apparent success of this new approach in pavement design.

5. During the analytical study, it was assumed that there is no load-transfer capability across a crack. This is especially true when the ratio of crack width to vertical differential movement of a crack is high. Results of this study imply that a SAMI will perform even better if there is some load-transfer capacity through a crack.

6. A better understanding of SAMI properties is needed through laboratory testing.

7. When a SAMI is used, the thickness of overlay becomes less critical. This may result in very economical approaches to overlay design.

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Characterizing Fatigue Life for Asphalt Concrete Pavements

J. BRENT RAUHUT AND THOMAS W. KENNEDY

The evaluation of fatigue life for asphalt concrete pavements is very difficult because of limited knowledge as to fatigue damage relations for real pavements, reliable testing data for only a limited number of mixtures, and limited information as to how the fatigue life potential of an asphalt concrete pavement varies with temperature and mixture characteristics. This paper seeks to overcome some of these difficulties by (a) proposing a typical fatigue relation for a typical asphalt concrete mixture in place, (b) presenting procedures for modifying fatigue relations with changes in temperature and mixture stiffness, (c) proposing a procedure for taking specific mixture characteristics into account, and (d) offering a simplistic method of transforming fatigue life predictions into predictions of area cracked.

The characterization of fatigue life for asphalt concrete pavements is extremely complex and has been the subject of study by a number of researchers for more than two decades. Various laboratory tests have been employed, but the most common one involves small beams subjected to repetitive loading with either constant load or constant strain. The beams have been simply supported, supported on springs, or supported on a rubber medium to simulate a base and subgrade. One characteristic shared by all beam tests and other types of common laboratory tests is that none come close to simulating actual field conditions and a realistic crack-propagation process.

Shell Laboratories attempted a more accurate simulation with wheel-tracking tests, which produced more realistic crack propagation in small asphalt concrete slabs, but may have overpredicted fatigue life because of the lack of environmental effects. More recently, a number of attempts have been made to use laboratory test curves in conjunction with elastic-layer theory to predict the occurrence of fatigue cracking distress in real pavements. Because the laboratory relations almost always grossly underpredict fatigue cracking, shift factors have been used to translate the predictions to approximate fatigue life actually measured in the field. Such fatigue curves appear to be the best available for use in predicting fatigue cracking.

The form of the fatigue relations in common use is derived from a logarithmic relation between either stress or strain and the number of load cycles to failure. The relations between the logarithm of stress or strain and the logarithm of load cycles are considered to be linear, which results in the following general equation:

$$N_i = K_1 \epsilon_i^{-K_2}$$

where

N_i = number of load cycles to failure for a loading that results in a tensile strain,
ϵ_i = calculated strain under load,
K_2 = inverse of absolute value of slope of logarithmic function, and
$K_1 = N_i \epsilon_i^{K_2}$ for any pair N_i and ϵ_i that satisfies the logarithmic function.

Both K_1 and K_2 depend primarily on material characteristics and temperature.

A very important problem with this form of fatigue life characterization is that this equation is extremely sensitive to small variations in the value of K_2, while test results for specimens that fail in tension are quite scattered. Further, the occurrence of fatigue cracking in the field is itself quite variable, even for apparently identical sections. Considering that laboratory test results are also of limited quality, the predictions for individual sections may be expected to be poor, and statistical means for a number of sections are not likely to be especially accurate either.

Unfortunately, in view of our limited capabilities for predicting fatigue cracking, it is the most significant distress experienced by our highways and the primary generator of maintenance and repair requirements. Therefore, it must be considered in design and analysis, and we must characterize the fatigue life potential of asphalt concrete mixtures as best we can.

It is not generally feasible to conduct fatigue testing programs for specific mixes to support routine analyses or designs because of the sophisticated laboratory equipment required and very high costs for such a program; therefore, some other approach must be developed. One reasonable approach is to select a fatigue relation that is reasonably typical for asphalt concrete pavements in place and to modify that relation on some rational basis to reflect the effects on fatigue life of established characteristics of the specific mixture of interest. The purposes for this paper are then to

1. Propose a fatigue relation for a typical asphalt concrete mixture in place,
2. Present procedures for modifying fatigue relations with changes in temperature or mixture stiffness,
3. Propose a procedure based on relative mixture stiffness for taking specific mixture characteristics into account, and
4. Offer a simplistic method of transforming fatigue life predictions into predictions of area cracked.

TYPICAL FATIGUE RELATIONS FOR ASPHALT CONCRETE PAVEMENTS IN PLACE

Thirteen fatigue relations for mixture temperatures of approximately 70°F (from previous studies) are plotted in Figure 1. Descriptions of the tests (or other bases for development) are given elsewhere (1-13).

Six relations were obtained from standard laboratory beam tests (2,6-10). Two relations were also obtained from laboratory tests, but the specimens rested on elastic supports (5,11). Two other relations were produced from multiple regression analyses on American Association of State Highway Officials (AASHO) Road Test data and are related to present serviceability index (PSI) (3,4). Witczak (3) specifically related tensile strain in the bottom of the AASHO Road Test pavements to number of load repetitions to reduce the PSI to 2.5. ARE (4) related tensile strain to measured cracking, but in terms of 18-kip equivalent single-axle load (ESAL) based on PSI. Three relations were obtained from laboratory beam-test results transformed to represent field conditions (1,12,13). In general, laboratory beam-test results grossly underpredict fatigue cracking distress, while the other relations may underpredict or overpredict.

Figure 2 includes only the three relations transformed to represent field conditions and the relation by Meyer and others (2) that was derived from stress-controlled beam testing related to 20 percent class 2 and 3 cracking at the Brampton Test Road. Note also that a relation developed by Finn and others (1) to represent 45 percent measured cracking at the AASHO Road Test is essentially identical to that of Meyer and others. It can be seen from Figure 2 that these four curves are very similar, and it appears reasonable to select from this grouping a relation to be considered typical. The relation selected was the one developed by Finn and others to represent less than 10 percent cracking at the AASHO Road Test. The relation from Meyer and others, which represents 20 percent class 2 and 3 cracking at the Brampton Test Road or 45 percent cracking at the AASHO Road Test, could also be used to represent failure or need for maintenance if a higher level of damage is preferred.

MODIFYING TYPICAL FATIGUE RELATIONS WITH CHANGES IN TEMPERATURE OR MIXTURE STIFFNESS

Typical fatigue relations were selected in the previous section to represent two different levels

Figure 1. Fatigue relations at 70°F, including results of laboratory testing, wheel-tracking tests, and efforts to represent field conditions.

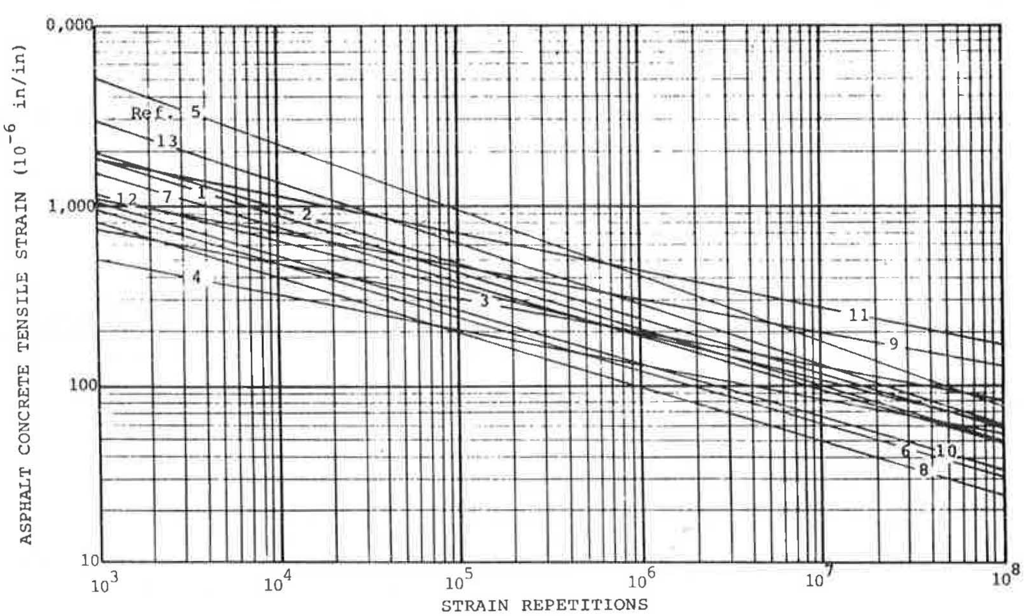


Figure 2. Fatigue relations for asphalt concrete that represent pavements at pavement temperature of 70°F.

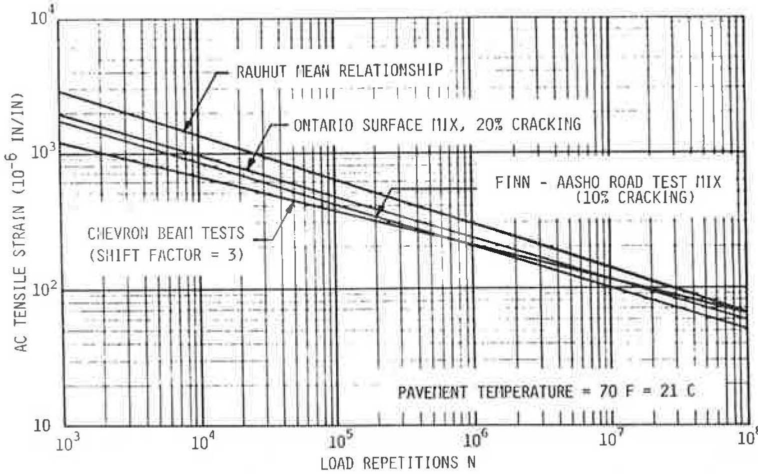
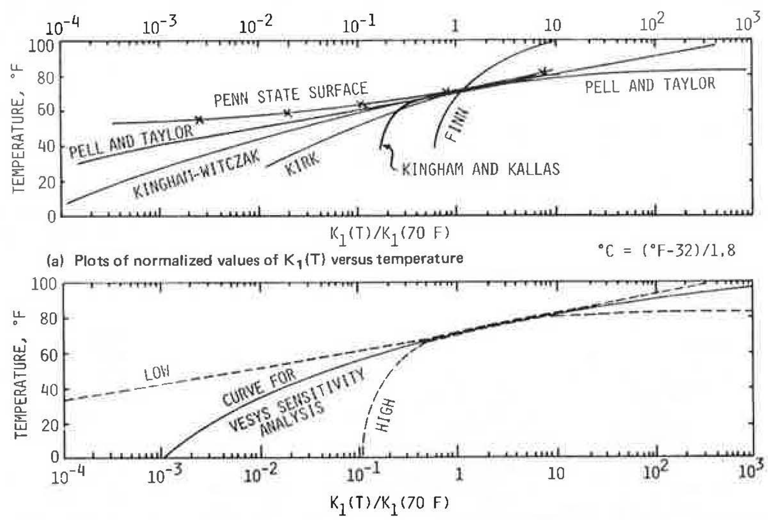


Figure 3. Relation between normalized K_1 and temperature.



(a) Plots of normalized values of $K_1(T)$ versus temperature

(b) Plots of normalized values of $K_1(T)$ used for modifying $K_1(70°F)$ in VESYS II sensitivity analysis

of fatigue cracking distress. These typical fatigue relations will require modifications to (a) represent specific mix characteristics and (b) represent changes in temperature or mixture stiffness. Procedures for the first type of modification will be provided in a subsequent section, but procedures for modifying the typical fatigue relations with temperature or mixture stiffness are the subjects of this section.

Modifications with Temperature

Witczak (3) recommended fatigue relations as functions of temperature. Rauhut and others developed procedures for varying fatigue curves with temperature, which is described and applied elsewhere (12), and these procedures were adopted by Meyer and others (2). The relation by Witczak assumes that the exponent K_2 is constant and that the multiplying coefficient K_1 varies with temperature. Rauhut and others, on the other hand, noticed some slight variations of K_2 with temperature, and this trend was included at a limited rate of change in their procedure.

The study by Rauhut was based on limited data in the literature on variations in fatigue relations with temperature for the same mixes. The data used appear elsewhere (12, Table 17) and are plotted for $K_1(T)$ normalized for $K_1(70°F)$ in Figure 3a. A curve has been added for the Finn relation (1), with $K_1(T)$ obtained by substituting values of resilient modulus for 40°, 70°, and 100°F from test results on the AASHO mix. It can be seen that the variations in $K_1(T)$ with temperature are quite different for the various sets of test results, ranging from approximately one order of magnitude for the Finn relation to many orders of magnitude for the Penn State surface mix.

Figure 3b indicates the range of variations and a curve selected on the basis of judgment to be used in converting $K_1(T)$ as a function of temperature. Meyer and others (2) developed a multiple regression equation for the curve selected as follows:

$$\log [K_1(T)/K_1(70°F)] = -2.952 + 0.000\,58T^2 \quad (1)$$

The relation for $K_2(T)$ (12) is as follows:

$$K_2(T) = K_2(70°F)[1 - 0.001(T - 70)] \quad (2)$$

By substituting 70°F for T in Equation 1, it can be readily seen that the regression curve does not pass through the value 1 when the ratio $K_1(T)/K_1(70°F)$ is 1. A somewhat more general and more accurate relation may be obtained as follows:

$$\log [K_1(T_1)/K_1(70°F)] = -2.952 + 0.000\,58T_1^2 \quad (3)$$

$$[K_1(T_1)/K_1(70°F)] = 10^{(-2.952 + 0.000\,58T_1^2)} \quad (4)$$

Similarly,

$$[K_1(T_2)/K_1(70°F)] = 10^{(-2.952 + 0.000\,58T_2^2)} \quad (5)$$

Dividing Equation 4 by Equation 5 and taking logarithms gives the following:

$$\begin{aligned}\log[K_1(T_1)/K_1(T_2)] &= \log[10^{(-2.952 + 0.000\,58T_1^2)} \div 10^{(-2.952 + 0.000\,58T_2^2)}\\ &= \log 10\,(-2.952 + 0.000\,58T_1^2 + 2.952 - 0.000\,58T_2^2)\\ &= 0.000\,58\,(T_1^2 - T_2^2)] \end{aligned} \quad (6)$$

Figure 4 (3) is a copy of a plot that shows the Kingham-Witczak relations in solid lines for a wide range of temperatures. Superimposed on this plot are relations generated by Equations 2 and 6 for identical temperatures based on the Kingham-Witczak relation at 70°F. As can be seen, the Witczak and Rauhut modifications for temperature are quite similar above around 60°F, but differ rather dramatically for the colder temperatures. This would be expected, considering the differences between the Kingham-Witczak relation (Figure 3a) and the typical relation selected for previous work (Figure 3b).

It may be seen that Equation 6 reaches a limit at a temperature of 0°F, as the values of temperature are squared. The effects of freezing mixture temperatures on fatigue relations are really not known, but such a limit appears reasonable as fatigue life potential is correlated strongly to mixture stiffness, which tends to reach an upper limit with decreasing temperature at around 20°F. Equation 6 is recommended for modifying K_1 with temperature as (a) it represents a relatively typical variation with temperature (a compromise within the broad range of variation shown in Figure 3a), and (b) the data for temperatures below freezing are very limited, but the only known data (13) are more consistent with the results from Equations 2 and 6 than with the Kingham-Witczak relation.

Equation 2 was developed during a period when the approach of considering K_2 constant and varying K_1 to account for temperature effects was generally accepted. It was noted by Rauhut at that time that there were sometimes considerable variations in K_2, and Equation 2 was developed to introduce this trend. However, much of the data then available indicated relatively constant values of K_2, so the trend was introduced at a very limited rate. There is a fairly close correlation between K_1 and K_2 that is documented in Figure 5, which was extracted from various sources (3,11, 14,15). (Note that variations of log K_1 along the abscissa of Figure 5, in general, represent variations in sample temperature.) Values of $K_2(T)$ and $K_1(T)$ obtained by application of Equations 2 and 6 to the typical relation for 10 percent cracking were also plotted. As can be seen, the Kingham-Witczak relation plots as a horizontal line because of the assumed constant value of K_2, and this appears to be inconsistent with the clear correlation established by the preponderance of test data. Also, the Finn relation, modified by Equations 2 and 6, plots almost horizontal, as Equation 2 only included a moderate rate of change with temperature.

In order to obtain the best representation of variation of $K_2(T)$ with $K_1(T)$, it was decided to

Figure 4. Comparison of Witczak fatigue relations for various temperatures and Witczak's 70°F fatigue relation modified with temperature by using Equations 2 and 6.

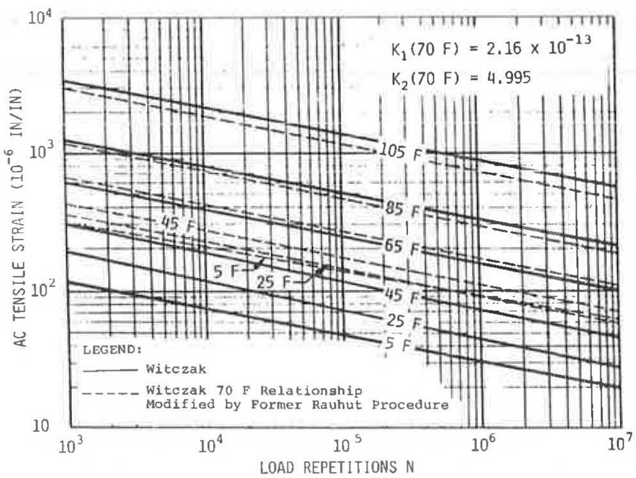
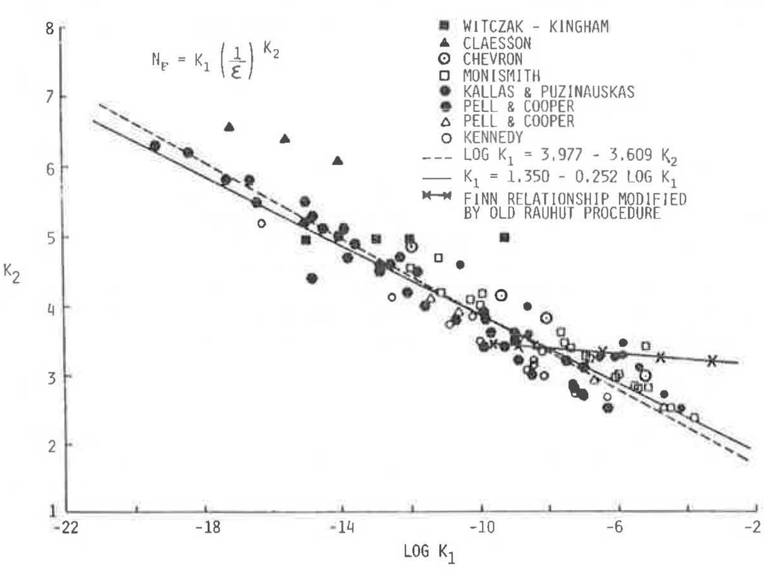


Figure 5. Combined relations between K_2 and K_1 from various studies.



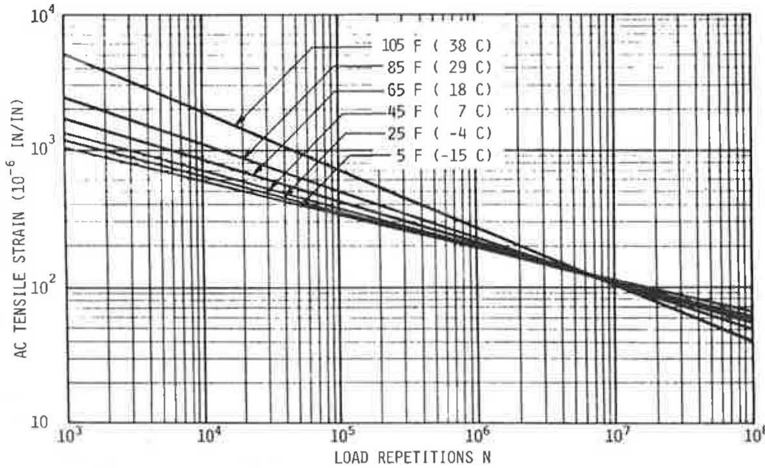


Figure 6. Fatigue relation for 10 percent cracking selected as typical, plotted for different temperatures (using new transformation relation).

use the regression equation from Figure 5 in lieu of Equation 2 for calculating K_2:

$$K_2 = 1.350 - 0.252 \log K_1 \quad (7)$$

Study of Figure 5 indicates that most of the flexure data lies above the regression line and the rotating cantilever and indirect tensile test data below. As the typical values of K_1(70°F) and K_2(70°F) of 7.87×10^{-7} and 3.29, respectively, appear to represent typical flexural test data better than the general regression line in Figure 5, the line was simply transformed to better represent the specific test results (at the same slope) by using Equation 7 to calculate K_2 for the known K_1 and adding the difference ΔK_2 of 0.40 between actual and calculated K_2 to the equation, i.e.,

$$K_2\ (T) = 1.75 - 0.252 \log K_1\ (T) \quad (8)$$

By using values of $K_1(T)$ calculated by Equation 6 for the typical relation and calculating consistent values of $K_2(T)$ with Equation 8, the relations plotted in Figure 6 are obtained. As can be seen by comparing the general distribution of the relations with temperature in Figure 4 to the distribution in Figure 6, changes in predicted fatigue life with changes in temperature will be considerably reduced by use of Equation 8 in lieu of Equation 2 (and indeed there is almost no change for strains approximating 0.0001). The question then arises as to which is most accurate.

The preponderance of test data now available, including results for 11 mixes (8), indicates that K_2 does vary with temperature or stiffness but at different rates for different mixes and perhaps different types of tests and procedures. However, it is not really known how either $K_1(T)$ or $K_2(T)$ vary in a field pavement with temperature, as all field relations have either been extrapolated from laboratory data by shifting the curves to approximate measured cracking levels as functions of calculated strains and repetitions of loads or either directly or implicitly based on ride quality measured at the AASHO Road Test instead of fatigue cracking. Although we may only hope to limit the magnitudes of errors to be expected because of our limited state of knowledge, our best judgment (based on available data) indicates that $K_2(T)$ is best predicted by Equation 8.

The procedure for modifying the fatigue relations with temperature adopted is the use of Equation 6 to calculate values of $K_1(T)$ for specific values of seasonal pavement temperature and Equation 8 to calculate consistent values of $K_2(T)$.

Modifications with Mixture Stiffness

Although modifications to the fatigue relations for some applications may best be made with temperature as described previously, it may be more useful to modify them with mixture stiffness for other applications.

Table 1 provides dynamic moduli, fatigue relations at various dynamic moduli, the initial strain for each fatigue relation that will cause failure at 1 million load repetitions, identification of the source for the data, and other values of use in subsequent developments. These data were studied and plotted in various formats for use in developing a means of converting the fatigue relation from one stiffness to another. It was found that plots of the logarithm of the ratio of an arbitrary fatigue coefficient $K_1(E)$ to $K_1(E = 500\text{ ksi})$ against the logarithm of the ratio of the corresponding dynamic modulus to 500 ksi approximated straight lines. Plots of these relations for each of the first six mixtures that appear in Table 1 are shown in Figure 7. Specific points have been included for plots 1, 2, and 5 to illustrate slight nonlinearities. In drawing the straight lines, emphasis was given to fit for the higher values of stiffness where the relation was most linear.

The general equation for any of the lines plotted in Figure 7 is as follows:

$$\log [K_{1i}/K_1\ (E = 500\text{ ksi})] = 0 + S \log (E_i/500\text{ ksi})$$

where K_{1i} equals K_1 consistent with arbitrary value of dynamic modulus E_i, and S is the slope of the line (S will be negative in each case).

By moving S within the logarithm as an exponent and carrying $K_1(E = 500\text{ ksi})$ to the other side of the equation,

$$K_{1i} = K_1\ (E = 500\text{ ksi})\ (E_i/500\text{ ksi})^S \quad (9)$$

For generality, K_{1i} can be divided by another arbitrary base value, K_{1j}, so that any known value of K_1 that represents any specific mixture stiffness may be used to obtain others. The result of this division is

$$K_{1i} = K_{1j}\ (E_i/E_j)^S = K_{1j}\ (E_j/E_i)^{-S} \quad (10)$$

The values of slope for the plots in Figure 7 have also been entered on that figure and are ex-

Table 1. Fatigue relations for asphalt that vary with material stiffness.

Plot No.[a]	Reference No.	Dynamic Modulus E (ksi)	Log (E/500 ksi)	K_2	Initial Strain ϵ Leading to Failure at 10^6 Load Repetitions	$K_1 = 10^6 \epsilon^{k_2}$	$K_1 = K_1$ (E = 500 ksi)	Log [K_1/K_1 (E = 500 ksi)]	K_2/K_2 (E = 500 ksi)
1	1	100	-0.70	3.291		3.11×10^{-6}	3.95	0.60	1.0
		200	-0.40	↑		1.72×10^{-6}	2.20	0.34	
		500	0	Assumed		7.87×10^{-7}	1.00	0	
		800	0.20	Constant		5.27×10^{-7}	0.67	-0.17	
		1200	0.38	↓		3.73×10^{-7}	0.47	-0.32	
		2000	0.60	3.291		2.41×10^{-7}	0.31	-0.51	1.0
2	16	100	-0.70	5.26	4.8×10^{-4}	3.52×10^{-12}	39.8	1.60	1.027
		200	-0.40	4.95	3.4×10^{-4}	6.77×10^{-12}	76.5	1.88	0.967
		300	-0.22	5.12	2.3×10^{-4}	2.36×10^{-13}	2.7	0.42	1.0
		500	0	5.12	1.9×10^{-4}	8.86×10^{-14}	1.0	0	1.0
		1000	0.30	5.12	1.4×10^{-4}	1.86×10^{-14}	0.21	-0.68	1.0
		2000	0.60	5.12	1.1×10^{-4}	5.39×10^{-15}	0.061	-1.22	1.0
3	3	60	-0.92	5.00	8.2×10^{-4}	3.71×10^{-10}	2613	3.42	1.0
		150	-0.52		4.8×10^{-4}	2.55×10^{-11}	180	2.25	
		200	-0.40	↑	3.6×10^{-4}	6.05×10^{-12}	42.6	1.63	
		300	-0.22	Assumed	2.5×10^{-4}	9.77×10^{-13}	6.9	0.84	
		500	0	Constant	1.7×10^{-4}	1.42×10^{-13}	1.0	0	
		600	0.08	↓	1.5×10^{-4}	7.59×10^{-14}	0.54	-0.27	
		1000	0.30		0.9×10^{-4}	5.90×10^{-15}	0.04	-1.38	
		1500	0.48		0.6×10^{-4}	9.16×10^{-16}	0.0064	-2.19	
		2000	0.70	5.00	0.4×10^{-4}	9.02×10^{-17}	0.000 64	-3.20	1.0
4	11	40	-1.10	3.22	3.1×10^{-4}	5.20×10^{-6}	566	2.75	0.843
		75	-0.82	3.31	2.8×10^{-4}	1.63×10^{-6}	178	2.25	0.866
		150	-0.52	3.40	2.5×10^{-4}	5.66×10^{-7}	62	1.79	0.890
		300	-0.22	3.66	2.3×10^{-4}	4.46×10^{-8}	4.9	0.69	0.958
		500	0	3.82	2.1×10^{-4}	9.18×10^{-9}	1.0	0	1.0
		600	0.08	3.93	2.1×10^{-4}	3.25×10^{-9}	0.35	-0.45	1.029
		900	0.26	4.12	1.9×10^{-4}	4.76×10^{-10}	0.05	-1.28	1.079
		1500	0.48	4.28	1.8×10^{-4}	8.63×10^{-11}	0.009	-2.03	1.120
		4000	0.90	4.78	1.6×10^{-4}	7.13×10^{-13}	0.000 08	-4.10	1.251
5	14	100	-0.70	6.10	7.0×10^{-4}	5.65×10^{-14}	253	2.40	0.967
		200	-0.40	6.10	5.1×10^{-4}	8.18×10^{-15}	36.7	1.56	0.967
		300	-0.22	6.10	4.4×10^{-4}	3.32×10^{-15}	14.9	1.17	0.967
		500	0	6.31	3.7×10^{-4}	2.23×10^{-16}	1.0	0	1.0
		700	0.15	6.54	3.2×10^{-4}	1.44×10^{-17}	0.064	-1.19	1.036
		1000	0.30	6.54	2.8×10^{-4}	6.01×10^{-18}	0.023	-1.57	1.036
6	17	50	-1.00	2.59	1.4×10^{-4}	1.02×10^{-4}	21 030	4.33	0.710
		100	-0.70	2.85	1.3×10^{-4}	7.87×10^{-6}	1628	3.22	0.781
		250	-0.30	3.27	1.1×10^{-4}	1.16×10^{-7}	24	1.38	0.896
		500	0	3.58	1.0×10^{-4}	4.83×10^{-9}	1.0	0	1.0
		1000	0.30	4.07	9.5×10^{-5}	4.22×10^{-11}	0.0087	-2.06	1.115
		4000	0.90	5.07	9.0×10^{-5}	3.16×10^{-15}	6.55×10^{-7}	-6.18	1.389
	15	114	-0.64	2.66	2.4×10^{-5}	5.01×10^{-7}	786	2.90	0.729
		204	-0.39	3.20	3.7×10^{-5}	3.07×10^{-9}	4.8	0.66	0.877
		500	0	3.65	2.8×10^{-5}	6.37×10^{-10}	1.0	0	1.0
		584	0.67	3.73	2.7×10^{-5}	1.29×10^{-11}	0.02	-1.69	1.022

[a]Figure 7.

tremely variable, as might be expected from review of the similar information in Figure 3a. Although a value of the slope S for the particular mixture stiffness should be used when available for the transformations possible through application of Equation 9, it will be necessary, as for temperature, to select a slope that may be used as typical. The mean value for the six values of slope shown in Figure 7 is -3.84. However, it might also be reasonable to give more credence to the three lines that are very close together. If plots 3, 4, and 5 are given twice as much weight as the other three plots, the mean value is then -4.09. As this is a rather arbitrary decision in view of the variability, a value of -4 for the slope S was selected for convenience. Equation 10 then becomes

$$K_{1i} = K_{1j} (E_j/E_i)^4 \qquad (11)$$

Attempts to obtain better functions through regressing the data in Table 1 for plots 3, 4, and 5 did not result in functions that predicted any better than Equation 11. As no marked improvement was gained by the regression equations and they tended to stray from the origin and perform poorly in that vicinity, Equation 11 was selected for modifying K_1 with mixture stiffness. Although it may be used for any E_j, best results will be obtained for E_j as near 500 ksi as available.

Where specific fatigue test data are available, a mixture-specific value of the negative of slope S may be obtained and substituted for the value -4 assumed in Equation 11.

COMPARISONS OF FATIGUE CRACKING PREDICTIONS FOR FATIGUE CONSTANTS MODIFIED WITH TEMPERATURE AND WITH MIXTURE STIFFNESS

Procedures have been proposed for modifying the fatigue coefficient K_1 on the basis of temperature and on the basis of mixture stiffness by using Equations 6 or 11, respectively. Once a modified

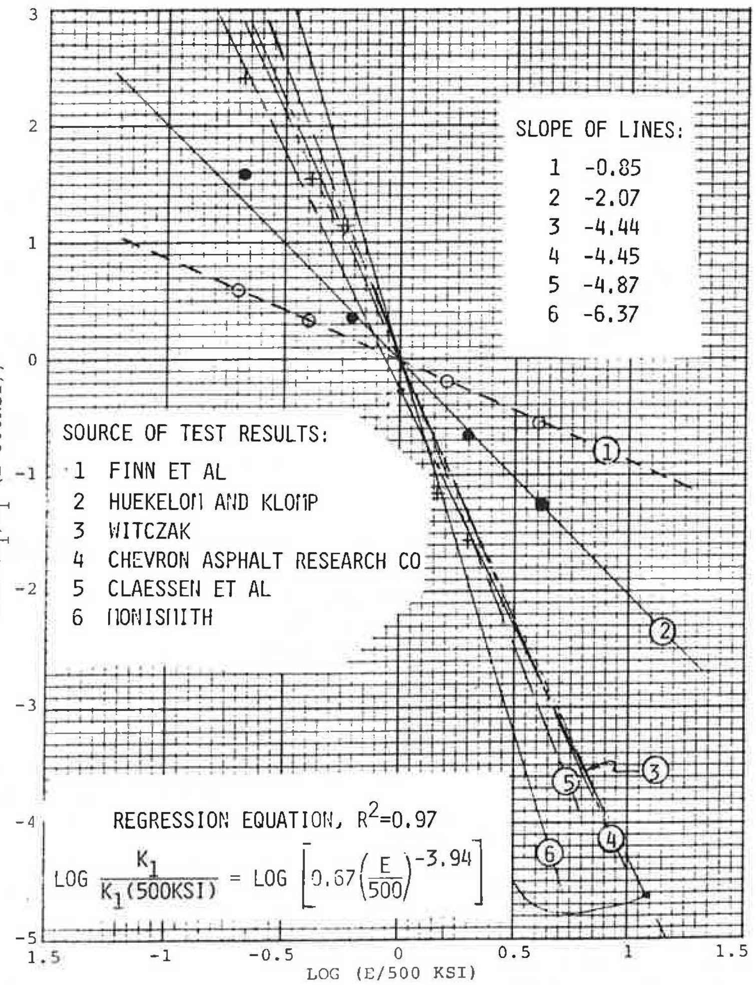


Figure 7. Plot of logarithms of ratio of K_1 for an arbitrary value of dynamic modulus E to K_1 when E is 500 ksi against E(ksi)/500.

value of K_1 is calculated, the exponent K_2 is obtained from Equation 8 in either case. These procedures were developed independently by selecting typical curves from widely scattered data (see Figures 3b and 7). As either procedure may reasonably be used, it will be useful to compare the resulting predictions for fatigue cracking.

Table 2 gives the results of applying these two procedures for two asphalt concrete mixtures and for three levels of tensile strain at the bottom of an asphalt concrete layer. The two mixtures were a relatively stiff mix reported by Witczak (18) and the AASHO Road Test mixture (4), which is of approximately average stiffness. Mixture stiffnesses related to mix temperatures were available and are included for each mix.

In view of the approximate nature of the relations used (Equations 6, 8, and 11), it is rather surprising that the differences in predicted load repetitions to 10 percent cracking are relatively small. For the 36 combinations of strain and fatigue relations calculated, the percentages of difference only exceeded 20 percent five times, four of which were for the larger strain of 0.001. The percentages of differences were in fact 10 percent or less for 26 of the 36 predictions.

For the more typical strain levels of 0.0001 to 0.0003, only 2 of the 24 sets of calculations resulted in differences greater than 20 percent, and both were for the higher temperature of 105°F past the range of the fatigue data on which the relations were developed. The differences for only 6 of the 24 exceeded 10 percent.

It appears from these comparisons that either temperature or mixture stiffness may be used as a basis for obtaining seasonal modifications to fatigue relations once a base relation in terms of either temperature or mixture stiffness is available.

MODIFYING TYPICAL FATIGUE RELATIONS TO REFLECT SPECIFIC MIX CHARACTERISTICS

Typical fatigue relations have been selected, and means of modifying these typical relations to account for changes in temperature or mixture stiffness were then proposed. The purpose of this section is to propose a procedure for modifying typical fatigue relations to reflect the characteristics of specific mixes. A thorough treatment of the variations in fatigue characteristics as related to various mix characteristics is not possible, as sufficient data do not exist in the literature for such a development. However, it is known that the fatigue life of a pavement is affected by at least the following parameters:

1. Mixture stiffness or dynamic modulus E,
2. Air voids,
3. Asphalt content,
4. Pavement temperature,
5. Viscosity of asphalt cement, and
6. Gradation and characteristics of the aggregate.

Table 2. Calculated fatigue cracking by using fatigue constants modified on basis of temperature and mixture stiffness.

		Fatigue Constants				Predicted Load Repetitions to 10 Percent Cracking					
		Based on Stiffness		Based on Temperature		Based on Stiffness			Based on Temperature		
Temperature (°F)	E (ksi)	$K_1 (E_i)$	$K_2 (E_i)$	$K_1 (T_i)$	$K_2 (T_i)$	$\epsilon = 1x10^{-3}$	$\epsilon = 3x10^{-4}$	$\epsilon = 1x10^{-4}$	$\epsilon = 1x10^{-3}$	$\epsilon = 3x10^{-4}$	$\epsilon = 1x10^{-4}$
105	245	$1.37x10^{-5}$	2.98	$4.08x10^{-4}$	2.60	$1.19x10^4$	$4.31x10^5$	$1.14x10^7$	$2.57x10^4$	$5.89x10^5$	$1.02x10^7$
85	500	$7.87x10^{-7}$	3.29	$2.55x10^{-6}$	3.16	$5.83x10^3$	$3.06x10^5$	$1.14x10^7$	$7.70x10^3$	$3.46x10^5$	$1.11x10^7$
70	810	$1.14x10^{-7}$	3.50	$1.14x10^{-7}$	3.50	$3.60x10^3$	$2.44x10^5$	$1.14x10^7$	$3.60x10^3$	$2.44x10^5$	$1.14x10^7$
65	940	$6.30x10^{-8}$	3.56	$4.47x10^{-8}$	3.60	$3.02x10^3$	$2.19x10^5$	$1.09x10^7$	$2.82x10^3$	$2.15x10^5$	$1.12x10^7$
45	1500	$9.72x10^{-9}$	3.77	$2.46x10^{-9}$	3.92	$1.98x10^3$	$1.86x10^5$	$1.17x10^7$	$1.42x10^3$	$1.59x10^5$	$1.18x10^7$
25	2300	$1.76x10^{-9}$	3.96	$3.79x10^{-10}$	4.12	$1.33x10^3$	$1.57x10^5$	$1.22x10^7$	$8.68x10^2$	$1.24x10^5$	$1.14x10^7$
5	3300	$4.15x10^{-10}$	4.11	$1.70x10^{-10}$	4.21	$8.87x10^2$	$1.25x10^5$	$1.14x10^7$	$7.25x10^2$	$1.15x10^5$	$1.18x10^7$
105	62	$3.33x10^{-3}$	2.37	$2.81x10^{-3}$	2.39	$4.29x10^4$	$7.44x10^5$	$1.01x10^7$	$4.16x10^4$	$7.39x10^5$	$1.02x10^7$
85	200	$3.07x10^{-5}$	2.89	$1.76x10^{-5}$	2.94	$1.44x10^4$	$4.65x10^5$	$1.11x10^7$	$1.16x10^4$	$4.01x10^5$	$1.01x10^7$
70	500	$7.87x10^{-7}$	3.29	$7.87x10^{-7}$	3.29	$5.83x10^3$	$3.06x10^5$	$1.14x10^7$	$5.83x10^3$	$3.06x10^5$	$1.14x10^7$
65	600	$4.00x10^{-7}$	3.36	$3.00x10^{-7}$	3.39	$4.81x10^3$	$2.75x10^5$	$1.10x10^7$	$4.44x10^3$	$2.62x10^5$	$1.09x10^7$
45	1220	$2.22x10^{-8}$	3.68	$1.69x10^{-8}$	3.71	$2.43x10^3$	$2.04x10^5$	$1.16x10^7$	$2.28x10^3$	$1.99x10^5$	$1.17x10^7$

Note: Base values are as follows: $K_1 = 7.87x10^{-7}$, $T_2 = 70°F$, $K_2 = 3.29$, and $E_j = 500$ ksi.

There is a limited amount of data available that indicates the effects of percentage of air voids, asphalt content, and mixture stiffness on the fatigue life of specific mixes. However, the interaction of these three parameters and the others listed above are not defined, so it appears necessary to select one of these three that is believed to explain the most variation in fatigue life and to base modifications to the fatigue characterizations on changes in that parameter. It is believed that the mixture stiffness explains much more about variations in the fatigue relations than any other parameter and, fortunately, more test data are available in terms of mixture stiffness. Therefore, variations in the fatigue relation will be based on variations in mixture stiffness for a specific mix tested at a specific temperature from the stiffness measured at the same temperature for the mixture on which the typical fatigue relation is based.

Equation 11 may be used to calculate the new base value of K_1 and Equation 7 to calculate a consistent base value of K_2. These base values may then be used to obtain other mixture-specific relations for other values of temperature or mixture stiffness.

OVERALL PROCEDURE FOR YEAR-ROUND CHARACTERIZATION OF FATIGUE LIFE

By using the procedure described above, fatigue relations may be obtained for specific mixes in terms of variations in their stiffness at a specified temperature from that of a typical mixture. These relations may then be modified in terms of variations in temperature or mixture stiffness to characterize fatigue life potential during various periods or seasons of the year. The only data needed are the mixture stiffness at 70°F, which can be obtained from a single dynamic-modulus test at 70°F or from a plot of mixture stiffness versus specimen temperature.

The first step is to obtain values of K_1 and K_2 that better represent the specific mixture. This is accomplished by substituting the mixture stiffness at 70°F into Equation 11 along with $K_{1j} = 7.87x10^{-7}$ and $E_j = 500$ ksi to calculate K_{1i} for the mixture. K_2 for the mixture is then obtained by substituting K_{1i} as K_1 in Equation 8.

The second step is to select analysis seasons for the year and to establish appropriate pavement temperatures to represent each season. Winter, summer, fall, and spring are usually used, sometimes with an additional short season in colder climates to represent spring thaw. (The calculation of pavement temperatures is itself a complex procedure and is outside the scope of this paper.)

Once the seasonal pavement temperatures have been established, seasonal temperatures or mixture stiffnesses may be obtained and used in Equations 6 or 11 to obtain seasonal values of K_1. Seasonal values of K_2 may then be obtained through use of Equation 8.

These seasonal fatigue relations may then be used along with calculated strains (considering seasonal values of layer stiffnesses) to arrive at values of load repetitions to failure that represent each season. Miner's hypothesis may then be used with ratios of seasonal traffic estimates to seasonal failure repetitions to accumulate consumption of fatigue life with time or traffic.

PREDICTING FATIGUE LIFE AS AREA CRACKED

Prediction of failure in fatigue depends on the definition of failure for which the fatigue relation is based. When sufficient loads have been experienced for the linear summation of cycle ratios (Miner's hypothesis) to reach unity, a damage index (DI) is defined as unity and the pavement is considered failed.

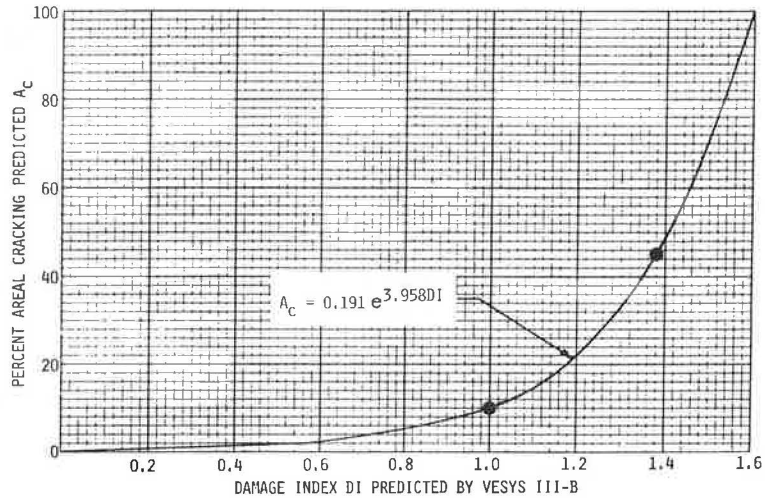
As measurement of fatigue cracking is usually as a percentage of areal cracking, it would be more convenient to predict fatigue in terms of areal cracking than as fraction failed, as is usually currently done. As the typical fatigue relation is based on 10 percent areal cracking, we know that a predicted DI of unity corresponds to 10 percent cracking for the typical fatigue relation adopted. Review of fatigue relations by Finn and others (1) indicates that the only difference is in K_1, so division of K_1 at 45 percent cracking by that at 10 percent cracking ($1.084x10^{-6} \div 7.87x10^{-7} = 1.38$) indicates that a damage index of 1.38 will correspond to 45 percent cracking. These two points are plotted in Figure 8, and a relation that is believed to represent a reasonable distribution has been developed and plotted through these points. Comparisons with similar developments for different purposes by Von Quintus and others (19) on the basis of the same fatigue relations indicate an almost identical shape for the relation.

It is proposed that the relation below be used to transform predicted DI to percentage of areal cracking (A_c):

$$A_c = 0.19e^{3.96DI} \tag{12}$$

Inspection of Figure 8 indicates that Equation 12 will predict the first noticeable crack at the

Figure 8. Curve for transforming DI to percentage of areal cracking.



surface when the predicted damage index reaches 0.4. Ten percent cracking will be predicted at a DI of unity, 45 percent at a DI of 1.38, and 100 percent at a DI of 1.583. For constant rates of traffic (numbers and axle-load distributions), this means that 40 percent as much traffic is required to produce the first crack as is necessary to induce 10 percent cracking. Thirty-eight percent more traffic is required to progress from 10 to 45 percent cracking. For example, in the primary range of interest between 5 and 45 percent cracking, Equation 12 is believed to provide reasonable predictions for the growth of areal cracking.

SUMMARY

The evaluation of fatigue life for asphalt concrete pavements has in the past been very difficult because of limited knowledge as to fatigue damage relations for real pavements, availability of laboratory test data for only a limited set of mixtures, and limited information on the variations of fatigue life potential with pavement temperature and mixture characteristics. This paper offers procedures to overcome some of these difficulties and to improve the confidence levels at which such evaluations may be made. Although the typical fatigue relations and approximate procedures proposed are considered to offer a much greater opportunity for successful prediction of fatigue life than has been generally possible, no claims of great accuracy are made and fatigue life characterizations for specific mixtures in place should be used in the fortunate and unusual event that they should be available.

The proposed transformation from prediction of a DI to prediction of percentage of class 2 and 3 areal cracking is believed to be more meaningful to most engineers. It also offers an interval of prediction rather than relation to a single condition defined as failure. As with many fatigue-related functions, however, the rate of development of areal cracking used in developing Equation 12 is based on only one mixture and environment, and it might be expected to vary for others, as well as for other changed conditions.

Prediction of fatigue life in asphalt concrete pavements may vary considerably from actual fatigue performance in the field; however, the accuracy of such predictions has improved in the past decade due to contributions by numerous researchers. It is hoped that the procedures proposed will contribute to this positive trend.

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Structural Design of Flexible Pavements: A Simple Predictive System

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During the past two decades, much effort has gone into the development of rational pavement design procedures that are intended to be integrated into a more general framework. The Federal Highway Administration has developed a computer program package known as VESYS II to predict the structural responses and hence the integrity of flexible pavements. The program is quite formidable for the local design engineer. Therefore, a simple predictive system is needed to be widely used for the structural design of flexible pavements. A simple computer program package is presented that includes (a) regression formulas for tensile strain and rut depth computations, cracking prediction, and evaluation of the rut depth variance; (b) modification and calibration of the American Association of State Highway Officials (AASHO) Road Test serviceability model, where rut depth variance replaces the slope variance; (c) seasonal (monthly) characterization of pavement materials and discrete representation of axle-load distribution; and (d) special treatment for overlay analysis. The procedure is illustrated and the results are discussed. The good agreement between the results and measured values and the simplicity of the program make it very attractive. It could be programmed on a desk (micro) computer.

During the past two decades, much effort has gone into the development of rational pavement design procedures that were intended to be integrated into a more general framework. The Federal Highway Administration (FHWA) (1) has developed a computer program package known as VESYS II (and its modifications and extensions VESYS IIM, VESYS A, and VESYS G) to predict the structural responses and hence the integrity of flexible pavements. The program is based on an advanced viscoelastic analysis that is, however, a rather significant departure from more conventional systems. The implementation and use of the VESYS package seems appropriate for a statewide study, but it appears quite formidable for the design engineer. At the same time, pavement structural subsystems have been developed for the Transportation Research Board (2). The computer programs--one for fatigue cracking and permanent deformation (named PDMAP) and one for low-temperature cracking (named COLD)--were proposed for implementation and field calibration. The PDMAP package does not include the synthesis of the two subsystems and does not predict pavement performance.

This paper presents the integration of the fatigue and permanent deformation subsystems into a computer program package. The proposed system is simple, reliable, and also general, which facilitates its use by most people who deal with pavement design and maintenance. It includes the following:

1. Modification and calibration of the American Association of State Highway Officials (AASHO) Road Test serviceability model to enable pavement performance prediction by using computed damages,
2. Cracking prediction based on the commonly used fatigue law and probabilistic considerations to express mechanistic variables into cracked areas,
3. Permanent deformation prediction based on quasi-elastic analysis,
4. Regression formulas and closed-form probabilistic solutions for evaluation of the variables involved in the performance model (the probabilistic solutions are similar to those used in VESYS G),
5. Overlay application analysis to permit pavement maintenance strategy studies, and
6. Seasonal (monthly) characterization of pavement materials and discrete representation of axle-load distribution.

SERVICEABILITY MODEL

The AASHO Road Test serviceability model relates serviceability index to variables that describe pavement damage, i.e., cracking, patching, rutting, and slope variance (3). The AASHO model reads as follows:

$$PSI = 5.03 - 1.91 \log_{10} (1 + \overline{SV}) - 1.38 \overline{RD}^2 - 0.01 \sqrt{\overline{C} + \overline{P}} \quad (1)$$

$R^2 = 0.84$