DAMAGE MODEL FOR PREDICTING TEMPERATURE CRACKING IN FLEXIBLE PAVEMENTS

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A model for predicting temperature cracking has been developed. Temperature cracking as predicted by the model is the appropriate addition of low-temperature cracking, which occurs when the thermal tensile stress exceeds the asphalt concrete tensile strength; and thermal-fatigue cracking, which occurs when the thermal-fatigue distress due to daily temperature cycling exceeds the asphalt concrete fatigue resistance. During model development, stochastic variations in material properties were considered. The model has since been computerized. Inputs to the program are the basic material properties and conventional weather variables that can be easily obtained. The major output from the program is the temperature cracking in ft/1,000 ft^2 (m/1000 m^2) as a function of age from construction. Analysis of the Ontario test roads and the Ste. Anne Test Road has shown the model predictions to be reasonable. The model is in a modular form so that any change that may develop through the advancement of asphalt concrete technology can be added without major revision of the basic framework.

TEMPERATURE cracking is one of the severe problems with flexible pavements in the United States and Canada. The problem is not only the bad effect on the highway user but also the distress that occurs in the pavement later. The consequence, which depends on the type of subgrade, could be loss of support or swelling, but above all there is a loss of ridability and an increase in the frequency and cost of maintenance. The most common method for selecting an asphalt concrete mixture to avoid temperature cracking involves determining the fracture temperature, the temperature at which the tensile stress exceeds the tensile strength. According to this method, the pavement will fail thermally as soon as its temperature drops to the fracture temperature. However, there have been many cases in which only a few thermal cracks form at first, which increase in number yearly until the road is considered to be failed. The purpose of this research is to develop a model for predicting temperature cracking in asphalt concrete during its service life by using materials laboratory data and available weather information. Temperature cracking as predicted is the addition of two forms of cracking:

1. Low-temperature cracking, which occurs when the thermal tensile stress exceeds the asphalt concrete tensile strength; and
2. Thermal-fatigue cracking, which occurs when the thermal-fatigue distress due to daily temperature cycling exceeds the asphalt concrete fatigue resistance.

In comparisons of the temperature cracking predicted by the model and that measured in the Ontario test roads (1) and the Ste. Anne Test Road (2, 3, 4), the model predictions have been reasonable. The model is a tool that will help the highway design engineer select the most appropriate asphalt concrete mixture that will result in no or very little temperature cracking. The model can also be used to distinguish among the different

Publication of this paper sponsored by Committee on Flexible Pavement Design.

asphalt suppliers and to select the best asphalt for avoiding temperature cracking; this will help reduce the maintenance cost.

DESIGN APPROACH

To make the right approach to any problem requires that the causes be known. After the causes are known, the next step is to develop models for analyzing the problem; then one can construct a simple and useful system.

A general system approach to pavement design involves

1. Inputs—material characteristics, load frequency and intensity, environmental conditions, variations associated with the inputs, etc.;
2. Submodels—techniques developed to analyze the problem under consideration, which can be based on theory, experience, axioms, etc.;
3. Outputs—stress, strain, strength, etc.;
4. Distress—cracks, roughness, rutting, etc.; and
5. Performance—evaluation of distress manifestations from the user's point of view.

The major part of the model is the development of the temperature cracking submodels. The four submodels that were developed, each of which has its own function and serves as an input to the next one, are

1. Submodel 1—simulation of pavement temperatures;
2. Submodel 2—estimation of asphalt concrete stiffness, prediction of in-service aging of asphalt, and estimation of thermal stresses;
3. Submodel 3—prediction of low-temperature cracking; and
4. Submodel 4—prediction of thermal-fatigue cracking.

Because of space limitations, however, only submodels 3 and 4 are discussed here; complete details of submodels 1 and 2 will be published in a future report.

PREDICTION OF LOW-TEMPERATURE CRACKING

It is believed that asphalt concrete properties vary over the entire road length and, therefore, a single fracture temperature is an unsatisfactory criterion. Instead, the variability of the mixture properties should be accounted for by an appropriate stochastic approach.

The factors that control low-temperature cracking are the stress σ and the strength T. So that the variability of asphalt concrete properties in a particular road may be accounted for, it is assumed that both the stress and the strength vary normally and randomly along that road. The probability of failure is then defined as the probability of the stress exceeding the strength at any point on the road:

$$P(\text{failure}) = P(F) = P(\sigma > T) \tag{1}$$

By introducing $X = \sigma - T$, Eq. 1 can be rewritten as

$$P(F) = P(\sigma - T > 0) = P(X > 0) \tag{2}$$

Figure 1 shows a conceptual diagram showing the probability of failure on the normal distribution of X.

Because the density functions $f(\sigma)$ and $f(T)$ are assumed to be normally distributed, $f(X)$ is normally distributed and

$$f(X) = \frac{1}{SD_X\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{X - \overline{X}}{SD_X}\right)^2\right] \tag{3}$$

where

Figure 1. Difference distribution (X = stress strength).

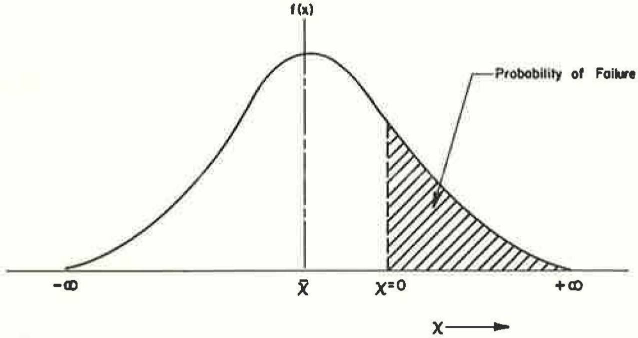
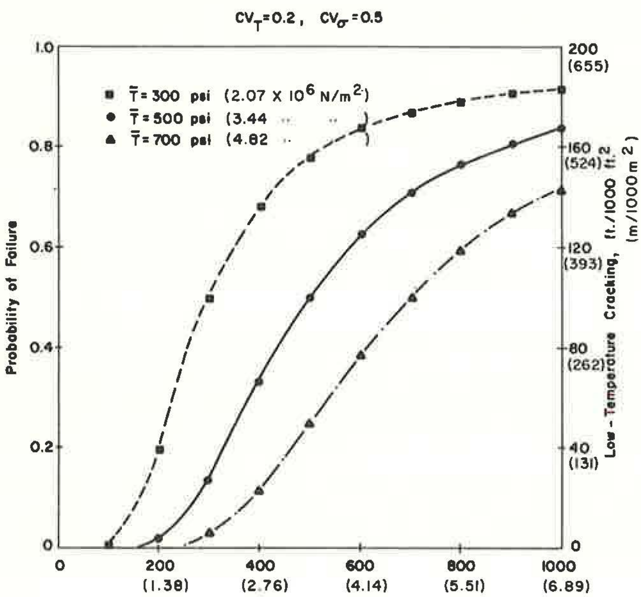


Figure 2. Effect of mean strength on low-temperature cracking.



$\bar{T}$ = mean value of the tensile strength,

CV_T = coefficient of variation of the strength,

CV_σ = coefficient of variation of the stress.

$f(X)$ = the density function of X,
SD_X = standard deviation of X, and
$\overline{X}$ = mean value of X.

Therefore,

$$P(F) = P(X > 0) = \int_0^{\infty} f(X)dX \tag{4}$$

By substituting Eq. 3 into Eq. 4,

$$P(F) = \frac{1}{SD_X\sqrt{2\pi}} \int_0^{\infty} \exp\left[-\frac{1}{2}\left(\frac{X-\overline{X}}{SD_X}\right)^2\right] dX \tag{5}$$

Variable X was normalized so that the normal tables could be used:

$$Z_X = \frac{X - \overline{X}}{SD_X} \tag{6}$$

Accordingly, the limits of the integration in Eq. 5 will be

1. Where X = o,

$$Z_{X_{min}} = -\frac{\overline{X}}{SD_X}$$

2. Where $X = \infty$,

$$Z_{X_{max}} = \infty$$

3. $dX = SD_X dZ$.

Equation 5 can then be rewritten in terms of Z as

$$P(F) = \frac{1}{\sqrt{2\pi}} \int_{Z_{X_{min}}}^{Z_X = \infty} e^{-\frac{z^2}{2}} dZ \tag{7}$$

If the lower limit of the integration of Eq. 7 is known, then the normal tables can be used to determine the probability of failure P(F):

$$Z_{X_{min}} = \frac{-X}{SD_X} = -\frac{(\overline{\sigma} - \overline{T})}{\sqrt{SD_{\sigma}^2 + SD_T^2}} \tag{8}$$

where

$\overline{\sigma}$ = mean value of the stress,
$\overline{T}$ = mean value of the strength,
SD_{σ} = standard deviation of the stress, and
SD_T = standard deviation of the strength.

As an example, the following values were assumed:

$\bar{\sigma}$ = 100 psi (689.4 kPa),
SD_σ = 50 psi (344.7 kPa),
$\bar{T}$ = 200 psi (1378.9 kPa),
SD_T = 40 psi (275.7 kPa), and

$$Z_{X_{\min}} = -\frac{(100 - 200)}{\sqrt{\overline{50}^2 + \overline{40}^2}} \cong +1.56.$$

From the normal tables, P(F) ≅ 6.0 percent, which means that 6.0 percent of the area of a road will fail if the assumed values of stress and strength occur.

Because thermal cracks take the form of transverse cracks, they are usually reported as the average frequency per mile or, as reported in the AASHO Road Test, in lin ft/1,000 ft^2. The minimum spacing between transverse cracks ranges from 4 to 5 (1.2 to 1.5 m). Consequently, it can be assumed that, if the spacing between transverse cracks reaches 5 ft (1.5 m) and the pavement is no longer restrained, the area of influence of each transverse crack will be equal to its length times a width of 5 ft (1.5 m). Therefore, to transfer a predicted area of thermal cracking, the area can be divided by the width of influence, which is about 5 ft (1.5 m).

For example, if the probability of failure is 6.0 percent, 60 ft^2 will fail every 1,000 ft^2 (60 m^2/1000 m^2). In terms of linear cracking that will be 60/5 or 12 ft/1,000 ft^2 (60/1.5 or ~ 40 m/1000 m^2).

Equation 8 is mainly used in the low-temperature cracking submodel. The four variables in Eq. 8 were varied over a reasonable range so that the model's behavior could be studied. The results of this analysis are shown in Figures 2, 3, and 4 from which the following conclusions are drawn:

1. When the average tensile stress is equal to the average tensile strength, the probability of failure is 50 percent, regardless of the stress and strength coefficients of variation.
2. For both stress and strength, the higher the coefficient of variation is, the higher the low-temperature cracking will be, up to a probability of failure of 50 percent, after which the reverse is true.

PREDICTION OF THERMAL-FATIGUE CRACKING

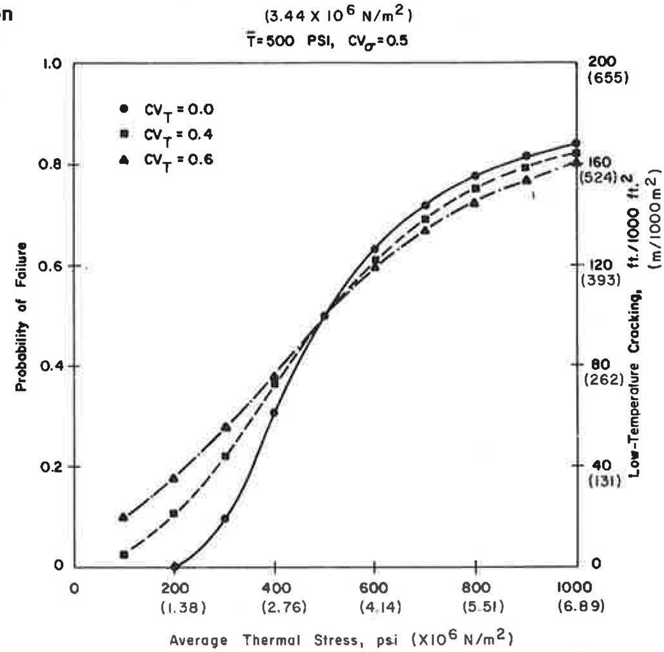
Pavement behavior (stress, strain, etc.) under temperature cycling was analyzed so that the relation between temperature cycling and the fatigue concept could be studied. The analysis showed that temperature cycling simulates a constant strain rather than a constant stress-fatigue distress. The distress effect of each cycle depends on the maximum stiffness and strain during that day (cycle) (Fig. 5). The pavement is subjected to one cycle/day (360 cycles/year); each cycle has a different distress intensity from all others. Furthermore, hardening of asphalt is an important phenomenon that should be considered. As time passes, the asphalt gets harder and hence, on the average, the asphalt concrete stiffness increases year after year. It is believed that stiffness is the major factor distinguishing asphalt concrete mixes, with reference to their ability to withstand repeated temperature cycling. Figure 6 shows a conceptual relation between strain level and the number of cycle applications until failure for different stiffnesses. The general relation may be written as

$$\bar{N}_{ij} = A_j\left(\frac{1}{\epsilon_i}\right)^{B_j}$$

where

i = the strain level,
j = the stiffness level,
$\bar{N}_{ij}$ = the average number of cycle applications until failure under strain level i and stiffness level j,

Figure 3. Effect of coefficient of variation of strength on low-temperature cracking.

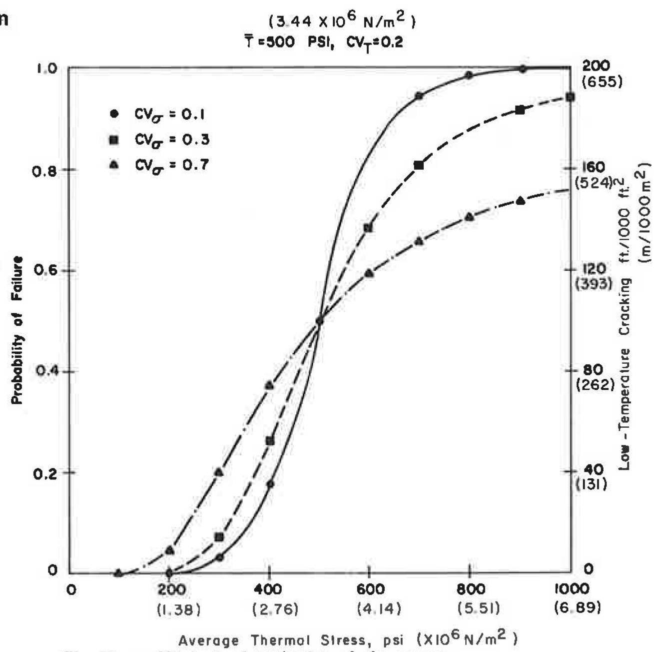


CV_T = coefficient of variation of the strength,

$\bar{T}$ = mean value of the tensile strength,

CV_σ = coefficient of variation of the stress.

Figure 4. Effect of coefficient of variation of stress on low-temperature cracking.



CV_σ = coefficient of variation of the stress,

$\bar{T}$ = mean value of the tensile strength,

CV_T = coefficient of variation of the strength.

Figure 5. Schematic diagram of assumed behavior of pavement strain, stiffness, and stress during normal day.

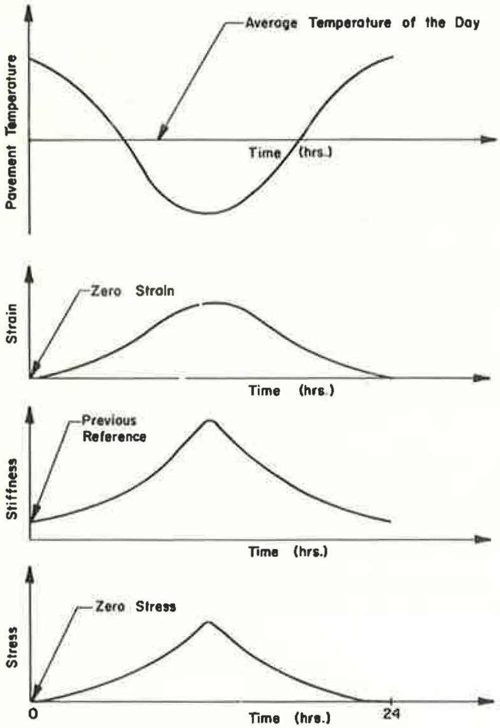
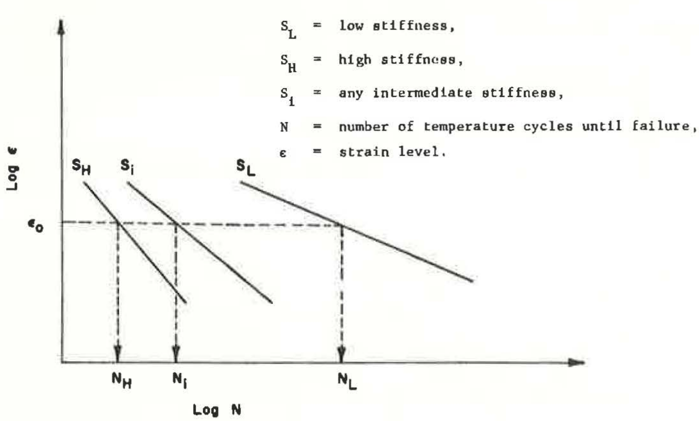


Figure 6. Conceptual diagram of relation between strain and number of cycle applications until failure under a constant strain fatigue mode.



ϵ = strain, and

A_j, B_j = fatigue constants at a stiffness level j.

According to the preceding concept, the fatigue constants will vary with stiffness. An experiment was designed to determine these constants in the laboratory and to establish a criterion for estimating the cumulative damage. However, because of the high cost of such an experiment, it was suggested that the experiment be performed later, and, therefore, fatigue constants were estimated from available data. So that the cumulative damage due to temperature cycling could be estimated, the following formula was used:

$$D = \sum_{i=1}^{K} \sum_{j=1}^{M} \frac{n_{ij}}{N_{ij}} = \frac{n_{11}}{N_{11}} + \frac{n_{12}}{N_{12}} + \ldots + \frac{n_{1M}}{N_{1M}} + \frac{n_{21}}{N_{21}} + \frac{n_{22}}{N_{22}} + \ldots + \frac{n_{2M}}{N_{2M}} + \ldots + \frac{n_{K1}}{N_{K1}} + \frac{n_{K2}}{N_{K2}} + \ldots + \frac{n_{KM}}{N_{KM}} \qquad (9)$$

where

D = accumulated damage,

K = number of equal strain level groups,

M = number of equal stiffness level groups,

n = actual number of cycle applications, and

N = number of cycle applications until failure.

In this formula, it was assumed that the damage caused by each cycle was irrecoverable and hence the cumulative damage was a simple addition of all individual damages disregarding their sequence of occurrence.

The logarithm of the average number of cycles until failure $\overline{N}_{ij}$ has been shown to be normally distributed (5). For a particular significance level α, the number of cycle applications until failure $N_{\alpha_{ij}}$ can be expressed as

$$\log N_{\alpha_{ij}} = \log \overline{N}_{ij} - Z_\alpha SD_{\log N} \qquad (10)$$

where

Z_α = value from the normal tables that corresponds to a significance level α; and

$SD_{\log N}$ = standard deviation of the logarithm of N.

From Eqs. 9 and 10, the probability of failure P(F) can be expressed as

$$P(F) = \text{probability}\left(\sum_{i=1}^{K} \sum_{j=1}^{M} \frac{n_{ij}}{N_{\alpha_{ij}}} \geq 1.0 \right) \qquad (11)$$

The best way to explain the above concept is through a numerical example.

For a particular road section under particular environmental conditions, the accumulated damage $\left(\sum_{i=1}^{K} \sum_{j=1}^{M} \frac{n_{ij}}{N_{\alpha_{ij}}} \right)$ was estimated after each month from construction at different significance levels. The relationship between the accumulated damage and the significance levels after x months from construction is shown in Figure 7 which shows that P(F) = 8 percent. If one wants to transfer the probability of failure into cracking, the previously explained procedure must be used: Cracking in $ft^2/1{,}000\ ft^2$ ($m^2/1000\ m^2$) = $0.08 \times 1{,}000 = 80.0$, and cracking in lin ft/1,000 ft^2 ($m/1000\ m^2$) = 80.0/5.0 = 16.0.

The estimated cracking from this model is referred to as thermal-fatigue cracking. The developed submodel for predicting thermal-fatigue cracking is unique in nature

Figure 7. Relation between accumulated damage and significance levels after x months.

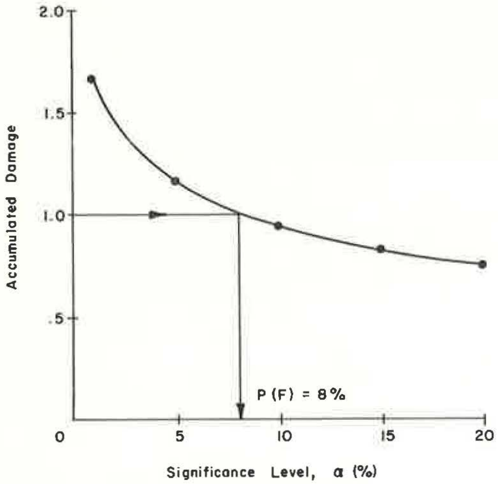
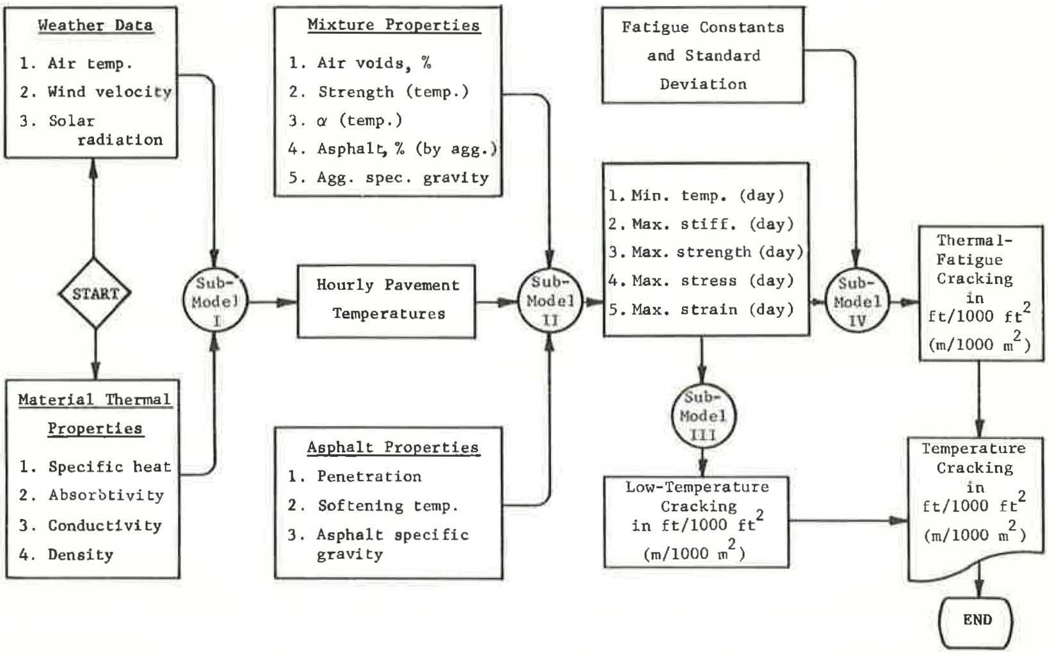


Figure 8. Summary flow chart of developed temperature cracking model.



because this is the first time that both fatigue and stochastic concepts are being used to predict the distress resulting from temperature cycling. The usefulness and the behavior of the model are discussed in the next section.

WORKING MODEL

A summary flow chart of the developed computer program for the damage model is shown in Figure 8. The involved steps were

1. Calculate the daily mean air temperature and solar radiation;
2. Calculate hourly pavement temperature for each day;
3. Locate the maximum and minimum pavement temperatures for each day;
4. Estimate the stiffness at the middle of the temperature intervals and the increments of strain and stress by starting from the maximum temperature and moving down, on an hourly basis, to the minimum temperature;
5. Accumulate the increments of strain and stress to estimate the maximum strain and stress for that day;
6. Estimate the strength corresponding to the maximum stress;
7. Predict low-temperature cracking;
8. Predict thermal-fatigue cracking; and
9. Add the low-temperature and thermal-fatigue cracking to obtain the total temperature cracking.

Model Behavior

Figure 9 shows the relationship between temperature cracking and the number of years from construction as predicted by the model. In Figure 9, the values of temperature cracking correspond to assumed asphalt mixture properties and surrounding environmental conditions, and they are not necessarily typical values. However, the rate of increase in low-temperature and thermal-fatigue cracking is usually similar to what is shown; i.e., the rate of increase of low-temperature cracking is usually much less than that for thermal-fatigue cracking. Study cases performed with the system showed that the rate of increase of thermal-fatigue cracking is higher during the winter than the summer (Fig. 10). Furthermore, it is important to note that the major cause of temperature cracking is low temperature or thermal fatigue, depending on the asphalt mixture properties and the surrounding environmental conditions.

Model Verification

A search was carried out to locate some projects in which temperature cracking was measured and reported separately from traffic load cracking. Unfortunately, very few projects were found where such measurement was reported. Two of these projects were used to verify the system. A description of each project and the results of the analysis follow.

Ontario Test Roads—In this project, McLeod (1) made a survey of temperature cracking after 8, 9, 10, and 11 years of service of asphalt pavements on 3 southwestern Ontario test roads about 40 miles (64.4 km) apart that were constructed in 1960, all over clay subgrades. Each test road was 6 miles (9.66 km) long and contained three 2-mile (3.22 km) test pavements. The pavement in each 2-mile (3.22 km) test section contained a single 85/100 penetration asphalt cement. Three 85/100 penetration asphalt cements from three different asphalt suppliers were used in each of the three 6-mile (9.66 km) test roads. The properties of the asphalts from the different suppliers are given elsewhere (1). All the necessary information about the mixture properties was available except the tensile strength, which was assumed to have a maximum value of 500 psi (3.45 MPa). The environmental variables were estimated from the closest available weather station (6). The fatigue constants were kept the same throughout the verification. Figures 11, 12, and 13 show the comparison between the measured and predicted thermal cracking for the three asphalt suppliers. Because there is not any basis on which to differentiate between the three roads, they can be considered as replicates. However, because the fatigue constants were adjusted with only one section

Figure 9. Schematic diagram of relation between thermal cracking and the number of years from construction (not necessarily typical).

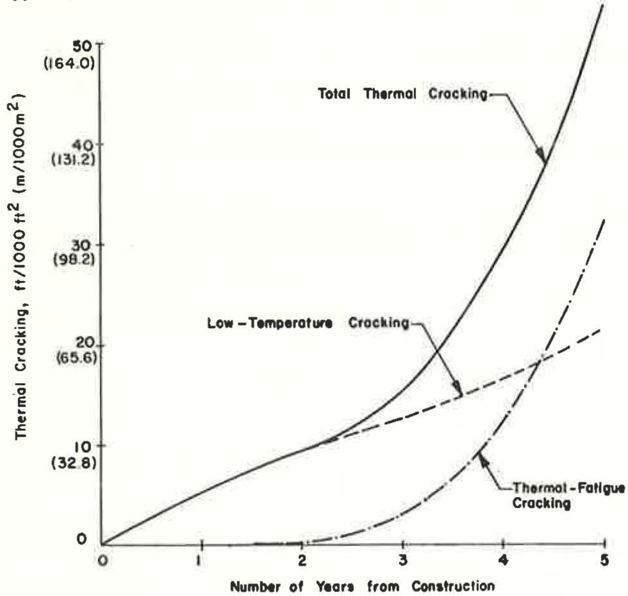


Figure 10. Detailed diagram of increase in thermal-fatigue cracking after each month from construction.

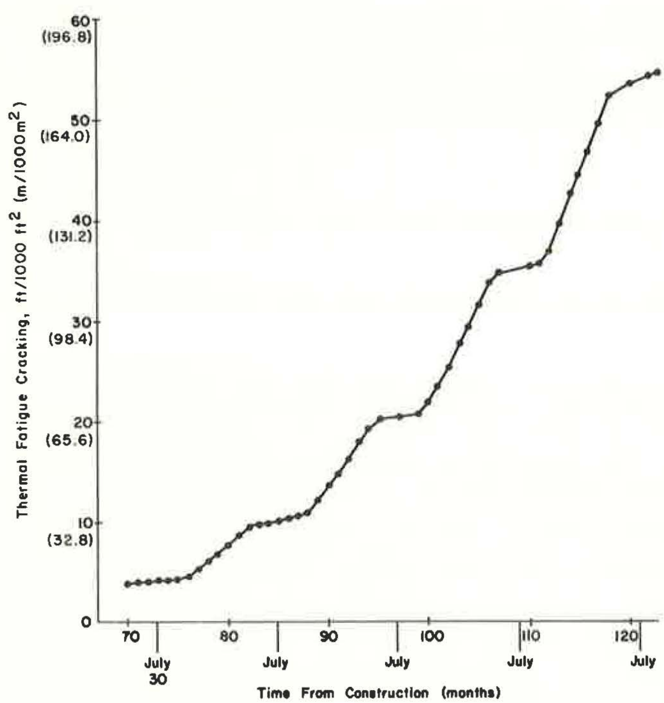


Figure 11. Comparison of predicted and measured thermal cracking (asphalt supplier 1).

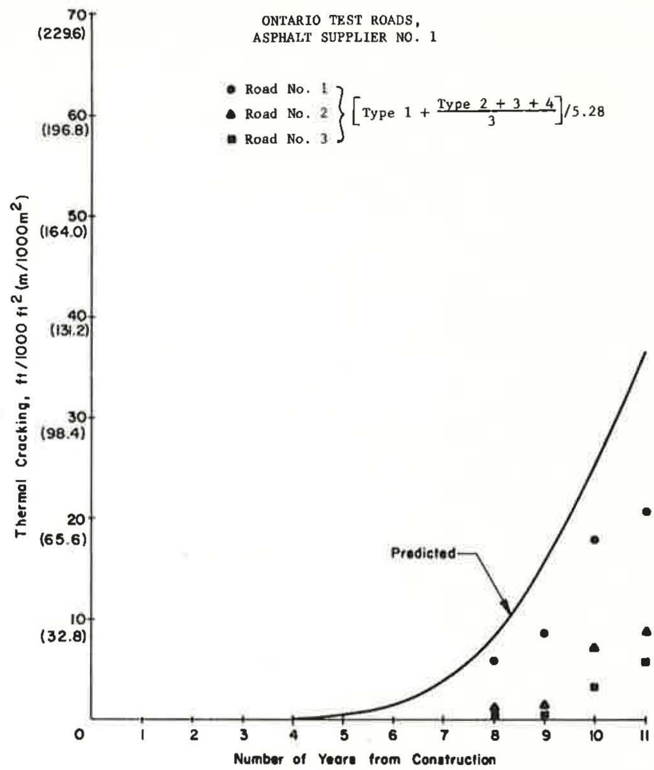


Figure 12. Comparison of predicted and measured thermal cracking (asphalt supplier 2).

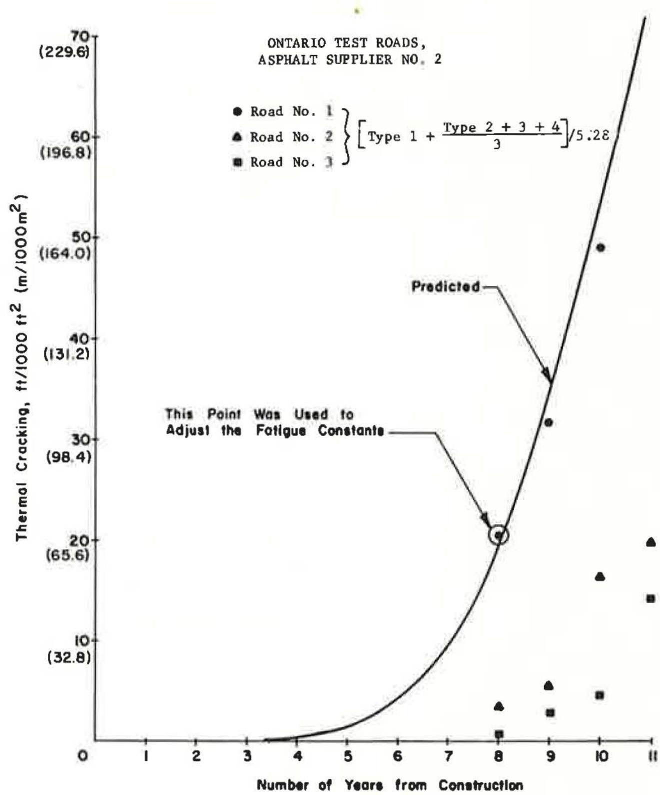


Figure 13. Comparison of predicted and measured thermal cracking (asphalt supplier 3).

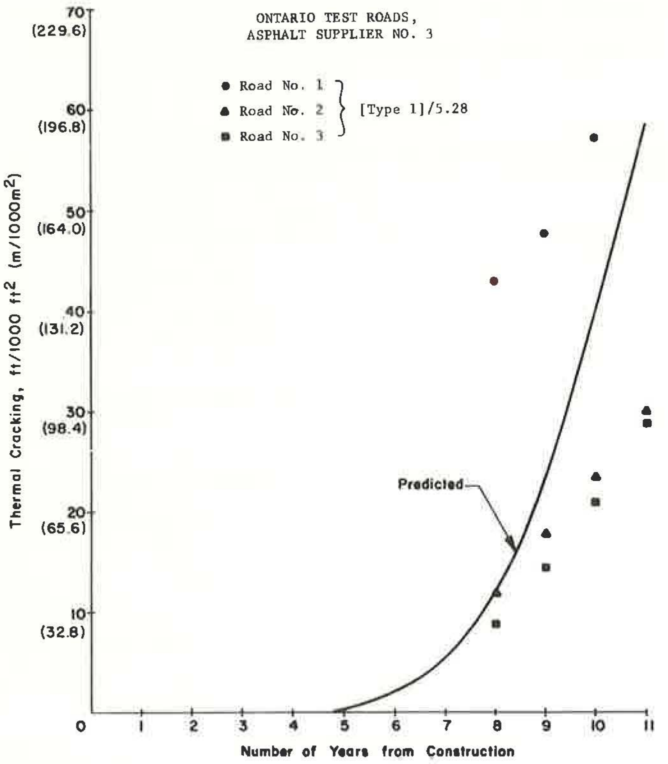
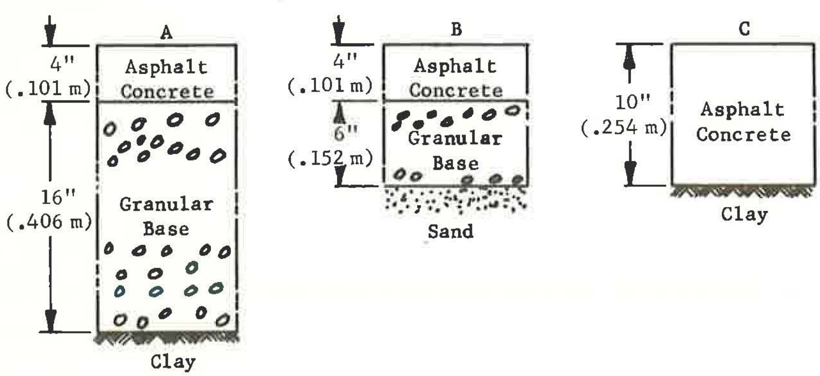


Table 1. Comparison of measured and predicted temperature cracking after 2 years from construction (Ste. Anne Test Road).

Asphalt Type	Section	Structure	Measured Crack (ft/1,000 ft^2) (4)	Average (ft/1,000 ft^2)	Predicted Crack (ft/1,000 ft^2)
150/200 LVA	63	A	51.0		
	67	B	154.0	76.0	98.9
	64	C	22.9		
150/200 HVA	62	A	7.5		
	66	B	5.6	5.5	9.5
	65	C	3.3		
300/400 LVA	61	A	25.0	13.1	1.7
	68	B	1.25		

Note: 1 ft = 0.3048 m. 1 ft^2 = 0.0929 m^2.



(road 4, asphalt supplier 2), it would be more appropriate to compare the predicted thermal cracking with that measured in road 1. In general, the agreement between the measured and predicted cracking seems to be encouraging.

Ste. Anne Test Road—The test road (2, 3, 4) was constructed in 1967 for the study of transverse cracking of asphalt pavements. It is located 25 miles (40.2 km) east of Winnipeg near Ste. Anne, Manitoba. The characteristics of the test road were described (4) as follows: "The road is composed of twenty-nine 400-ft (122 m) pavement sections, 24 ft (7.32 m) wide, constructed on clay and sand subgrades. The test section variables include two different types and three different grades of asphalt, two asphalt contents, two aggregate gradations, limestone and granite aggregates and three road structure designs." These variables were selected because it was thought that they were potentially important in the study of transverse pavement cracking. All the mixture properties are available (2, 3, 4) except the maximum tensile strength, which was determined for samples containing the optimum asphalt content by Christison et al. (7). The fatigue constants were kept the same as for the Ontario test roads. The comparison between the measured and predicted temperature cracking is given in Table 1, which indicates that the agreement is reasonable.

SUMMARY

A damage model for predicting temperature cracking is described. Analysis of the Ontario test roads and the Ste. Anne Test Road has shown that the model predictions are reasonable. The model has been computerized. The inputs to the program are the basic material properties and the conventional weather variables that are easy to obtain. The major output from the program is the temperature cracking in ft/1,000 ft^2 (m/1000 m^2), which is measured each year from construction. Temperature cracking as predicted from the model is the appropriate addition of low-temperature and thermal-fatigue cracking. The model has been developed in a modular form so that any change that may develop through the advancement of asphalt concrete technology can be added without major revision of the basic framework.

ACKNOWLEDGMENTS

This investigation was conducted at the Center for Highway Research at the University of Texas at Austin. The authors wish to thank the sponsors, the Texas Highway Department, and the Federal Highway Administration, U.S. Department of Transportation.

The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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DISCUSSION

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The paper by Shahin and McCullough represents a sound approach to the development of a model for predicting the extent of temperature cracking of flexible pavements. The problem is complex and congratulations are extended to the authors for a thorough and scholarly paper. However, several factors in the paper appear overly generalized, which leaves room for comment.

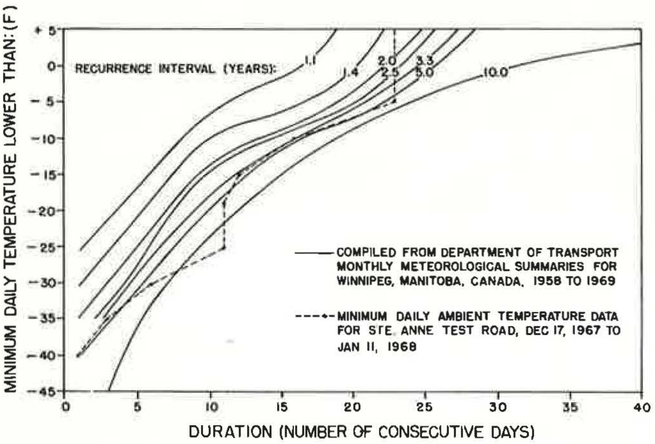
INITIATION AND PROGRESSION OF TRANSVERSE CRACKING

The initiation of transverse cracking in the Ste. Anne Test Road pavements was detected with continuously monitoring crack detection circuits and was recorded automatically with pavement temperatures measured at various depths (13). The pavements were inspected several times a week during the first two winters and at less frequent intervals in subsequent years.

Most of the transverse cracking occurred in the first winter after construction (1967/1968) during prolonged low-temperature cycles, when the asphalt concrete was cooled throughout its thickness (4). A study of 10 years of minimum daily temperature data was carried out to determine the most severe annual low-temperature cycle in southern Manitoba and to estimate its interval of recurrence (14). Figure 14 shows that the low-temperature cycle, during which many of the test pavements experienced their greatest cracking, has an estimated recurrence interval of 5 years in the lowest temperature range. If the potential for pavement stress buildup has not been eliminated altogether by transverse cracking, additional cracking with time could be expected as the pavements are exposed to low-temperature cycles that are more extreme or of greater duration than those experienced previously. Subsequent winters have not appeared to be much more severe than the 1967/1968 winter, and some of the pavements that cracked extensively have exhibited little or no additional transverse cracking. Some of the pavements that cracked to a lesser degree, or did not crack during the first winter, have cracked subsequently. For these pavements, laboratory studies have shown that age-hardening of the asphalt is significant in lowering pavement resistance to low-temperature-induced cracking (15), as stated by Shahin and McCullough.

In all cases, pavement transverse cracking occurred in winter (low-temperature periods), which pointed to thermal shrinkage (16) as the main mechanism of transverse

Figure 14. Interval of recurrence and duration of annual lowest temperature cycle.



cracking in the pavements. Observations at the Ste. Anne Test Road have not yielded evidence to date of a long-term thermal-fatigue cracking mechanism as described by the authors. Increases in transverse cracking with pavement age have been attributed mainly to age-hardening of the asphalt binder coupled with the recurrence of low-temperature cycles.

INFLUENCE OF PAVEMENT STRUCTURE VARIABLES ON TRANSVERSE CRACKING FREQUENCY

The most significant factors influencing transverse cracking of the Ste. Anne Test Road pavements were asphalt type and grade (4). For the pavements that cracked, the frequency of transverse cracking was influenced by the subgrade type and the thickness of the asphalt concrete. For example, the 4-in. (100 mm) LV 150/200 penetration asphalt concrete pavement structure A (Table 1) with a heavy clay subgrade exhibited an average of 260 full-width cracks per mile (160 per km), and structure B (Table 1) with a sand subgrade had 660 cracks per mile (410 per km). However, the 10-in. (250 mm) full-depth asphalt concrete structure C on the clay subgrade had an average of 105 cracks per mile (65 per km), which is considerably less than the number of cracks in the 4-in. (100 mm) asphalt concrete structure A. These findings are significant in their effect on transverse cracking frequency and, as such, it would be more appropriate to compare them individually with the authors' predicted cracking frequency rather than collectively, as has been done (Table 1).

On the basis of a minimum observed spacing of 5 ft (1.5 m) between transverse cracks, Shahin and McCullough have assumed this to be the requirement of an unrestrained case for all pavements and have used this in predicting the probability of pavement failure. This is considered to be a generalization that could result in a large overestimate of distress for some pavements. For example, the LV 150/200 penetration asphalt pavements in structure A attained an average spacing between transverse cracks of 20 ft (6 m) the first winter after construction. This is considered approximately equal to the equilibrium crack spacing of a few hundred miles of old pavements constructed with a similar asphalt in clay subgrade areas. No further transverse cracking in these test road pavements has been observed since the first winter, and little additional transverse cracking is expected in the future.

The authors' damage model has the potential of serving as a useful aid in the selection of materials and designs to eliminate or minimize temperature-associated cracking of flexible pavements and to provide an estimate of related maintenance costs. However, as Shahin and McCullough have indicated, more inputs to the program are required.

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AUTHORS' CLOSURE

The authors appreciate Deme's interest in their work. Computer analysis of the Ste. Anne Test Road pavements indicated that the temperature cracking was due to low thermal shrinkage and thermal fatigue. What percentage of the temperature cracking was due to low temperature or thermal fatigue depended on the type of asphalt. Both of these mechanisms of temperature cracking are functions of the stiffness of the asphalt concrete, which is a function of the age-hardening of the asphalt. When the stiffness of the asphalt was estimated, age-hardening was accounted for through regression models. Our analysis indicated that pavement cracking occurred in winter; however, Deme's conclusion that the main mechanism is thermal shrinkage is not necessarily true, because cracking due to thermal fatigue also occurs in winter.

The calculation of thermal stresses for different pavement structures was not accounted for with the assumption that temperature cracking starts at the surface and that surface slab is fully restrained until the spacing of transverse cracks reaches 5 ft (1.5 m). The 5-ft (1.5 m) assumption was adopted from current literature; however, that does not mean that the spacing should reach 5 ft. More research is needed in the area of calculating thermal stresses for different pavement structures.

Finally, the authors agree with Deme that the most significant factors influencing transverse cracking are the asphalt type, grade, and age-hardening properties. Therefore, the results shown in Table 1 were reported for the different asphalt types.