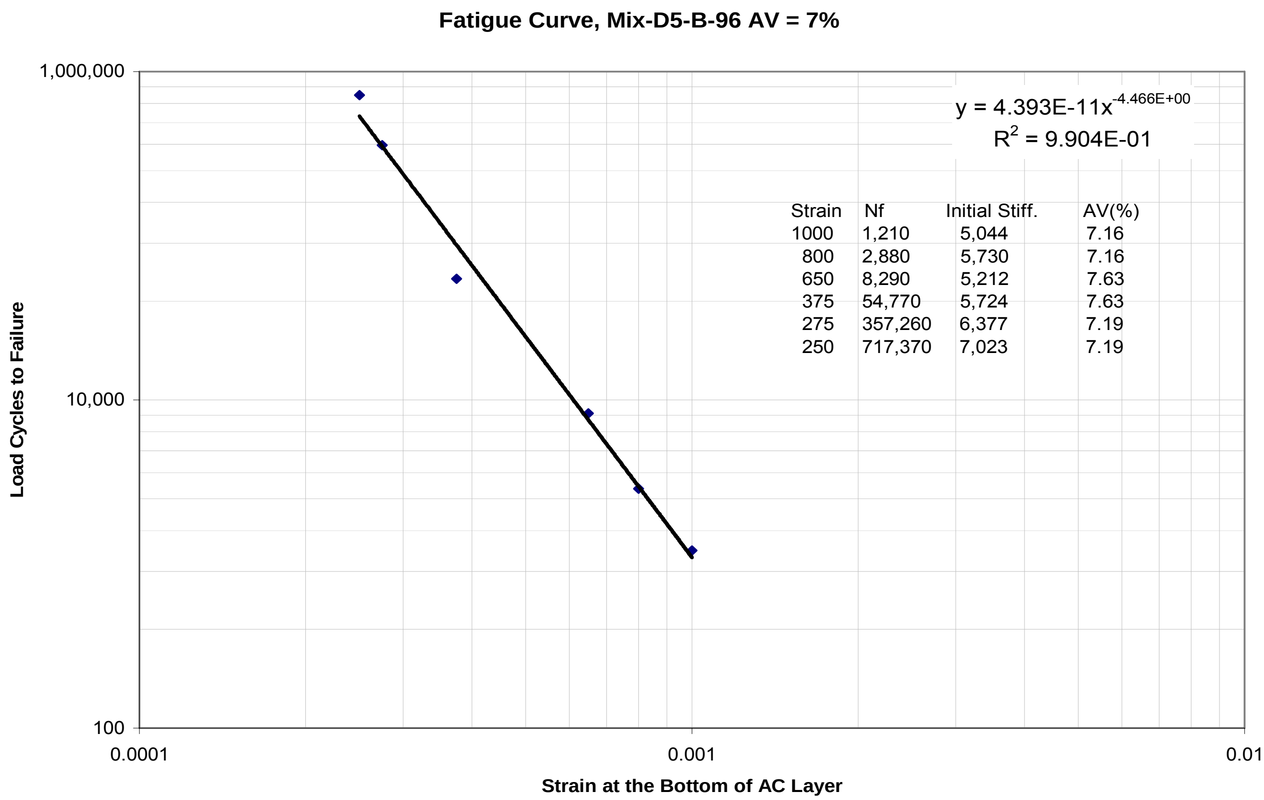


Figure 3 . Typical fatigue curve.

5.1 FATIGUE ANALYSIS OF IDOT MIXTURES – SET ONE

A total of 25 mixtures as described previously from different districts of Illinois were included in this set of data. Controlled strain mode of loading was used in these tests and the strain model was used to describe all mixtures. Table 4 contains the traditional fatigue relation coefficients K1 and K2. Two levels of air voids were used for these IDOT mixtures 4.0 percent and 7.0 percent. The mixtures were either surface or binder mixtures. As shown in Table 4, the IDOT mixtures exhibit good fatigue behavior as shown by the high exponent in the strain model.

Table 4. Fatigue Coefficients for Set One IDOT Mixtures

Mix #	Mix Type	AV (%)	K1	K2	R ^ 2
1	Binder	4	2.427E-07	-3.5839	0.959
		7	2.371E-11	-4.7273	0.987
2	Surface	4	1.464E-13	-5.6395	0.975
		7	1.094E-14	-5.9888	0.997
3	Surface	4	4.014E-10	-4.2839	0.996
		7	2.018E-11	-4.6728	0.999
4	Binder	4	7.239E-12	-5.0042	0.991
		7	8.669E-08	-3.6952	0.859
5	Binder	4	8.928E-11	-4.4113	0.979
		7	9.387E-10	-4.0835	0.949
6	Surface 2D	4	2.112E-12	-4.9921	0.985
		7	1.257E-11	-4.754	0.996
7	Binder	4	4.607E-10	-4.2195	0.891
		7	5.684E-09	-3.9514	0.965
8	Surface	4	3.196E-12	-5.0680	0.983
		7	1.630E-10	-4.5282	0.979
9	Binder	4	2.126E-11	-4.6428	0.992
		7	3.645E-11	-4.6031	0.997
10	Binder	4	1.840E-11	-4.5700	0.980
		7	2.549E-9	-3.873	0.872
11	Surface	4	1.012E-10	-4.4141	0.977
		7	3.299E-12	-4.8895	0.997
12	Surface	4	3.982E-15	-6.1320	0.954
		7	5.163E-10	-4.4843	0.95
13	Binder	4	2.291E-14	-5.3755	0.996
		7	1.809E-9	-3.8883	0.892
14	Binder	4	9.595E-10	-4.3944	0.996
		7	1.469E-09	-4.0288	0.955
15	Surface	4	9.198E-11	-4.5461	0.994
		7	1.320E-09	-4.1768	0.999
16	Surface	4	5.054E-11	-4.6220	0.994
		7	2.559E-13	-5.3127	0.9999
17	L-Binder 2D	4	2.144E-11	-4.7893	0.997
		7	4.292E-10	-4.3825	0.968
18	L- Binder 1D	4	2.241E-09	-4.1074	0.997
		7	5.357E-13	-5.2084	0.996
19	Surface 1D	4	1.856E-09	-4.0725	0.997
		7	1.640E-10	-4.3785	0.990
20	Surface 2D	4	3.651E-11	-4.5619	0.998
		7	1.385E-09	-4.0238	0.978
21		4	1.337E-10	-4.2785	0.9815
		7	1.022E-10	-4.3027	0.9778
D5-B-96	Binder	4	6.472E-12	-4.6781	0.984
		7	4.393E-11	-4.4660	0.99
D6-B-96	Binder	4	3.365E-10	4.1350	0.994
D6-S-96	Surface	4	2.018E-09	-4.0121	0.993
		7	4.165E-12	-4.8541	0.997
D7-B-96	Binder	4	4.753E-13	-5.076	0.976
		7	1.756E-11	-4.471	0.975

Comparing the fatigue curves of the IDOT mixtures based upon examination of the traditional strain model curves is difficult. Mixtures all have very individual curves, and the air void levels produce individual curves. Analysis of the K1 and K2 values provides a means of comparing the differences between mixtures with a potential for examining the effects of different mixture variables on the K1 coefficient.

5.1.1 K1 – K2 Relationships

The K1 and K2 values for this set of mixtures are representative of typical values for HMA. This is shown in Figure 4 which plots Log K1 versus K2. The relationship shown here follows that found by Myre 1992, and other studies that are shown in Figure 5. The data in Figure 5 illustrate the uniformity of this relationship between K1 and K2 that should be expected of all mixtures. This figure will be discussed in more detail in a later section.

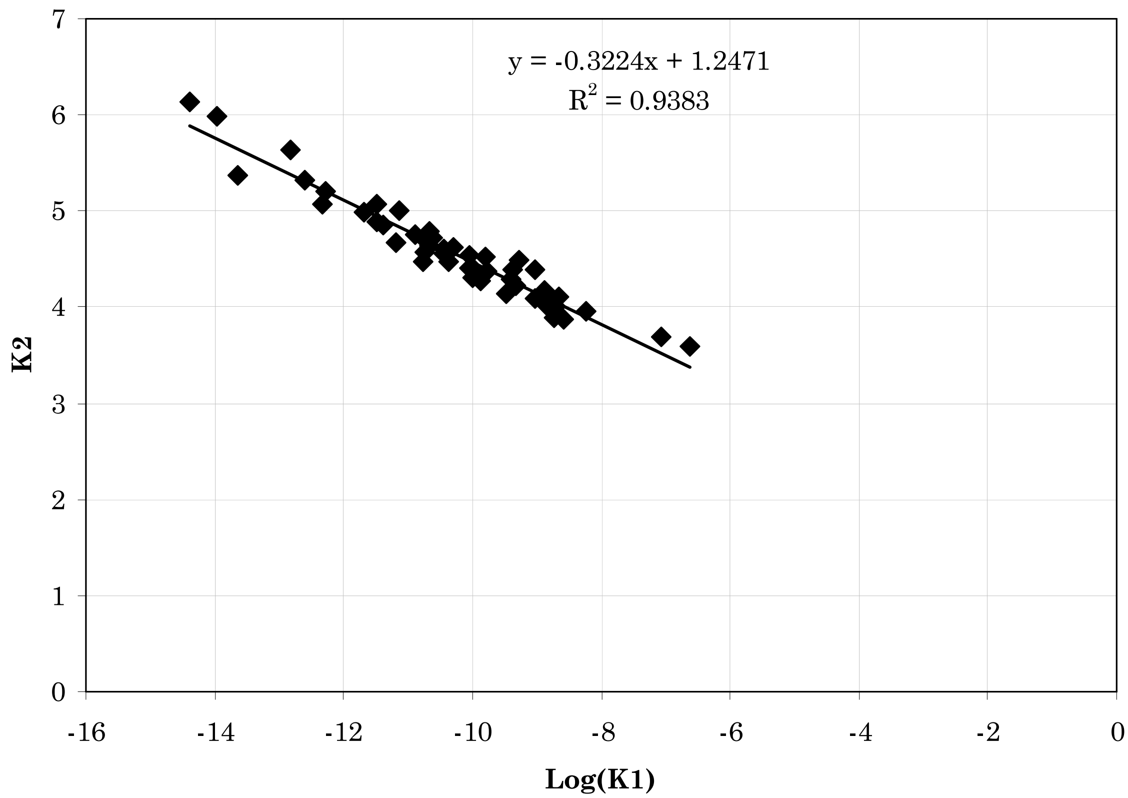


Figure 4. Log(K1) versus K2 relation in the traditional fatigue equation for IDOT Set One mixtures.

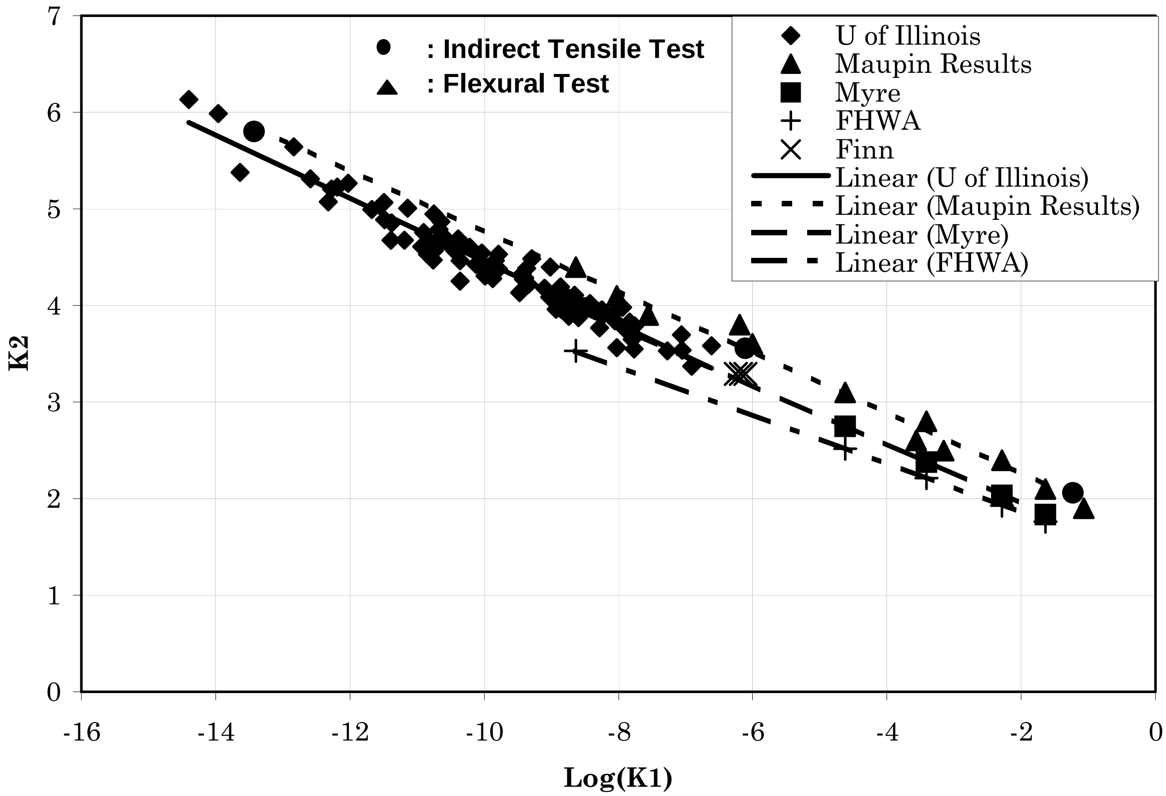


Figure 5. Log (K1) versus K2 relation from different studies.

The data in Figure 5 contain all the Set One IDOT mixtures tested in this study, at all air void levels. This analysis would indicate that the K2 values could be expected to be above 3.5, with an average value of 4.5. Because these data are composed of different mixtures, surface and binders, two different air voids, and different mix designs, it would be beneficial to determine if there are any differences in these values that can be attributed to mixture differences.

Figure 6 shows the K1-K2 relation for the binder and surface mixtures at 7 percent air voids. Figure 7 shows this relationship for 4 percent air voids. While there are trends evident in the data which might support some discussion of engineering property effects, the variability in the data makes it impossible to show a statistical difference between the two mixtures and two density levels. This variability will be investigated further under this project in a report that examines the impact of mixture variables on K1 and K2. It is hoped that since previous models have utilized mixture variables to describe K1, a relationship will be developed from these data that would provide an indication of how K1 could be varied between different mixtures and binders, and what impact this would have on the corresponding K2 value.

It is worth noting here that all of the Set One mixtures have a K2 value above 3.0, with only one below 3.5, 90 percent of all tests have a value above 4.0, and half of all tests have a value above 4.5.

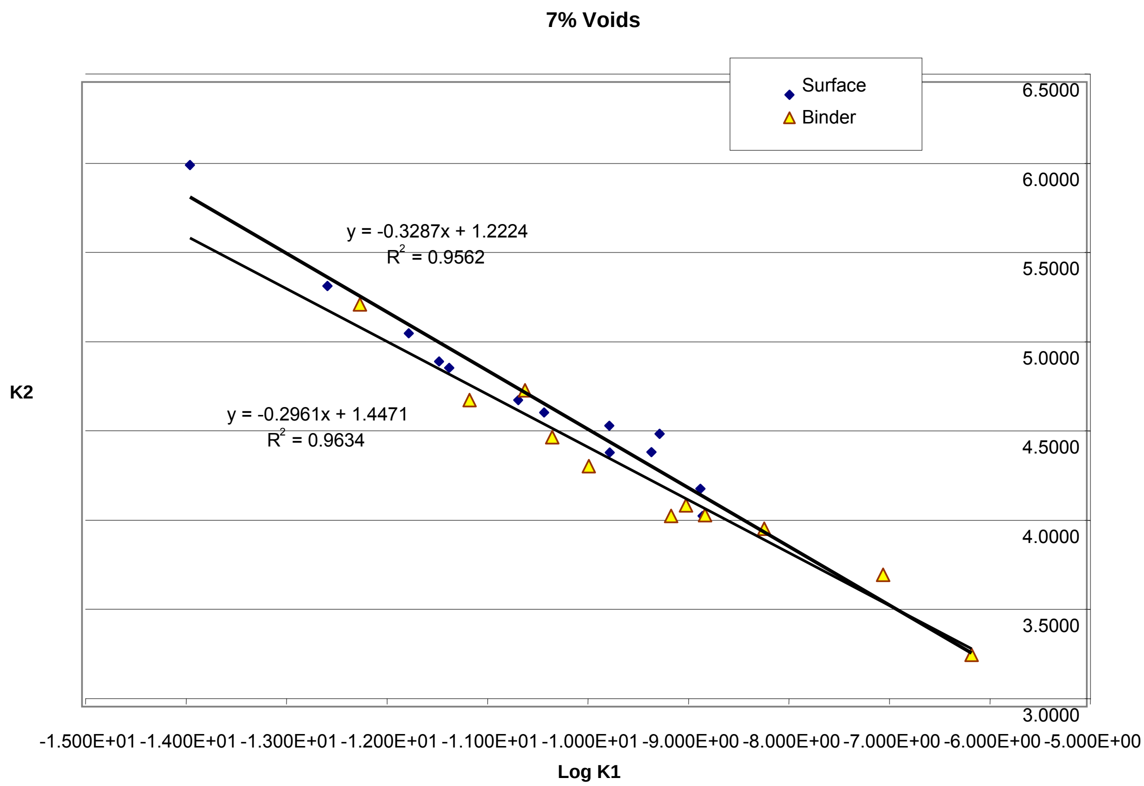


Figure 6. K1 - K2 Relationship for 7 percent air voids.

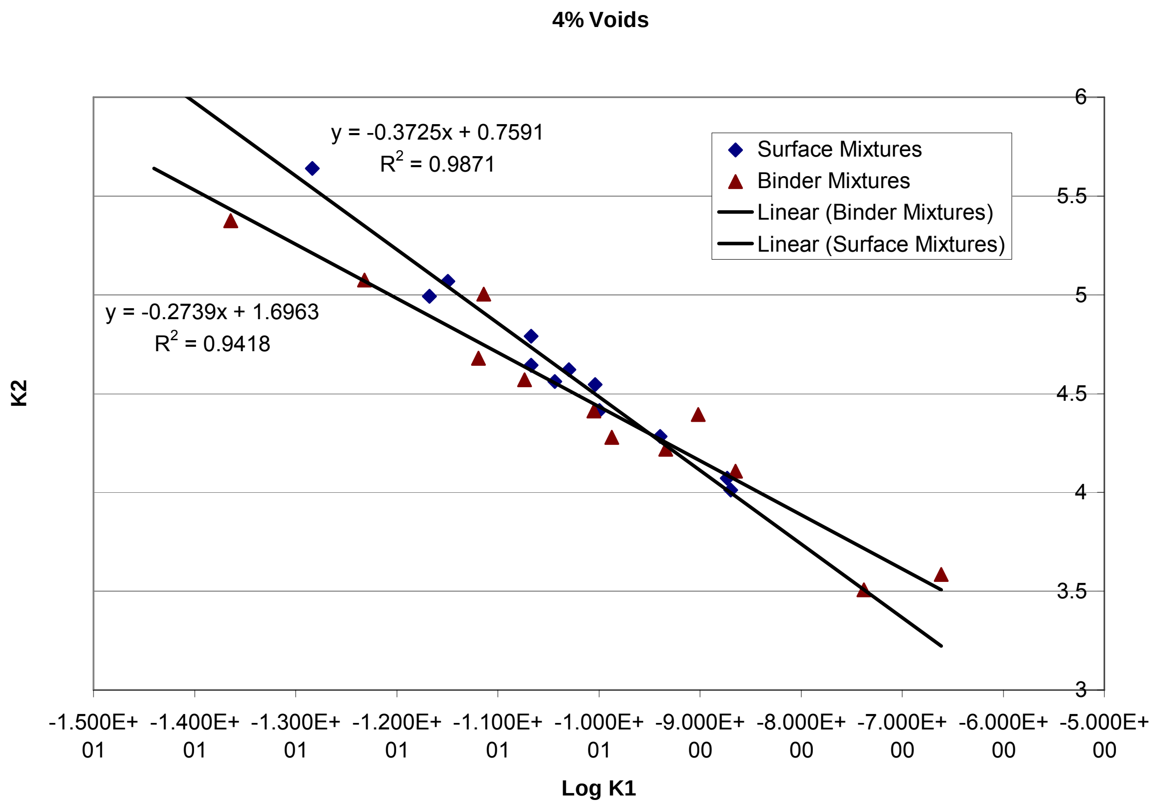


Figure 7. K1 – K2 Relationship for 4 percent air voids.

5.1.2 Fatigue Endurance Limit

The fatigue endurance limit (FEL) is determined by testing at strain levels lower than those used in the standard testing, typically to 100 and 70 micro-strain. The presence of an endurance limit can be shown when the fatigue curve deviates from its slope, K2, and flattens out, as shown in Figure 8. This clearly indicates that a small strain value will produce an extraordinarily long fatigue life; in essence the resistance to fatigue damage accumulation is seemingly infinite.

The fatigue endurance limit testing on samples from IDOT Set One is included in Figure 9. These data clearly indicate that for all mixtures tested there is a limit below which the fatigue curve flattens. This establishes the existence of an FEL. This testing does not establish a specific strain level, but the range appears to be in the 70 to 100 micro-strain range, with no samples requiring strains below 70 micro-strain to exhibit the endurance limit behavior. This limit value appears to be related more to the binder used, with polymer-modified binders exhibiting the larger limiting strains for the limit. This clearly establishes the existence of an FEL but does not provide mixture composition insights.

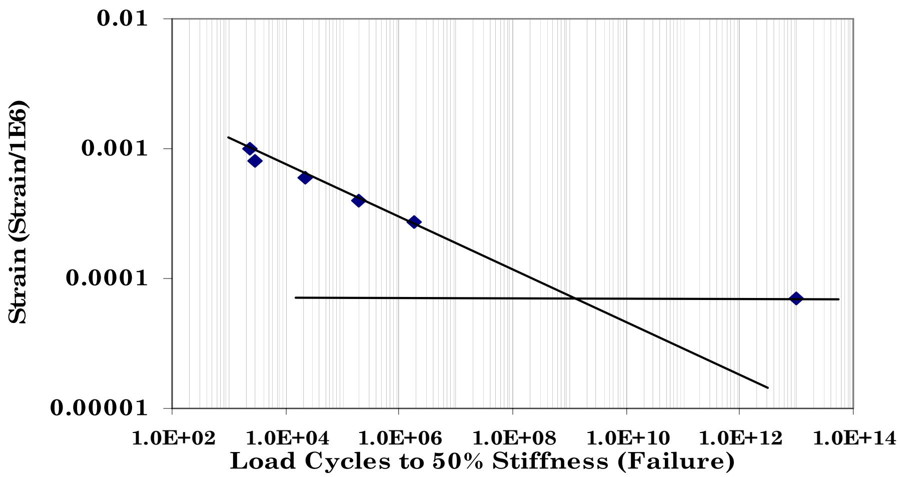


Figure 8. Fatigue curve with fatigue endurance limit.

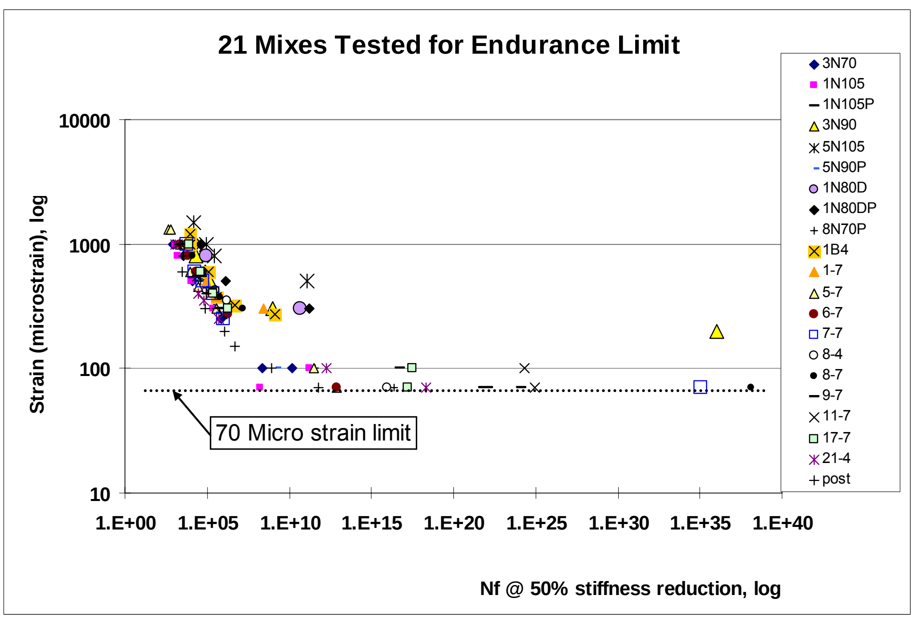


Figure 9. Fatigue endurance tests for IDOT Set One mixtures.

5.2 ELHMAP FATIGUE RESULTS - SET TWO

Table 5 contains the fatigue test results for the ELHMAP mixtures, both the standard and the RBB mixtures. These results are more applicable to current IDOT conditions with gyratory mix designs and PG graded binders. The K1 - K2 relationship is plotted in Figure 10. These data include the standard mixtures with polymer and neat binders, different gyration compaction levels, and the RBB mixtures. There is no discernible difference in this relationship from what was seen previously for the Set One mixtures. Although two curves can be drawn, there is no statistical difference between the two sets of fatigue tests. Indeed, when these data are compared to all data in Figure 5, there is no difference from all historical data, the older Set One IDOT mixtures, and the Myre data. It is clear that the fatigue data for these mixtures are consistent and representative.

Table 5. Fatigue Coefficients for Set Two IDOT Mixtures

Sample	K1	K2	R^2
1N105	2.59E-11	4.4869	0.976
1N105P	5.04E-12	4.862	0.995
3N70	6.10E-12	4.7183	0.983
3N70P	2.54E-10	4.2308	0.948
6N50	7.01E-12	4.7547	0.969
6N50P	1.64E-10	4.3257	0.988
8N70	7.40E-08	3.5735	0.985
8N70P	1.08E-10	4.3928	0.983
5N90	1.93E-10	4.1809	0.966
5N90P	1.19E-10	4.3739	0.988
3N90T	1.81E-12	4.9031	0.982
3N90TP	2.20E-10	4.2341	0.982
3N90	6.74E-10	4.0423	0.993
3N90P	1.81E-08	3.9631	0.957
1N80D	5.83E-13	5.50664	0.974
1N80DP	6.25E-13	5.5139	0.849
2N90	2.93E-10	4.2583	0.972
2N90P	1.68E-11	4.8279	0.986
5N105	2.97E-09	4.5025	0.995

“P” indicates additional asphalt – rich bottom binder mix

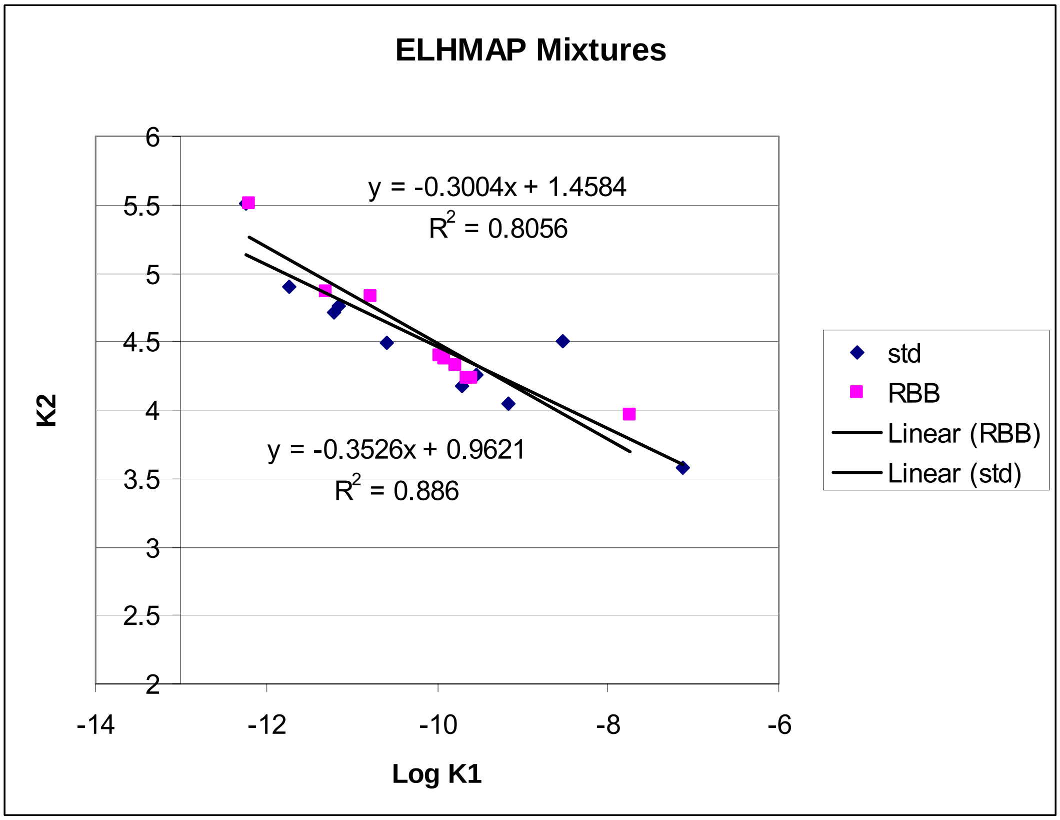


Figure 10. K1 - K2 Relationship for ELHMAP Set two mixtures.

5.2.1 K1 – K2 Relationships

The K1 - K2 relationship for these two sets of mixtures is directly relevant to current mixtures and potentially applicable for pavement thickness design. Examining the K2 exponent, there is a clear trend. It must be noted, as discussed with the Set One data: all of the mixtures in Set Two have a K2 value above 3.5; 90 percent of all tests have a value above 4.0; and half of all tests have a value above 4.5.

The RBB and standard mixture data examined in this report have significantly different compositions given the extra asphalt content and increased density. The analysis of Set One data would suggest that no difference will be seen due to either of these variables; however, this comparison could be misleading since the Set One data did not include similar mixtures where only the asphalt content and density varied. The first difference is shown in Figure 11, which illustrates the flexural modulus values, in mPa. There is clearly an increase in the modulus produced by the additional asphalt cement and the lower air voids in these mixtures.

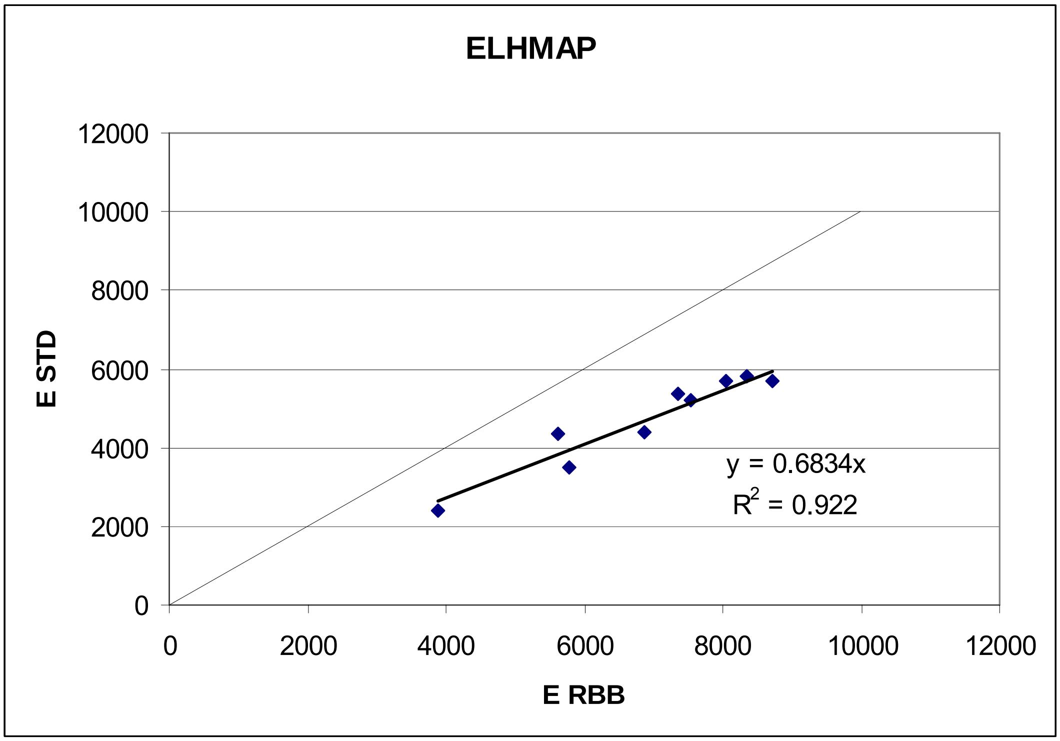
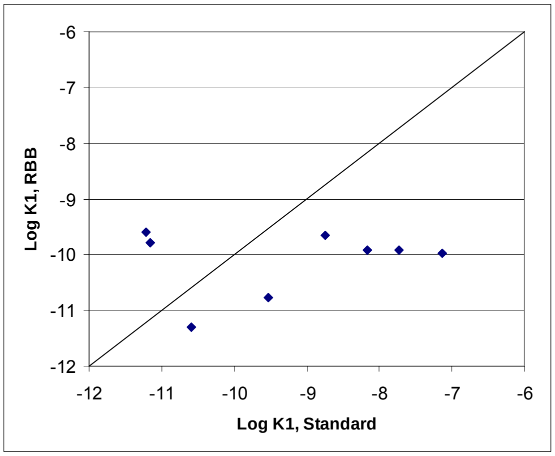


Figure 11. Flexural modulus comparison of RBB mixtures and standard mixtures.

A second consideration is whether there is a consistent change in either the K1 or K2 values produced by the formulation of the RBB. It has been postulated that the extra asphalt produces a more fatigue-resistant mixture. Figure 12 compares the K1 and K2 values for the standard and RBB mixtures. There is an indication that the K2 value increases slightly, and the K1 values change correspondingly. These data would indicate that the extra asphalt content and decreased air voids will have an effect on fatigue resistance as shown by the traditional fatigue algorithm. A more detailed statistical analysis of these data sets is required to determine the property interactions present that affect any relation between K1 and mixture variables that may not be discernible from the visible investigation presented here. If fatigue equations are to be selected based on mixture composition, the detailed analysis must be performed to determine if a relationship exists. The significant finding of the testing, and the analysis presented here, is that the utilization of a model with appropriate K1 and K2 values selected for individual mixtures can be supported by the data, and the production of an RBB does not invalidate expected performance trends, and may improve the performance slightly.



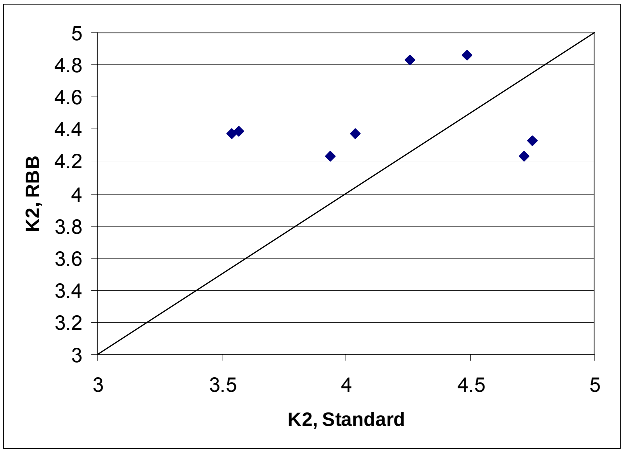


Figure 12. Comparison of K1 and K2 values before and after manufacturing the RBB from the standard mixture.

5.2.2 Fatigue Endurance Limit

Figure 13 presents the endurance limit testing for the standard ELHMAP mixtures, repeated from Figure 9. There is a clear indication of the endurance limit for these mixtures, with that limit being between 70 and 300 micro-strain. The mixtures with the highest strain values are the polymer-modified mixtures (shown in bold in the legend), lending support to the impact of the binder type on the endurance limit.

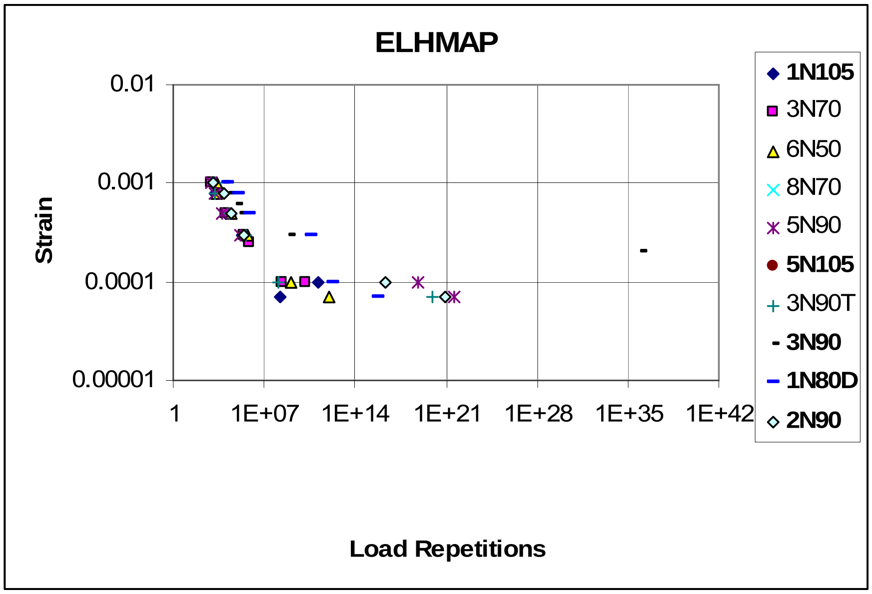


Figure 13. Endurance limit fatigue results for ELHMAP standard mixtures (with polymer-modified mixtures in bold).

Figure 14 presents the endurance testing data for the RBB mixtures. The examination of these traditional fatigue curves indicates that the production of an RBB appears to produce the same performance relative to the FEL. Most certainly the RBB does not lose the ability to exhibit a fatigue endurance limit.

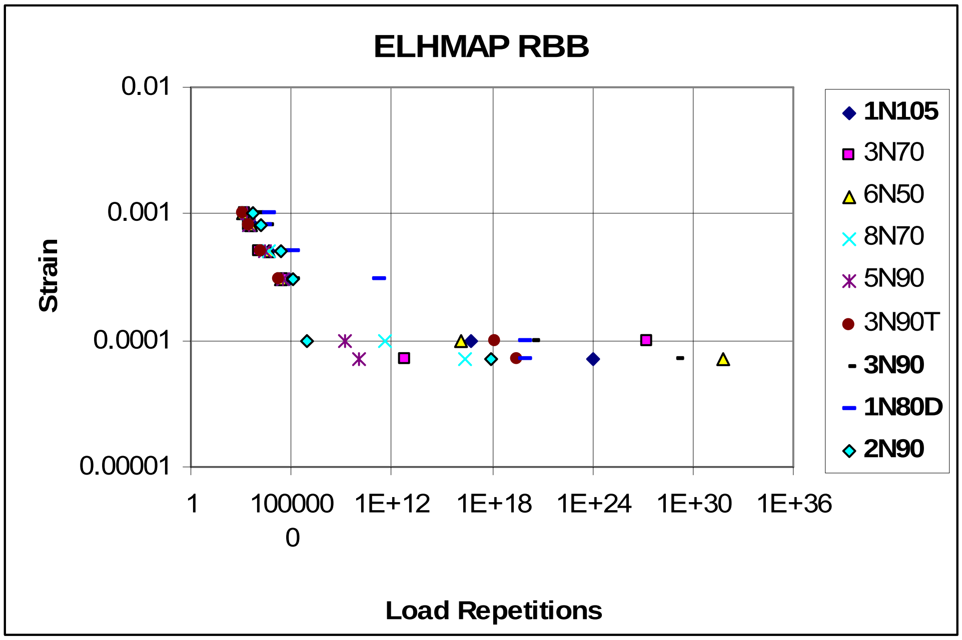


Figure 14. Endurance limit fatigue results for ELHMAP RBB mixtures (with polymer-modified mixtures in bold).

5.3 FEL COMPARISON

Extracting the FEL value for each mixture following the dissipated energy approach of Shen and Carpenter 2005 provides a unique comparison of the FEL values. Figure 15 shows the variation of FEL with the flexural modulus for the two different mixtures. These data indicate that the changes in modulus do alter the FEL. Because these mixtures are all IDOT mixtures, they are similar in features such as mix design and gradation and most of the modulus differences may be attributed to differences in binder type. The heavily modified polymer modified mixtures have a lower modulus at the 20 °C temperature used in the fatigue test.

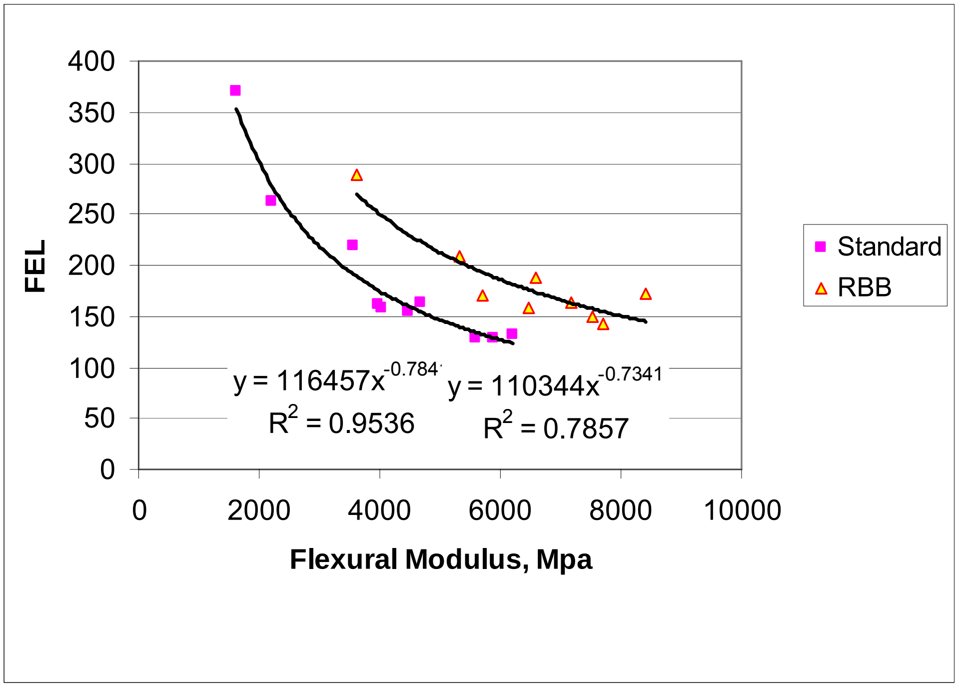


Figure 15. Variation in FEL with flexural modulus.

This relation with modulus would first lead to the expected result that if the RBB mixtures had higher modulus values due to the lower air voids, that the FEL should be lower than the corresponding standard mixture. In fact, this is not true. Figure 16 directly compares the FEL values of RBB mixtures and the standard mixtures. It is apparent that there has been no change in the FEL produced by the RBB procedure with increased asphalt content and increased modulus.

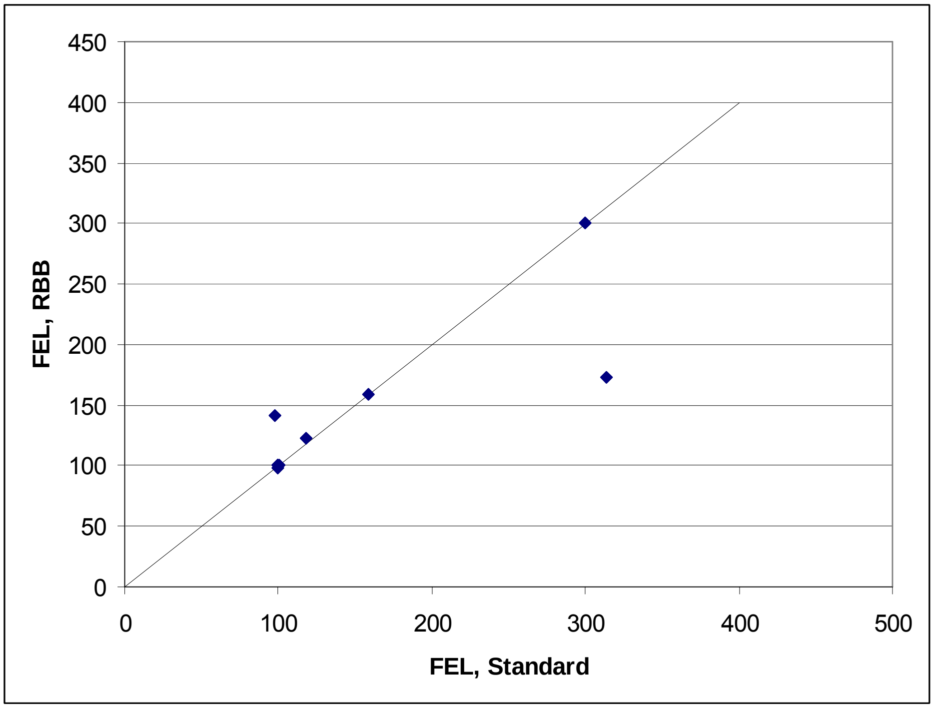


Figure 16. Comparison of the FEL for the standard and RBB mixtures.

CHAPTER 6 SUMMARY

Figure 17, a repetition of Figure 5, shows the relation between K1 and K2 for all the IDOT mixtures and other studies by Finn et al. 1977, Maupin and Freeman, 1976, and Myre 1992. As shown in this figure, the data from this study and the Myre 1992 study produce one line. The data from the Maupin and Freeman study (static compaction of Marshall mixtures) and FHWA are located above and below the line obtained from this study, respectively. The Maupin and Freeman, 1976 data closely reproduce the trend established in this study. In the Finn et al. 1977 NCHRP 1-10B model that is the basis for the NCHRP 1-37A design model of Basyouny and Witczak 2005, different values of the complex modulus (E*) are inserted into the fatigue relation. At different levels of modulus, the exponent (K2) has a fixed value of 3.29 while K1 changes over a narrow range. The resultant plot of the Finn et al. relation results in a horizontal line as shown by the Xs all grouped tightly on the curve. This same behavior can be assumed for the NCHRP 1-37A fatigue model as it is also based on a constant exponent. The use of a constant exponent, while allowing the coefficient to vary, is a major shortcoming in a fatigue model as the data presented here clearly illustrate that K1 and K2 vary in a consistent interrelated manner, and a constant K2 is erroneous when K1 is allowed to vary.

Until further analyses are conducted to establish some substantial relationship between mixture variables and the K1 or K2 coefficients, it is most appropriate to select a K2 value that is representative of current mixtures being produced, and then determine the corresponding K1 value that matches the K2 value. While not precisely representing actual mixture variability, it is a correct procedure that will produce a conservative fatigue algorithm for design.

This finding points out an interesting nature of the FEL. It is principally a result of the binder that is related to the healing potential of the binder. The healing effect of a binder and its relation with the FEL was shown by Shen and Carpenter, 2005. This was for one mixture, two binders, but it clearly establishes the fact that the FEL is the point where the damage of the load is recovered by the healing of the asphalt binder. The results here clearly show that different binders produce different FEL values, but mixtures with the same binder in different density mixtures with increased asphalt content do not show a different FEL. The interpretation of the testing on these mixtures is that asphalt binder type and the healing characteristics of the binder will have the biggest effect on producing different FEL values for different mixtures.

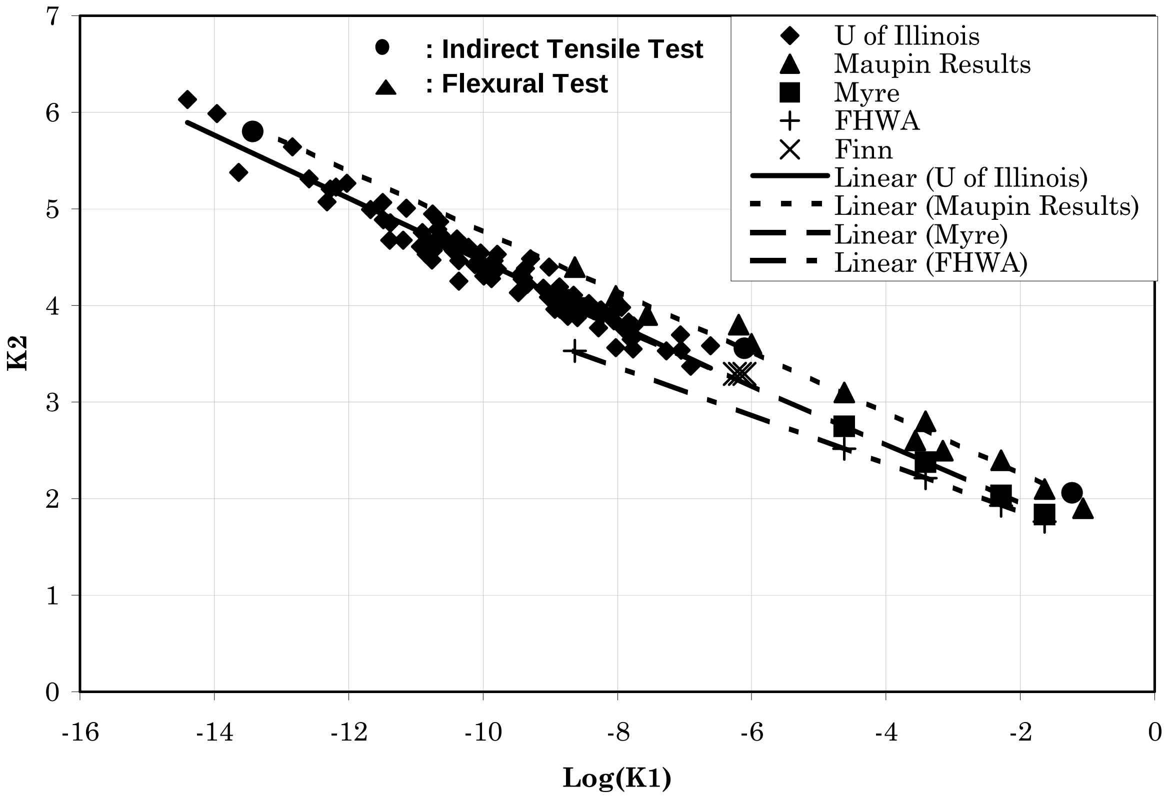


Figure 17. Log (K1) versus K2 relation from different studies.

Utilizing the correspondence between K1 and K2, the following fatigue coefficients would be recommended for IDOT binder mixtures based on testing conducted previously on IDOT mixtures, and for the mixtures representing today's construction.

Total Conservative

K2 = 3.5, K1 = 10^{-7}

Ninety Percentile

K2 = 4.0, K1 = $10^{-8.5}$

Average

K2 = 4.5, K1 = 10^{-10}

The impact of using a different fatigue curve for different mixtures can be seen by comparing the fatigue life when each model is used in a structurally designed pavement that has been analyzed and shown to have a specific tensile strain, say 250 micro-strain. Because the model has no impact on the resulting strain, which is dependent on thickness and modulus, the only differences are the model coefficients. Using 250 micro-strain, the following fatigue lives are obtained:

Total Conservative – 404,000 load repetitions

Ninety Percentile – 809,000 load repetitions

Average – 1,600,000 load repetitions

If a constant value of K2 is used, say the 3.5 value, and the appropriate K1 values are used for all applications, and 250 micro-strain, the following fatigue lives are obtained:

Total Conservative – K2 = 1×10^{-7}, 404,000 load repetitions

Ninety Percentile – K2 = $1x10^{-8.5}$, 12,800 load repetitions

Average – K2 = K2 = $1x10^{-10}$, 404 load repetitions

These results clearly indicate the problems that can develop if acceptable K1 values are accurately developed from mixture composition considerations but are coupled with a constant K2 value. This situation can potentially yield totally erroneous fatigue designs that would make any field calibration extremely questionable.

6.1 PREDICTING K1 AND K2

More detailed analysis of these IDOT mixture data is under way to determine any correlation between mixture values and the K1 values that have been generated by other agencies. This correlation would provide some guidance in the selection of fatigue coefficients from considerations of the initial mix designs and indicate how conservative a selected set of standard values might be, and what, if any, specific mix variables would produce K1, K2 values that could be borderline acceptable. The initial work by Maupin and Freeman, 1976 related indirect tensile strength of Marshall samples to the fatigue coefficients, and their results were shown to produce coefficients that are very similar to those found in this study for the IDOT mixtures, as shown previously in Figure 17. Figure 18 presents the indirect tensile strength data from the IDOT mix designs for the ELHMAP mixtures presented here.

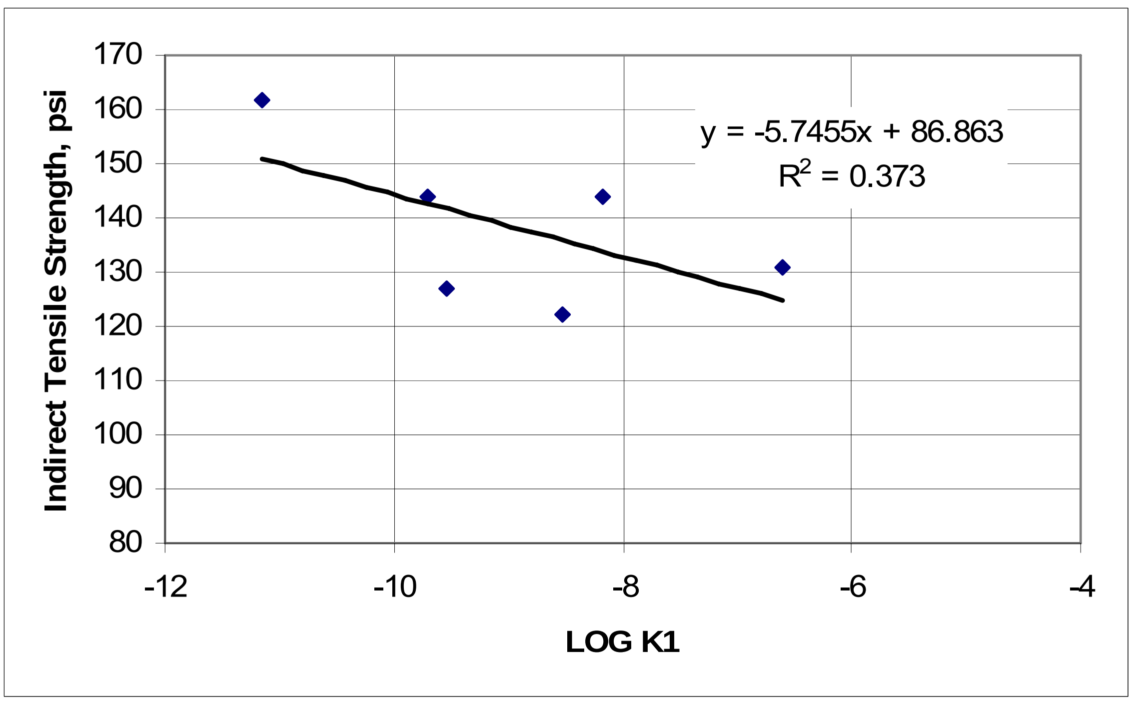


Figure 18. Indirect tensile strength relationship with K1 for ELHMAP mixtures.

The correlation for these data is weak, primarily because the strength data were collected during mix design, while the fatigue samples were taken at the plant, producing some possible inconsistency in the specimens. Different aging conditions may also be found between lab and field aging. However, the relation is consistent with that found in earlier studies in that a higher indirect tensile strength corresponds to a higher K2 value. This relationship could be pursued in the future if a more refined fatigue model is desired.

6.2 FATIGUE ENDURANCE LIMIT

The testing on all types of mixtures, and especially the ELHLMAP set of mixtures (Set Two) clearly supports the existence of a Fatigue Endurance Limit (FEL). Exhaustive testing indicates no strain value below 70 micro-strain needs to be used to have all mixtures exhibit the extraordinarily extended fatigue life associated with an FEL. The strain level associated with the FEL ranges up to around 100 micro-strain, but never below 70 micro-strain.

Specialty mixtures such as the polymer and fiber modified SMA mixture presented here (1N80D) can exhibit an FEL that is considerably above the 100 micro-strain level, but this mixture, while used as a binder mix, would most likely never be used as a lower lift in a flexible or full depth pavement, making a 70 micro-strain level an acceptable default value for an Extended Life Hot Mix Asphalt Pavement Design.

6.3 CLOSURE

Prediction of field fatigue performance from laboratory fatigue resistance testing for an HMA mixture has always been a difficult task. There is fundamental agreement that the phenomenological relationship between strain and load repetitions to failure accounts for the majority of the observed behavior with the addition of modulus adding a slight increase in the correlation. A variety of models developed with these relationships have been used in thickness designs; however, these models regardless of their format have a serious deficiency in that they assume a constant exponent, K2. Previous studies have shown a variable K2, but those studies chose to develop a final model with a constant K2 and a variable K1.

Problems with field performance have most likely been avoided because of the extremely large multiplier required to translate laboratory fatigue to field fatigue performance, as noted in several of the studies. This makes calibration of such models questionable, at best, and points out the need for a thorough evaluation of the data to confirm the relationship between K1 and K2 shown here to adjust the K2 in any chosen model. It is felt that with appropriately related K1 and K2 variables that the shift for field performance will be reduced, and especially any variability associated with the currently recognized wide spread in the multiplier. The large values (up to 700) are highly dependent on the level of failure set for the pavement. Total failure on the pavement surface will produce an extremely large multiplier on the laboratory coefficients, while 10 percent cracking will produce a very low multiplier. Because the models typically have different failure criteria, different multipliers are found for each model.

The data presented here clearly show the existence of a fatigue endurance limit. The magnitude of this FEL is most dependent on binder type and is not readily connected with mix composition. The magnitude of an FEL for all mixtures is never lower than 70 micro-strain, and for some mixtures it ranges up to 100 micro-strain, with polymer modified mixtures showing FEL values approaching 300 micro-strain. This provides a valid design concept for Extended Life Hot Mix Asphalt Pavements.

The production of an RBB mixture marginally increases the fatigue coefficients, and provides a slightly higher modulus. The benefits of these improvements must be compared to the cost of the mixture, and the structural improvements, which at this point would appear to be marginal.

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