Thermal Fracture Phenomena in Bituminous Surfaces

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This paper explores the phenomenon of thermally induced cracking in bituminous surfaces as a problem primarily associated with design. Low-temperature effects on pavement performance are examined and the need for modifying current mixture design procedures is discussed. Since any such design requirements would be concerned with fracture susceptibility, an approximate procedure for calculating thermally induced stresses is presented which recognizes the probable stiffness and temperature gradients that occur in field service.

A consideration of fundamental mechanisms involved is essential to adequately fulfilling design objectives; consequently, an hypothesis has been advanced to the effect that thermally induced cracking occurs in two main phases. These consist essentially of limited-depth crack initiation, and subsequent full-depth propagation with rising air temperatures. The pertinent mechanics are explained and the hypothesis appears to be compatible with observed phenomena.

Several practical implications of the hypothesis are discussed and these relate primarily to the development of mix designs to mitigate or reduce cracking and to the possible use of "good" and "poor" asphalts in the same pavement structure. It is suggested that more efficient and economical uses of available materials may be possible in designing for the low-temperature cracking problem.

•THE pavement design problem has received much attention during the past two decades and a considerable amount of information is available. Yet, we are in many ways little closer to understanding what this problem really is, either in a general sense or a specific case. Broadly, we recognize that the principal objective of a highway pavement is to provide for the safe, efficient and economical passage of road vehicles under all climatic conditions. However, we are in most cases unable to satisfy this objective as economically and efficiently as we desire, largely due to a state of imperfect technology. This involves an incomplete understanding of both the fundamental behavior and control of pavement component materials, or of their combined state in a pavement structure, and their contributions to the performance of the system as a whole under a variety of traffic, climatic and age conditions.

Hutchinson and Haas (1) have considered this situation in setting forth a systems analysis of the total highway pavement design process. Their analysis and recommendations are based on the premise that a completely rationalized process can only be developed through the use of techniques that permit the serviceability-age histories of alternate designs to be predicted along with the total cost streams necessary to produce these serviceability profiles. Additionally, their framework recognizes the need for a systematic identification of all performance variables and the placing of any one component's performance within the context of the system's performance as a whole. This requires an attendant advancement of the state of technology to the point where a component material, such as the bituminous mixture, can truly be "designed"; i.e., for any proposed pavement design, a bituminous material input for a specified performance requirement can be produced with predictable economic implications.

The intention of this paper is to explore the phenomenon of thermally induced fracture in bituminous surfaces with respect to:

1. The variables in low-temperature cracking,
2. Mixture design considerations for low-temperature conditions,
3. Calculating thermally induced stresses,
4. Defining the mechanisms of fracture initiation and propagation.

The major limitation of the paper is that the mechanisms are essentially postulated, although logically supported, and require experimental verification. Nevertheless, since the hypothesis may have some significant practical implications, it is considered useful to present it at this time in the hope that it will stimulate the necessary experimentation.

LOW-TEMPERATURE EFFECTS ON PAVEMENT PERFORMANCE

Advancing the state of technology to the point where a pavement component material can truly be designed implies a knowledge of the fundamental mechanisms that control in-service behavior. Only with this type of information can the design techniques mitigate any unsatisfactory effects of such behavior.

Low-temperature cracking is one of these unsatisfactory aspects of the overall behavior of flexible pavements. At the time of occurrence, it has relatively little effect on pavement riding quality. However, its implications, or "secondary deterioration" effects, such as bumps, dips, spalling, and potholes, can be highly detrimental to the performance and useful life of the pavement structure. The severity of this has been demonstrated by recent studies in Alberta (2), Saskatchewan (3, 4), Manitoba (5), Ontario (6, 7), Wyoming (8) and Connecticut (9). A result of these studies is that the primary mechanism of low-temperature cracking is receiving increased attention; some of this attention involves work on fundamental fracture phenomena (10). However, before we examine such fracture phenomena, several significant aspects must be appreciated; they include the following:

1. There can be a number of causes of fracture in bituminous mixtures; low-temperature shrinkage is only one of these.
2. The effects of asphalt aging on low-temperature physical properties of bituminous mixtures are very imperfectly understood.
3. Present practice in designing bituminous mixtures for physical behavior considers only higher temperatures (i.e., usually 140 F).

Low-Temperature Cracking

Pavements can deteriorate due to a variety of cracking types or other degradation under traffic and environmental factors. Low-temperature cracking itself of bituminous surfaces can also occur due to several causes that include:

1. Thermal stresses in the bituminous surface exceeding the tensile strength, without considering traffic loads;
2. Freezing shrinkage and cracking of the subgrade and propagation through the bituminous surface; and
3. Thermally induced stresses in the bituminous surface, coupled with traffic imposed stresses (i.e., cold pavement but unfrozen base and subgrade with "normal" deflection bowl under traffic), exceeding tensile strength.

Effects of Asphalt Aging

A considerable amount of effort has gone into studying in-service asphalt aging characteristics. Most of this work, however, has examined only the "average" effect throughout the depth of the pavement by extracting asphalt from core samples. Very little work has been done on "aging gradients" although it has been suspected for some years that the asphalt at the top of the layer can be significantly harder than at some depth, depending on how long the pavement has been in service. Kari and Santucci (11) have shown this phenomenon to exist on the basis of viscosity measurements but their work was primarily related to air void variations with depth on relatively new pavements. Recently, The Asphalt Institute has conducted some tests on the Virginia Test Road (12), and found, on the basis of fundamental viscosity at 140 F, that the extracted asphalt at the surface was up to ten times harder than at the bottom of the layer. As far as low-temperature behavior is concerned, the question of how such asphalt aging may affect cracking susceptibility has not yet apparently been investigated. It seems reasonable to assume though that it could be a significant factor in creating a stiffness gradient at the lower temperatures.

Low-Temperature Mix Design

Current methods of designing bituminous paving mixtures look for an end product that has a certain minimum stability at the maximum expected service temperature (usually 140 F), a deformation between certain limits at this temperature and certain air void and voids in mineral aggregate ranges. Additionally, we place various physical and chemical specification restrictions on the binder and aggregate components themselves. All these are supposed to bear some relationship to the in-service performance of the asphalt-aggregate system; yet, these criteria, most of which are high temperature oriented, are in reality little related to the range of environmental and loading conditions the material will be subjected to in its service life. One major reason has likely been the somewhat implicit assumption that high-temperature stability, deformation, skid resistance and durability were by far the most important considerations and that rapidly increasing stability with decreasing temperature negated the requirements for any design criteria in this area. Yet, the need for low-temperature design modifications in some regions was recognized by Rader (13) over 30 years ago when he stated, "Quantitative data obtained from low temperature tests should aid engineers in solving

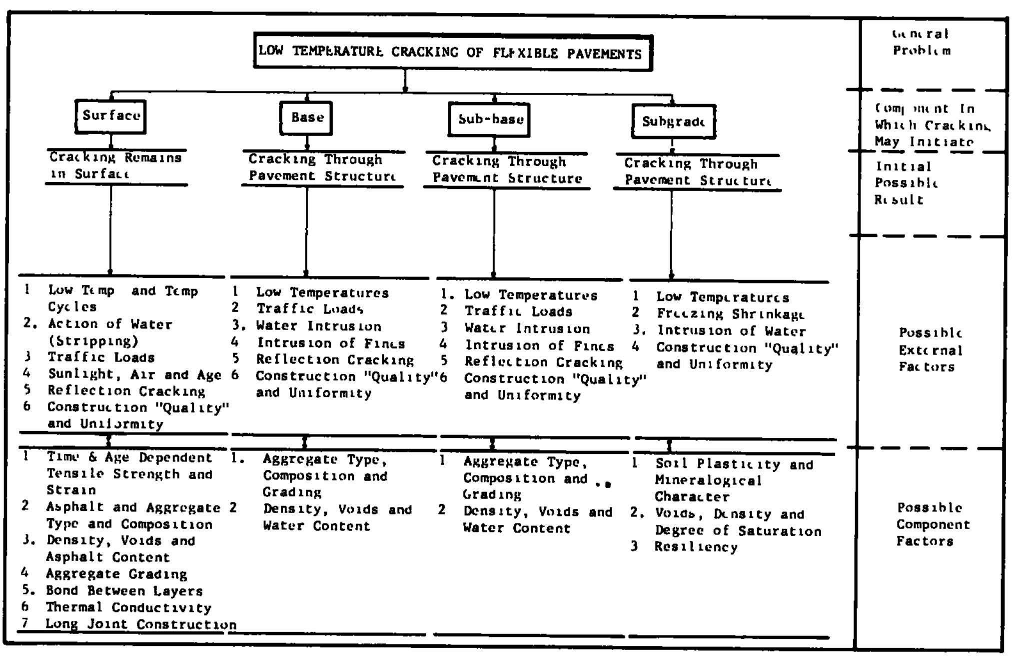

Figure 1. Factors of possible significance in low-temperature cracking of flexible pavements.

problems relating to the control of cracking." The appreciation of this need seems to have lain relatively dormant though until the markedly increased attention given to low-temperature cracking during the past five years. Consequently, except for some recent work in Alberta (14), design supplements or modifications for bituminous concretes in cold weather applications have not yet been developed.

Figure 1 summarizes the variety of factors that can be significant to low-temperature cracking. It illustrates that the variables can be external or related to the character of the component. The basic problem with respect to the bituminous materials is to identify and evaluate the component factors, in the light of the external factors, and to exercise a degree of control that maximizes the economic benefits accruing from reduced cracking.

THERMAL STRESSES IN BITUMINOUS SURFACES

Previous Efforts

Any comprehensive examination of thermal fracture phenomena in bituminous surfaces must necessarily consider the calculation of thermally induced stresses and a comparison with time and temperature dependent tensile strength of the mixture. Several attempts have been made to calculate such stresses and they include those by Monismith et al (15), Lamb et al (16), and Hills and Brien (17).

The first of these (15) utilized a stress equation developed by Humphreys and Martin (18) for an infinite slab composed of a linear viscoelastic material, subjected to a time-dependent temperature field. The equation provides for the calculation of the horizontal tensile stress field, as a function of depth, time and temperature. Temperature distribution was determined from a solution of the heat conduction equation, rather than from experimental field data. However, temperature distribution is merely an input to the stress equation; consequently, the method of determining it is of no concern insofar as solution of the stress equation is concerned. Relaxation moduli for inclusion in the equation were determined from the conversion of creep compliances from uniaxial creep test data.

The second stress calculation method (16) is much more simple in an analytical sense and considers the pavement surface as an elastic, infinitely long beam of finite width. For this case, where restraint is considered in two directions and no temperature gradient exists through the depth of the layer, the unit tensile stress in the longitudinal direction is given by:

$$\sigma_x(T) = E\alpha(\Delta T) + \mu f L$$

where

- E = Young's modulus;
- α = average coefficient of thermal contraction, over the temperature drop, ΔT;
- μ = Poisson's ratio for the material;
- f = coefficient of friction between the bituminous layer and the base; and
- L = one half the width of the pavement section.

Now, if E is replaced by a stiffness modulus, S, which varies with temperature and time of loading, a more accurate but still approximate value of longitudinal stress may be obtained. Lamb and his co-workers evaluated this modulus for the midpoint of their most extreme daily temperature drop range in the winter and obtained tensile stresses as high as 110 psi. The contribution of lateral restraint to this stress was found to be relatively minor.

The third method (17) has extended the concept of stiffness modulus over the entire temperature range and utilizes experimentally determined values of the modulus at the midpoint of small discrete temperature intervals. The equation for longitudinal stress, for the same infinitely long beam (neglecting lateral restraint) then becomes

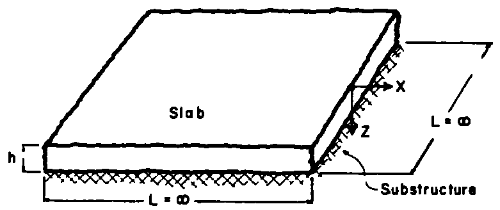

Figure 2. Infinite slab of viscoelastic material bonded to a rigid substructure.

$$\sigma_x(T) = \alpha \sum_{T = T_o}^{T = T_f} [S(\Delta T)]$$

where

S = stiffness modulus, determined experimentally, at midpoint of each ΔT; and
ΔT = temperature interval, of which a finite number are taken between $T = T_o$ and $T = T_f$.

Hills and Brien have used this equation to calculate thermally induced stresses in a beam of pure asphalt where the stiffness modulus was obtained at the midpoint of small temperature intervals, using a loading time corresponding to the cooling time for the temperature interval. They contend that any error involved in this use of corresponding times is likely to be small. The stress values obtained were then compared to tensile strengths and probable fracture temperatures determined. The actual temperatures for fracture, by direct measurement, were found to be in relatively good agreement, for a limited range of experimental work and no temperature gradients.

A Rigorous Approach to Determining Stresses

A rigorous analysis of thermally induced stresses can consider the material as viscoelastic in nature and this has been done by Humphreys and Martin (18) in considering an infinite flat slab bonded to a rigid substructure layer. This condition is shown in Figure 2 and the general field equations, in the absence of temperature dependence, used in their analysis may be listed as follows:

Strain Tensor:

$$\epsilon_{ij}(x_k, t) = \frac{1}{2}\left[\left(\frac{\partial}{\partial x_j}\right) u_i(x_k, t) + \left(\frac{\partial}{\partial x_i}\right) u_j(x_k, t)\right]$$

Equilibrium Condition:

$$\left(\frac{\partial}{\partial x_j}\right) \sigma_{ij}(x_k, t) = 0$$

Stress Strain:

$$s_{ij}(x_k, t) = \int_{-\infty}^{t} G_1(t - \tau)\left(\frac{\partial}{\partial \tau}\right) e_{ij}(x_k, \tau)\, d\tau$$

$$\sigma(x_k, t) = \int_{-\infty}^{t} G_2(t - \tau)\left(\frac{\partial}{\partial \tau}\right)\left[\epsilon(x_k, \tau) - 3\alpha\theta(x_k, \tau)\right] d\tau$$

where

ϵ_{ij} = strain tensor;
σ_{ij} = stress tensor;
u_i, u_j = displacements;
t = time;
s_{ij} = deviatoric components of the stress tensor;
G_1, G_2 = general relaxation moduli;
τ = a time variable for integration;
e_{ij} = deviatoric components of the strain tensor;

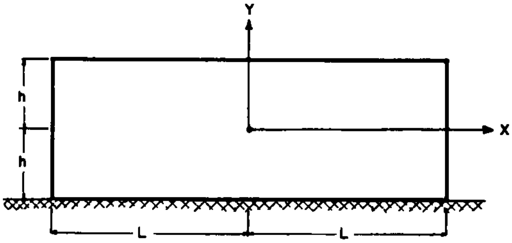

Figure 3. Finite plate of elastic material bonded to a rigid substructure.

σ = hydrostatic stress = σ_{kk};
ϵ = hydrostatic strain = ϵ_{kk};
α = coefficient of thermal expansion or contraction; and
θ = temperature change = $T(z, t) - T_0$.

The principle of time-temperature superposition has been used by Humphreys and Martin in obtaining the general constitutive equations which in turn permit the use of a "reduced time," ξ, in the last two equations. The application of a Laplace transform procedure to these modified equations, and the use of various stress and strain boundary conditions for an infinite slab in the first two equations, leads to the desired stress equation:

$$\sigma_{xx}(z, t) = -3\alpha_0 \int_0^t R\left[\xi(z, t) - \xi(z, \tau)\right]\left(\frac{\partial}{\partial\tau}\right)\theta(z, \tau)d\tau$$

where

$\sigma_{xx}(z, t)$ = horizontal stress in any direction, as a function of depth z, and time, t, for a slab of infinite extent in all horizontal directions (Fig. 2).

This stress equation was used by Monismith et al (15) in their analysis, which includes a definition of terms and an explanation of the numerical solution used. They point out, however, that the computed stresses, which are as high as about 3,300 psi for a particular case, may be somewhat greater than those which actually occur. This results from the assumption of infinite extent in the lateral direction, whereas the pavement may be more realistically simulated by an infinitely long strip. Consequently, although it is beyond the intention of this paper, there seems to be scope for exploring the case of such a strip, with its associated boundary conditions, using the general field equations quoted. The applicability of this type of approach could be checked in the laboratory with specimens of suitable geometry that are restrained in a manner simulating that of the prototype and subjected to a variable temperature field.

The determination of thermal stresses in restrained elastic plates has received attention from a number of authors, including a recent effort by Iyengar and Chandrasekhara (19). Their analysis assumed an isotropic and homogeneous material, uniform temperature change, no temperature variation with thickness, elastic constants independent of temperature, and a state of plane stress. The problem was formulated in terms of Airy's stress function. This involves the determination of a stress function, ϕ, which satisfies the following equation:

$$\frac{\delta^4\phi}{\delta x^4} + 2\frac{\delta^4\phi}{\delta x^2\,\delta y^2} + \frac{\delta^4\phi}{\delta y^4} = 0 \qquad (1)$$

where the x and y directions are as shown in Figure 3.

The stress component in the longitudinal direction, σ_x, which would be of prime interest for a pavement, is determined from

$$\sigma_x = \frac{\delta^2\phi}{\delta y^2} \qquad (2)$$

and the displacement component in the x direction (for the plane stress case) is determined from

$$u = \left(\frac{1+\nu}{2}\right)\frac{\delta\phi}{\delta x} + \frac{1}{E}\,\frac{\delta\psi}{\delta y} + \alpha$$

where

ν = Poisson's ratio for the material;
E = modulus of elasticity;
α = coefficient of thermal expansion or contraction;
ψ = a displacement function, defined as $\delta^2\psi/\delta x\,\delta y = \nabla^2\phi$ and $\nabla^2\psi = 0$.

Iyengar and Chandrashekhara (19) have considered the following expression for the stress function which satisfies the differential equation (Eq. 1), as well as the boundary conditions of zero longitudinal stress at the ends of the plate and zero normal stress at the free surface of the plate.

$$\begin{aligned}\phi = &\sum_{n=1,3}^{\infty} \frac{A_n \cos\beta_n x}{\beta_n{}^2 \sinh\beta_n h}\left[\beta_n y \sinh\beta_n y - \beta_n h \tanh\beta_n h \cosh\beta_n y\right]\\ &+ \sum_{n=1,3}^{\infty} \frac{B_n \cos\beta_n x}{\beta_n{}^2 \cosh\beta_n h}\left[\beta_n y \cosh\beta_n y - \beta_n h \coth\beta_n h \sinh\beta_n y\right]\\ &+ \sum_{s=1,3}^{\infty} \frac{C_s \sin\delta_s y}{\delta_s{}^2 \sinh\delta_s L}\left[\delta_s x \sinh\delta_s x - \delta_s L \tanh\delta_s L \cosh\delta_s x\right]\\ &+ \sum_{r=1}^{\infty} \frac{D_r \sin\gamma_r y}{\gamma_r{}^2 \sinh\gamma_r L}\left[\gamma_r x \sinh\gamma_r x - \gamma_r L \tanh\gamma_r L \cosh\gamma_r x\right]\end{aligned} \qquad (3)$$

where

A_n, B_n, C_s, D_r = Fourier constants;
$\beta_n = \frac{n\pi}{2L}$ where n = 1, 3, 5,...;
$\delta_s = \frac{s\pi}{2L}$ where s = 1, 3, 5,...;
$\gamma_r = \frac{r\pi}{h}$ where r = 1, 3, 5,....

Using boundary conditions for shear stresses and displacements appropriate to two particular cases, Iyengar and Chandrashekhara have shown how, after finite Fourier transformation and simplification, expressions can be determined for the four Fourier constants. Simultaneous solution of these expressions, using a finite number of terms for each series, makes it possible to obtain quite accurate values for the four unknowns. A numerical value for σ_x can then be found by differentiating Eq. 3, according to Eq. 2, and evaluating the result.

The foregoing analysis applies to uniform temperature through the plate; however, arbitrary temperature variation can be handled following the same procedure and using a general solution according to these same writers. Additionally, the solution is for the elastic case where the modulus, E, does not change with time. It may be feasible, however, to use a finite difference method by establishing E experimentally as a time and temperature dependent modulus for each small temperature decrement, similar to the previously mentioned approach of Hills and Brien (17). Then, the use of the more rigorous method of employing a stress function, for the elastic case, can be explored for varying boundary conditions and a finite length of pavement in terms of various L/h ratios (Fig. 3).

An Approximate Method for Determining Stresses

Stresses in an asphalt beam may be approximately determined, as previously pointed out, by the use of the equation for elastic stresses, with a time and temperature dependent stiffness modulus substituted for the modulus of elasticity. This modulus can be determined at the midpoint of small, discrete temperature intervals and the maximum tensile stress found by summating stress increments over the total temperature range. However, it is well known that due to the relatively low thermal conductivity of asphaltic concrete, significant temperature gradients usually exist through the depth of the surface layer. This has been well demonstrated by some recent work of The Asphalt Institute (20) and the Clarkson Institute of Technology (21), but very few such data exist on temperature variations in extreme cold weather areas. Nevertheless, it seems that more accurate determinations of temperature induced stresses and fracture susceptibility should definitely take the temperature gradient into account, as subsequently demonstrated in this paper. Additionally, the possibility of a "stiffness gradient," due to greater hardening of the asphalt binder at the surface (and exclusive of that created by the temperature gradient), may be taken into account as a further refinement.

The following listing of several possible "cases" considers these factors in extending the stiffness modulus concept to the determination of stresses at the top and the bottom of the bituminous layer. Intermediate stresses could be determined if the temperature and stiffness gradients were known through the depth of the pavement layer. Figure 4 shows a schematic representation of these cases for additional clarity. The stress calculations shown make the following simplifying assumptions:

1. The effect of lateral restraint on longitudinal stresses is omitted;
2. A linear temperature gradient prevails for certain initial intermediate or final conditions, which may be an oversimplification in some cases, as indicated by Straub's work (21); and
3. A linear stiffness gradient prevails, for certain conditions, which may or may not be reasonable since no data exist in this regard.

As well, to account for the time rate of temperature change, dT/dt, not being the same at the top and bottom of the layer in some cases, the summations of stress increments are shown over different total temperature intervals (although equal temperature decrements, ΔT, are used in all cases).

Case 1: The beam is subjected to a uniform temperature drop, with no temperature variation with depth, from T_o to T_f with a constant stiffness modulus through the depth at any one temperature. In this case, the maximum thermally induced, longitudinal stress, at any depth or at the surface is given by

$$\sigma_x(T) = \alpha \int_{T_o}^{T_f} S(r, T)\, dT$$

or

$$\sigma_x(T) = \alpha \sum_{T_o}^{T_f} S(\Delta T)\,[\Delta T] \quad \text{as an approximation}$$

where

α = average coefficient of thermal contraction over the temperature range;
$S(r, T)$ = temperature and time rate of temperature change dependent stiffness modulus;
$S(\Delta T)$ = stiffness modulus determined at the midpoint of ΔT and using a loading time corresponding to the time interval for the ΔT change;
ΔT = finite temperature interval between T_o and T_f;

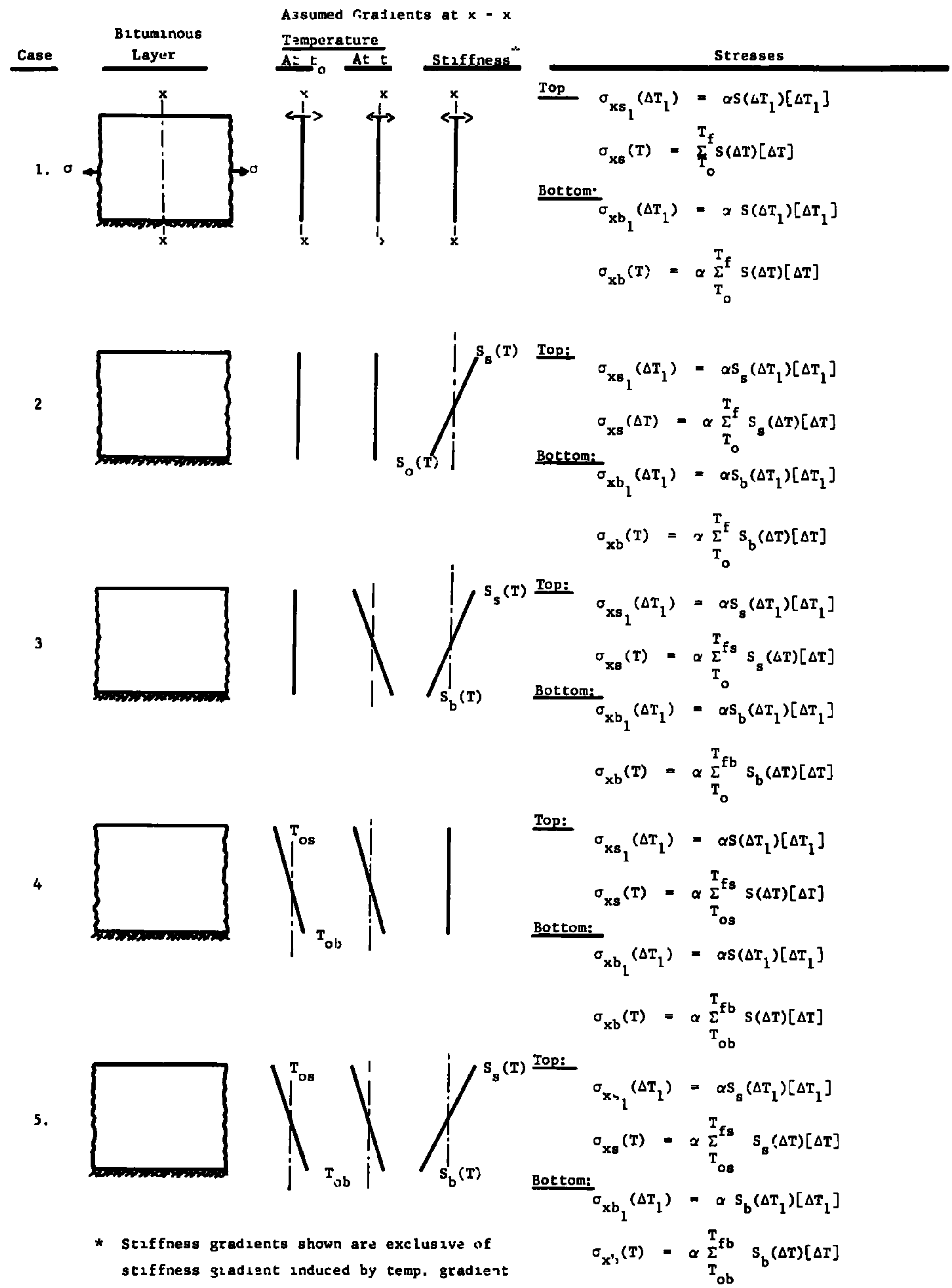

Figure 4. Approximate method for determining thermal stresses in a bituminous pavement surface.

( ) = as a function of; and
[ ] = multiplication.

Case 2: The beam is subjected to a uniform temperature drop, with no temperature variation with depth (as in Case 1), from T_o to T_f, with the stiffness modulus, $S(\Delta T)$, varying linearly through the depth (i.e., due to asphalt hardening). For this case, the maximum thermal stresses at the surface of the layer, $\sigma_{xs}(T)$, and at the bottom of the layer, $\sigma_{xb}(T)$, are given by

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_f} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_f} S_b(\Delta T)\,[\Delta T]$$

where

$S_s(\Delta T)$ = stiffness modulus at the surface of the layer at any temperature, T; and
$S_b(\Delta T)$ = stiffness modulus at the bottom of the layer at any temperature T.

Case 3: The beam initially has no temperature gradient but then is subjected to a temperature drop, with the rate of decrease, dT/dt, being greater at the surface than at the bottom, so that $T_o - T_{fs}$ represents the total temperature change at the surface and $T_o - T_{fb}$ the total temperature change at the base, during the total time interval. Additionally, the stiffness modulus varies linearly through the depth (as in Case 2). Then the maximum thermally induced stresses at the surface and bottom of the layer are given by, respectively

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_{fs}} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_{fb}} S_b(\Delta T)\,[\Delta T]$$

Case 3a: The situation is the same as in Case 3, except that there is no stiffness gradient. Here, the maximum thermally induced stresses at the top and bottom, respectively, are

$$\sigma_{xs}(T) = \alpha \sum_{T_o}^{T_{fs}} S(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_o}^{T_{fb}} S(\Delta T)\,[\Delta T]$$

Case 4: The beam has an initial, linear temperature gradient from T_{os} at the surface to T_{ob} at the base. It is then subjected to a temperature drop, with the rate of decrease, dT/dt, being greater at the surface than at the bottom, so that $T_{os} - T_{fs}$ represents the total temperature change at the surface and $T_{ob} - T_{fb}$ the total temperature change at the base. For this case, the maximum top and bottom thermally induced stresses, respectively, are

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S(\Delta T)\,[\Delta T]$$

Case 5: The situation is the same as Case 4, except that the stiffness modulus varies linearly with depth (as in Cases 2 and 3). Here the maximum thermally induced stresses at the top and bottom of the layer are given by, respectively,

$$\sigma_{xs}(T) = \alpha \sum_{T_{os}}^{T_{fs}} S_s(\Delta T)\,[\Delta T]$$

$$\sigma_{xb}(T) = \alpha \sum_{T_{ob}}^{T_{fb}} S_b(\Delta T)\,[\Delta T]$$

MECHANISMS OF LOW-TEMPERATURE CRACK INCEPTION AND PROPAGATION

Field Observations

The incidence of low-temperature cracking has been visually observed in many instances and some years ago, Baskin (22) noted: "[I]t was repeatedly noticed that cracking does not necessarily occur during the time of year when the pavement is at its lowest temperature. Rather is cracking most noticeable during the spring when pavement temperatures range all the way from 25 F to 45 F. Time and again we find pavements subjected to temperatures of -30 F or -40 F—and clear of snow—showing hardly any cracks." The same phenomenon has been observed by many others, as typified by one of the findings of the recent Saskatchewan study (3) which noted "transverse cracking became evident after prolonged cold temperatures followed by a sudden rise in temperature rather than after prolonged cold temperatures alone."

However, there has recently been some opinion, including that of the authors of this paper, that low-temperature cracks may actually occur as very fine or micro-cracks during the cold weather and that as warming occurs, these cracks open up and become visually apparent. The validity of this should be known when published results are available from several research projects that include the installation of electrical continuity strips for measuring the time of cracking (two known projects of this type are currently under way in Alberta and in Manitoba). Previously, it had been thought, as pointed out by Shields and Anderson (23), that several possible thermal reactions could occur to result in cracking:

1. Simple thermal contraction of the surface;
2. Base course restraint to contraction of the surface;
3. Sudden warming and subsequent weakening of a highly stressed surface;
4. Shrinkage cracking of the subgrade and subsequent reflection cracking through to the surface layer.

It may be noted that all these reactions seem to be consistent with the observed phenomena; however, some inconsistencies appear in relation to the hypothesis of micro-cracking at cold temperatures. These will be subsequently considered in more detail since an adequate understanding of the pertinent mechanisms involved is essential to the eventual development of control or predictive techniques relating to fracture susceptibility.

A Consideration of the Cracking Mechanism

Cracking of a bituminous surface layer will occur when the induced stresses, either externally applied or internally developed, exceed the tensile strength of the material. The externally applied stresses can occur either due to traffic or to "drag" from subgrade cracking while the internally developed stresses are thought to be primarily associated with temperature changes.

It has been thought by a number of people that the visual observation of cracks at higher temperatures, following cold weather, meant that the warm weather weakened the highly stressed bituminous layer. The consequence was thought to be an insufficient tensile strength in the material to withstand the thermally induced stress from the lower temperature. This is supported by an actual observation by Huculak (24), who reported that a pavement cracked audibly beneath his feet during the sudden warming of a chinook in Banff, Alberta. However, examination of this hypothesis in the light of what is physically possible and what has been observed, reveals several contradictions. Firstly, the hypothesis assumes that the cold base course is restraining the surface course, to maintain a high stress condition, while the surface of the layer suddenly warms and thereby is insufficiently strong to resist this imposed drag. But if the subgrade and base do not crack, they obviously have not contracted in a longitudinal direction and consequently are unable to impose drag stresses on the bituminous layer. Secondly, it seems that relaxation mechanisms may be able to act sufficiently fast to relieve any such imposed stresses, especially at the 40 F to 50 F temperatures that can occur in western Canada under rapid chinook wind warming. To check this second aspect, some recent testing of asphalt-concrete beams at the University of Waterloo has shown very high thermally induced stresses in a completely restrained beam. Subsequent very rapid warming of such specimens, while still completely restrained, resulted in no cracking and rapid relaxation of stresses. These results (Fig. 5) indicate that stresses can be relieved at a warming rate much exceeding that likely in field service. These experiments are admittedly limited in scope insofar as asphalt aging, asphalt type, aggregate influence, and certain other factors are concerned.

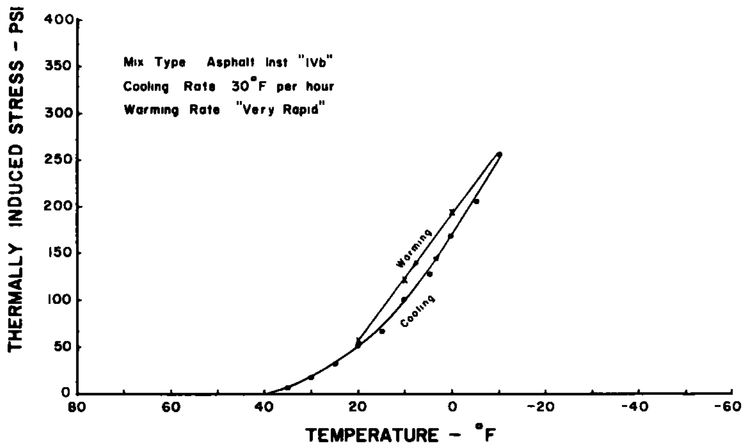

Figure 5. Thermally induced stress in a restrained asphalt-concrete beam.

Nevertheless, they tend not to support the postulated mechanism of cracking due to restraint and warming.

An Alternate Thermal Cracking Hypothesis

Any criticisms of postulated mechanisms, or advancement of new theories, must take into account the much observed fact that thermally associated cracks seem to appear primarily during the warmer weather. In other words, the cracks open up enough to be easily seen. Now if one does not accept the restraint cracking hypothesis (providing the cracking is not associated with traffic or subgrade cracks), then one might examine the fine cracking or micro-cracking postulation. This would contend that cracking does in fact occur during the cold weather, that these cracks are nearly invisible to the naked eye and that subsequent warming opens the crack. Yet, consideration of the properties of materials and the mechanics of the situation reveals a definite inconsistency. Simply, this is that if such a micro-crack did form in the bituminous layer, subsequent warming would, because of expansion, tend to close the crack rather than to open it. As well, if the crack occurred to full depth during the cold weather, it should tend to be V-shaped with a clearly visible opening at the top.

Thus, if we wish to accept the micro-cracking hypothesis, and if we accept the field observations of crack opening on warming, then we are required to formulate a new hypothesis consistent with observed phenomena. Figures 6 and 7 are schematic representations of two such possible mechanisms. They describe crack initiation and propagation in a finite series of steps, for ease of explanation, and also show the likely temperature and stress distribution patterns. The figures are fairly self-explanatory and do not require detailed discussion to illustrate the hypothesis involved. Basically, each mechanism postulates that the thermally induced crack initiates at the surface and that it penetrates to only a limited depth. Mechanism A (Fig. 6) theorizes that the crack is propagated through a stress imbalance created by warming at the surface while the remainder of the layer stays relatively cool. Then, after full-depth propagation, while the layer is still relatively cool, the crack appears visibly opened. As warming eventually spreads through the full depth of the layer (i.e., late spring or summer), the crack again closes because of expansion. This explanation of crack propagation does not seem inconsistent with Huculak's observation (24) in that he may have been witness to an extreme case of the postulated mechanism A.

Mechanism B (Fig. 7) postulates that the crack is propagated through a stress imbalance created by several cycles of day warming and night cooling, as one might expect in the spring. Then, as for the first case, the crack appears visibly opened at full-depth propagation and while the layer is still relatively cool, and closes when full-depth warming occurs.

The significant feature of both these postulated mechanisms is that the initial crack occurs to only a limited depth. This initial penetration may be very small, depending upon the temperature and stiffness gradients through the surface layer. In any case, it should be possible to check experimentally the validity of the hypothesis either in the field or in the laboratory.

Some Practical Implications

Anderson and Hahn (14) have approached the problem of thermal cracking from the point of laboratory evaluation of mixture designs, and quite logically they conclude: "It is expected that failure strain be considered as another test value to be determined, much the same as stability, flow, air voids, etc., are now obtained, prior to establishing the recommended combination of aggregate and asphalt, i.e., selection of mix design." This philosophy fits quite well with the crack initiation hypothesis advanced in that if such cracking does indeed occur to a limited depth during the cold weather, it may be possible to design the surface layer to avoid entirely or to at least reduce the cracking. The binder or leveling course, or the lower portion of a deep-strength bituminous pavement, may not require such modifications if, through the appropriate design modifications, cracking is prevented at the surface.

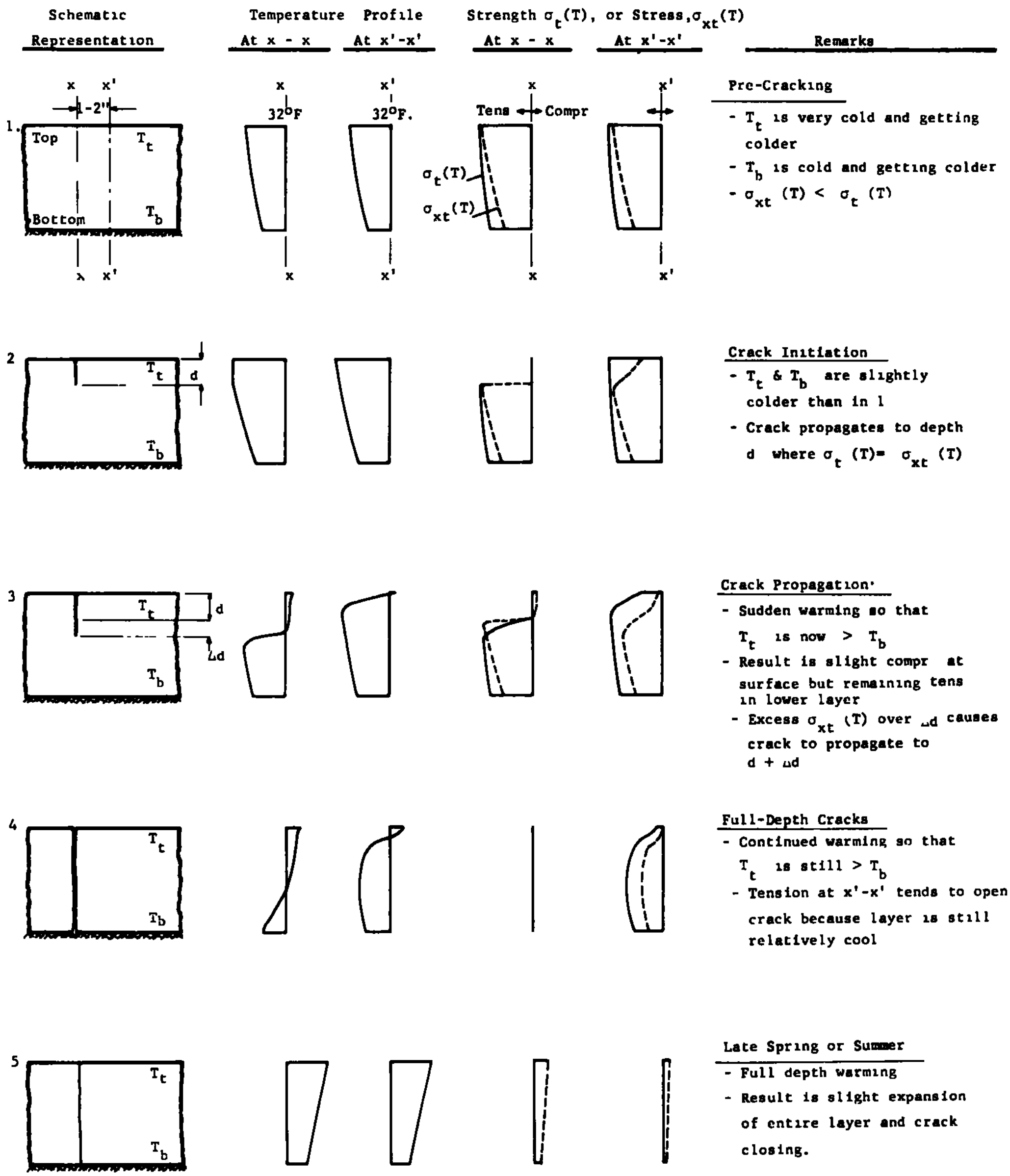

Figure 6. Possible mechanism A for low-temperature crack initiation and propagation in a bituminous pavement surface.

The first step required in such design is to evaluate the expected thermal conditions for winter conditions. It may be feasible to handle expected extreme low temperatures, for design purposes, on a return period or recurrence interval basis similar to that employed in hydrologic applications. Data from such studies as that conducted at the Clarkson Institute of Technology (21) can be most significant in providing the required temperature information. This information can then be used, with the appropriate laboratory data on tensile stress-strain-time-temperature-age characteristics, in calculating an expected thermally induced stress field in the bituminous layer for the design return period. Comparison with failure characteristics of the mixture can then be used to determine probable fracture.

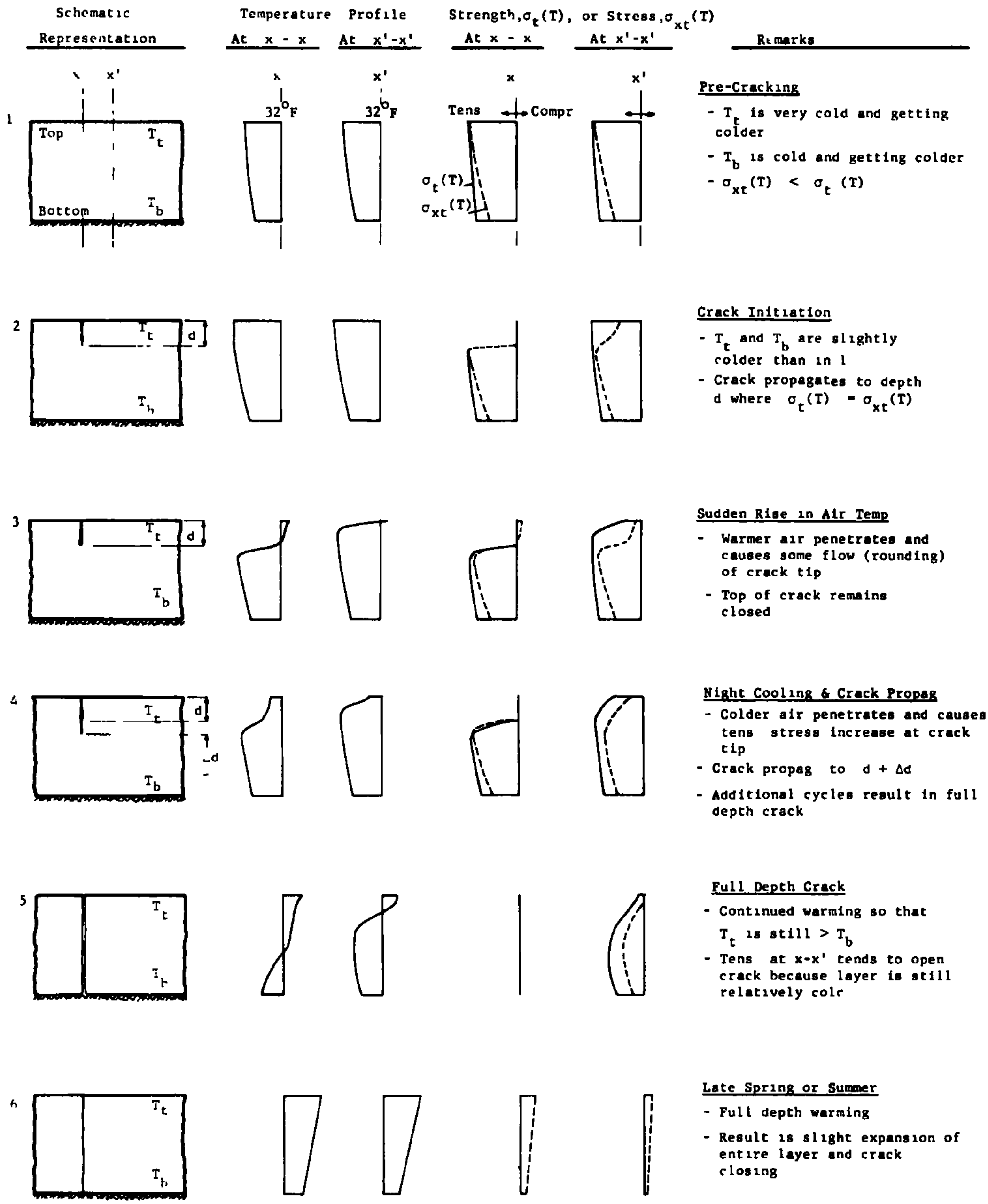

Figure 7. Possible mechanism B for low-temperature crack initiation and propagation in a bituminous pavement surface.

Another extremely important implication of the hypothesis advanced concerns the source of asphalt supply. Evidence is now being accumulated (2, 3, 14) that low-temperature transverse cracking frequency can be markedly affected by the asphalt, although it should be noted that McLeod (25) contends softer asphalt cements can reduce transverse cracking, with source apparently being a minor consideration. Nevertheless, there is a widespread opinion in Canada that we have "poor" and "good" asphalts, with regard to a variety of desirable properties including the one of reduced transverse cracking. (The terms "poor" and "good" may be far from satisfactory but find widespread usage in the industry in lieu of the availability of more objective descriptions.) Therefore, until such time as the state of technology has advanced to the point where refinery processing or additive addition can mitigate such problems as cracking, whatever the asphalt source, it may be wise to examine the potential role of the better asphalts. Since there is also considerable opinion that the proven supplies of good asphalts in Canada are limited, perhaps we are collectively guilty of some misuse. It is possible that the use of a superior asphalt for either the surface course or the top portion of a deep strength layer, where practical, may avoid low-temperature transverse cracking, providing the hypothesis of limited depth initial cracking is valid. This assumes, of course, that the temperature and stiffness gradients through the layer are such that the underlying mixtures, with the lower quality asphalts, are not overstressed.

It is, of course, realized that this implies the explicit recognition of premium grade asphalts, a practice which seems to have been largely avoided in Canada in a formal sense. Nevertheless, there are some agencies who acomplish this in a de facto sense, with their specifications. Yet, it seems that we may be wiser to extend such explicit recognition, through the appropriate specifications based on realistic physical tests, with much the same manner of reasoning that has traditionally led us to use higher quality aggregates in the higher portions of the total pavement structure. Such practices could tend to conserve the supplies of higher quality asphalts or to use them in a more efficient manner.

While the foregoing suggestions are based on limited evidence, they are considered to have sufficient promise to encourage the development of experimental programs concerned with the efficacy of using good and poor asphalts, in certain cases, in the same pavement structure. The potential payoff could be a more efficient and economical use of available materials as well as the economic benefits of reduced thermal cracking. In an attempt to answer some of these questions and in view of this potential payoff, the University of Waterloo has under way a sponsored research program in the areas of flexible pavement performance at low temperatures and in systems analysis of pavement design processes.

CONCLUSIONS AND RECOMMENDATIONS

The major conclusions arising from this paper and some pertinent suggestions may be summarized as follows:

1. An hypothesis has been advanced to the effect that low-temperature cracking of bituminous surfaces occurs in two main phases: these consist essentially of limited depth crack initiation, and subsequent full-depth propagation with warmer air and surface temperatures. Diagrams are presented to explain the pertinent mechanics and it is shown that this crack mechanism hypothesis is not inconsistent with observed phenomena. As well, certain inconsistencies between observed phenomena and previously advanced postulations are explored.
2. Low-temperature effects on pavement performance are discussed and the need for modifying current bituminous mixture design procedures to account for low-temperature service conditions is pointed out. The need for investigating the possible effects of varying degrees of asphalt aging with depth is also noted.
3. A procedure is presented for calculating thermally induced stresses in bituminous surfaces. It is approximate and can utilize stiffness moduli from experimental evaluation of the materials. The procedure recognizes the stiffness and temperature gradients that are likely present in field service and it explores a variety of possible situations. The validity of this approach has recently been demonstrated (26).
4. Certain practical implications of the cracking hypothesis advanced in this paper are discussed. These relate to reduced cracking through mix designs and through the use of good and poor asphalts in the same pavement structure. The application of such practices, which of course depends upon verification of the hypothesis, has the potential of greater efficiency or economy in the use of existing materials and in the economic

benefits of reduced cracking. Experimental programs to explore these implications seem warranted.

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