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Publication of this paper sponsored by Committee on Rigid Pavements.

Analytic Approach to Concrete Pavement Blowups

ARNOLD D. KERR AND PATRICK J. SHADE

The results of analyses of concrete pavement blowups are presented and discussed. The analyses are based on the assumption that blowups are caused by lift-off buckling of the pavement due to a rise in pavement temperature and moisture. A safe temperature and moisture increase is defined, and the way in which it depends on various parameters, such as pavement thickness, axial shearing resistance along the pavement-soil interface, and the thermal expansion coefficient, is shown. Also shown are the ways in which blowups may be affected by pavement curing temperature, resurfacing layers, and the reduction of pavement stiffness caused by heavy wheel loads and the age of the pavement. The results of the study should contribute to a better understanding of the mechanism of pavement blowups and the determination of the essential parameters. It also provides guidelines for prescribing measures to reduce or totally eliminate blowups in concrete pavements.

Blowups of concrete pavements have been a problem for highway and airport engineers for many years. As early as 1925, the problem was discussed in the Engineering News Record (1). A severe highway blowup that occurred in 1975 in Ohio (2) is shown in Figure 1.

There is general agreement that blowups are caused by axial compression forces induced in the pavement by a rise in temperature and moisture and that they usually occur at joints or cracks. Many highway engineers are of the opinion that a major cause of blowups is infiltration of debris into joints or cracks (3). However, blowups of continuously reinforced concrete pavements (CRCPs) without joints have also been observed (4, p. 52).

In the past few decades, many reports have been published on pavement blowups in the United States. A critical review of blowup studies by Yoder and Foxworthy (3), published in 1972, reveals many inconclusive findings. The status of the research on blowups was summarized by Gress (5,6) in 1976: "To date, work in this area has been qualitative and empirical and has not resulted in an understanding of the blowup mechanism." According to a 1978 report from England by Andrews (7), "the precise mechanism of blowups has not been established." It appears that the rather extensive research effort on blowups of concrete pavements conducted over the past decades did not lead to a solution of the problem because of the lack of a generally accepted theory that would establish the important parameters that affect pavement blowups.

Recently, Kerr and Dallis (8) and Kerr and Shade (9) developed analyses for the blowup of concrete pavements. The essential results of these studies are presented in this paper. The analytic details are presented elsewhere (8,9). In this paper, emphasis is placed on the assumed pavement blowup mechanism, the results obtained (presented as graphs), and the correlation of the pavement parameters that were used in these analyses with various factors that, to some investigators, appeared to affect the occurrence of blowups, as described in the literature (2,3).

BLOWUP MECHANISM AND ANALYTIC RESULTS

It is assumed that blowups are caused by lift-off buckling of a concrete pavement due to compression forces induced in the pavement by a rise in tempera-

Figure 1. Blowup of concrete highway pavement in Ohio.

Figure 2. Types of buckling in concrete pavements.

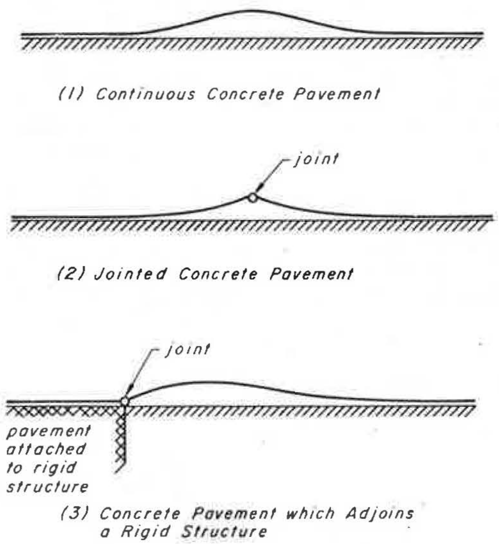
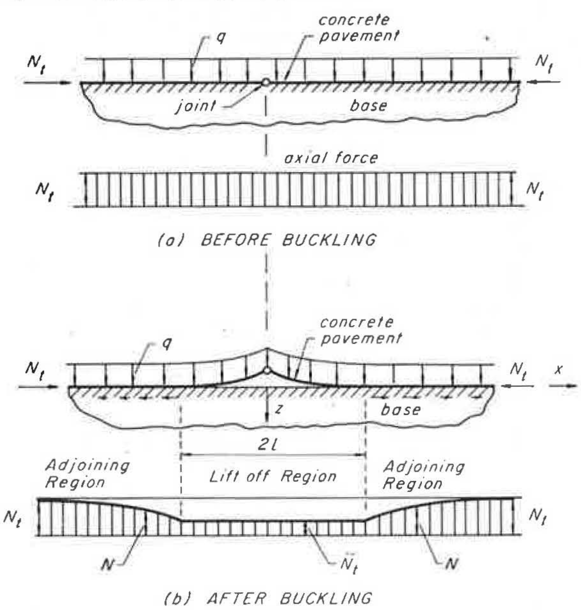


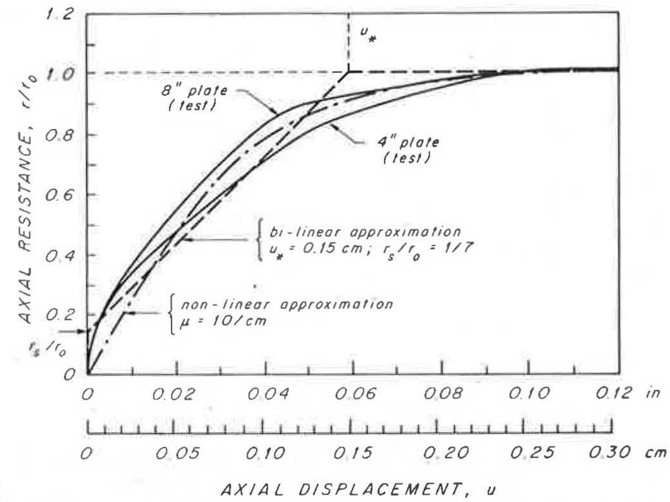
Figure 3. Axial forces in concrete pavements.



ture and moisture. The magnitude of the temperature and moisture increase that causes blowups also depends, among a variety of factors, on whether the pavement is continuous or jointed. Typical anticipated blowup modes are shown in Figure 2. Because an inextensible joint, such as a transverse crack, weakens the concrete pavement, jointed pavements will generally buckle at a lower temperature and moisture increase than continuous pavements. Spalling of the concrete at the joints (or transverse cracks) reduces joint stiffness. Expansion joints reduce the axial force in the vicinity of the joint. However, intrusions into such a joint hinder the slabs from expanding and cause higher compression forces and force eccentricities that may lead to spalling or buckling of the pavement at the expansion joint.

For the past decade Kerr (10,11) studied thermal buckling of railroad tracks, a closely related problem. The analyses by Kerr and Dallis (8) and Kerr

Figure 4. Resistance-displacement response.



and Shade (9) that are used in this paper are based on the methodology developed by Kerr (11).

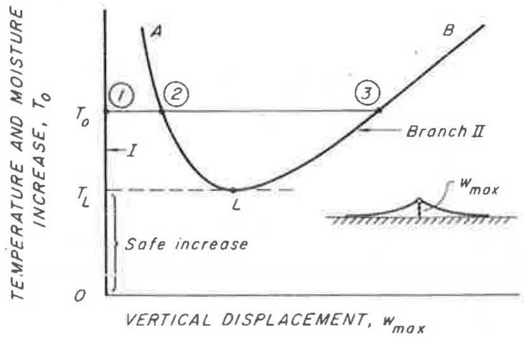
To describe the analytic model used, consider a long concrete pavement with a joint (hinge) (case 2 in Figure 2). Due to constrainted expansions, a temperature and moisture increase in a straight pavement induces an axial compression force (N_t), as shown in Figure 3a. For sufficiently large values of N_t, the pavement may buckle out vertically. Then, in the lift-off region of length 2l, some of the constrained expansions are released. This results in a reduction of the axial orce in the lift-off region to $\tilde{N}_t$. In the adjoining regions, the vertical displacements are very small. In these regions, due to the resistance to axial displacements at the pavement-base interface, the constrained axial expansions vary; so does the axial force $\tilde{N}_t < N < N_t$, as shown in Figure 3b.

In the analyses, the concrete pavement was replaced by an equivalent beam of rectangular cross section. Because the ratio of pavement width to thickness (b/h) is greater than 20, the bending stiffness (EI) was increased to $EI/(1 - \nu^2)$ to account for plate action.

It was assumed that the pavement is subjected to a uniform temperature and moisture change T_o above neutral and a uniformly distributed pavement weight q per unit length of axis. [A moisture increase (or drop) in the concrete slab can be expressed by an equivalent temperature increase (or drop). The neutral temperature is defined as the temperature at which the axial force in the pavement is equal to zero. For new pavements the neutral temperature is near the pavement temperature at which the concrete solidified, forming the pavement.] Furthermore, it was assumed that prior to and during buckling the response of the concrete pavement is elastic.

Graphs of axial resistance due to axial displacements at the pavement-base interface are shown in Figure 4. The test results (12,13) are shown as solid curves. In the analysis by Kerr and Dallis (8), this nonlinear response was described by the bilinear approximation shown in Figure 4 by dashed lines. Kerr and Shade (9) represented this response by the nonlinear relation $r = r_o \, tgh \, [\mu u(x)]$, shown in Figure 4 as a dash-dot-dash curve, where r_o is the sliding frictional resistance and μ is another parameter for fitting the analytic expression with the test data. Although the response shown in Figure 4 is nonelastic, the assumed representations are jus-

Figure 5. Equilibrium branches of pavement.



tified because during buckling the axial displacements monotonically increase.

An important feature of the formulations obtained by Kerr and Dallis (8) and Kerr and Shade (9) is that, although the resulting differential equations are nonlinear (geometric nonlinearity in the lift-off region and material nonlinearity in the adjacent regions), it was possible to solve them exactly and in closed form. The respective solutions yield the postbuckling displacements and the axial forces in the pavement before and after buckling.

Typical equilibrium branches obtained from the analyses are shown in Figure 5. Each point on the shown equilibrium branches corresponds to an equilibrium configuration of the pavement: branch I corresponds to the straight, unbuckled equilibrium states and branch II to the vertically deformed equilibrium configurations.

From the nature of the postbuckling equilibrium branches and their stability, it follows that the safe range of temperature and moisture increases to prevent pavement buckling can be determined solely from the postbuckling equilibrium branch. This range is $0 < T_o < T_L$, where T_L is the temperature increase that corresponds to the horizontal tangent of branch II. This concept is adopted in this paper and in the reports by Kerr and Dallis (8) by and Kerr and Shade (9).

The numerical evaluations of the derived solutions were performed for a pavement of constant rectangular cross section of width b = 725 cm (23.8 ft) and pavement thicknesses h = 20, 25, and 30 cm (8, 10, and 12 in.). The chosen pavement parameters are as follows: effective Young's modulus (E) = 3,000 kN/cm² (4.35 x 10⁶ lb/in.²), Poisson's ratio (ν) = 0.3, the coefficient of linear thermal expansion (α) = 0.9×10^{-5}/°C (0.5×10^{-5}/°F), and the specific weight of pavement material (γ) = 23.6 kN/m³ (150 lb/ft³). Because the reinforcement ratio in the pavements is generally very small (0.5 to 0.75 percent) and it is usually placed near the centroid of the cross section, the effect of the reinforcing bars was neglected in calculating $EI/(1 - \nu^2)$, the effective flexural stiffness of the pavement (14).

The shearing resistance values (r_o) at the pavement-base interface (as defined in Figure 4) per unit length of pavement, with b = 725 cm, determined from the test data of Teller and Sutherland (12, Figure 21) are as follows:

h (cm)	r_o (kN/cm)
20	0.77
25	0.86
30	0.91

Also based on the test data presented by Teller and Sutherland (12, Figure 20), it was assumed that $r_o/r_s = 7$ and $u_* = 0.15$ cm (0.07 in.). These values were needed for the bilinear approximation of axial resistance used by Kerr and Dallis (8).

For the nonlinear approximation of axial resistance, $r = r_o \operatorname{tgh}[\mu u(x)]$, used by Kerr and Shade (9), r_o is the same as shown above. To approximate closely the test data in (12,13), it was assumed that μ = 10/cm (25.4/in.) for all three pavement thicknesses.

The solutions for the pavement with a joint of zero stiffness (hinge) obtained by Kerr and Dallis (8) and Kerr and Shade (9) were evaluated numerically for the foregoing parameters. The results are shown in Figure 6. Within the accuracy of the graphs

Figure 6. Equilibrium branches for pavements with joint (hinge).

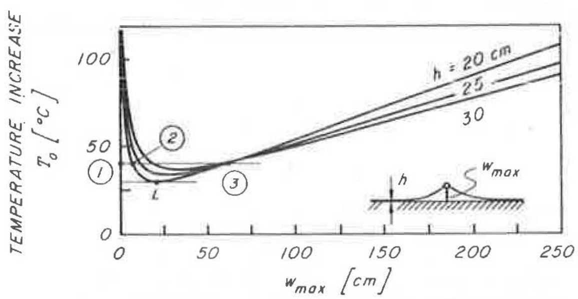


Figure 7. Equilibrium branches for continuous pavements.

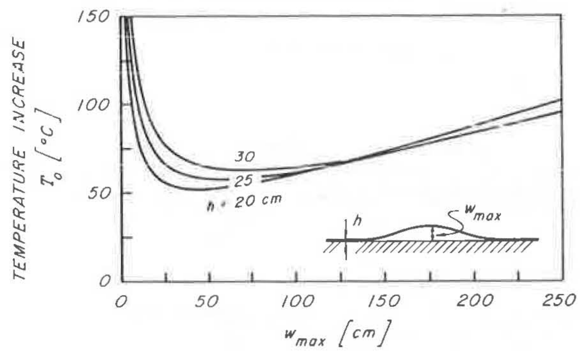
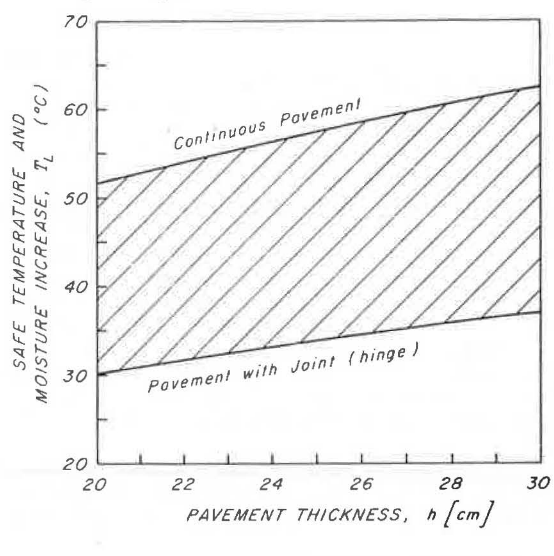


Figure 8. Safe temperature and moisture increase versus pavement thickness: effect of joint (hinge).



shown, they essentially coalesce, which indicates that either the bilinear or the nonlinear approximations used for axial resistance are sufficient for engineering purposes.

The corresponding results for continuous pavements (without joints) are shown in Figure 7. The shape of the postbuckling equilibrium branches is similar to the ones for the jointed pavements. However, the range of the safe temperature increases is larger, as anticipated. From Figure 7 it follows that a CRCP may buckle up. Thus, these pavements are also susceptible to blowups, as reported by Nussbaum and Lokken (4).

To show the effect of the transverse joint (hinge) and pavement thickness on the safe temperature increase, the T_L values for the continuous pavement and the jointed pavement are shown in Figure 8. Because a real joint (or transverse crack) will have some bending stiffness, the corresponding T_L value will be located between these two curves. If contraction of an expansion joint is possible, the corresponding T_L value will also be located above the lower curve. Thus, the curve for the pavements with a joint (hinge) is the lower bound for the T_L values. The curve for the continuous pavements is the upper bound when contraction of an expansion joint is not possible (for example, a contraction caused by intrusions).

DISCUSSION OF RESULTS

To demonstrate the usefulness of the graphs obtained, consider as an example a jointed pavement with h = 20 cm (8 in.) and b = 725 cm (23.8 ft). For the pavement parameters used, according to Figure 6, the safe temperature increase to prevent buckling by lift-off is $T_O = T_L \cong 30°C$ (86°F). This means that, for a temperature increase of less than 30°C above neutral, the pavement is safe against buckling by lift-off. For a temperature increase of T_O = 40°C (72°F) above neutral, there exist three equilibrium configurations and, when buckling does occur, the pavement moves to the stable equilibrium configurations (circled number 3 in Figure 6).

The magnitude of the lift-off displacements at the joint w_{max} depends, according to Figure 6, on the temperature increase T_O at which buckling takes place. For example, for h = 20 cm, if the pavement buckles at an increase of $T_O = T_L \cong 30°C$, w_{max} = 20 cm; however, when it buckles at an increase of t_O = 40°C, the vertical uplift at the joint is w_{max} = 65 cm (26 in.). Thus, a temperature increase of a third more than triples the value of w_{max}.

When the pavement thickness is increased by 50 percent to h = 30 cm (12 in.), the corresponding safe temperature increase, according to Figure 6, is T_L = 36°C (97°F). This increase is only 12 percent higher than for h = 20 cm. It should be noted that, when pavement thickness is increased, bending stiffness and axial shearing resistance between pavement and base increase but so does axial force (N_t) for a given temperature increase.

For a CRCP with h = 20 cm and b = 725 cm, the safe temperature increase above neutral is T_L = 50°C (90°F). Thus, for a straight pavement that was cast during the early spring (or late fall) with a neutral pavement temperature of 10°C (50°F), buckling by lift-off will not take place if the pavement temperature is lower than 10°C + 50°C = 60°C (140°F), a high pavement temperature usually not encountered in the field. This appears to be the reason why few blowups are reported for CRCPs.

It should be noted, however, that when a pavement with a dark upper surface is exposed to the sun the resulting pavement temperature may be substantially higher than the ambient temperature and may exceed the safe temperature increase (T_L). This should be taken into consideration, especially when overlays are chosen for the resurfacing of concrete pavements.

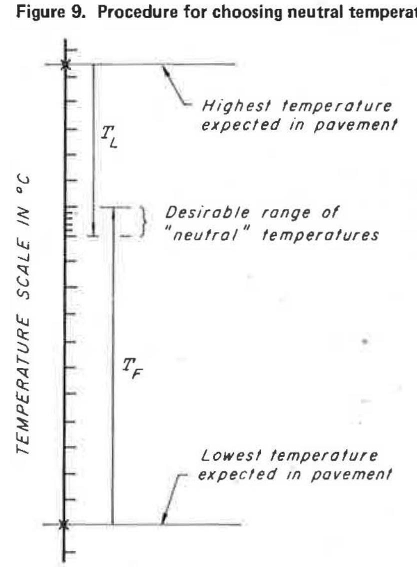
The preceding example also demonstrates the effect of the neutral temperature on pavement blowups. For example, if the pavement is cast during a hot summer day and the resulting neutral temperature is 30°C (86°F) instead of 10°C, as noted, buckling by lift-off will not take place if the pavement temperature is lower than 30°C + 50°C = 80°C (176°F). Although this higher neutral temperature may prevent blowups, it will lead to high axial tensile forces in the pavement during the winter, which may cause pavement ruptures. Thus, the construction season affects blowups as well as the formation of transverse cracks.

Ideally, one should determine, for a given location, a neutral pavement temperature that will prevent pavement blowups or ruptures. In this connection, it may be of interest to note that in railroad engineering, when continuously welded rails are installed, they are heated or cooled to a prescribed neutral temperature before they are anchored to the embedded ties in order to prevent problems of this type (15). A method for determining the desired neutral temperature is shown in Figure 9.

Next, the effect of a reduction in pavement bending stiffness $[EI/(1 - \nu^2)]$ on blowups is established. The effective bending stiffness of weakly reinforced concrete pavements may be reduced substantially by fine cracks caused by shrinkage, tensile forces due to temperature drops, heavy wheel loads, and so on. It is reasonable to expect that this reduction will increase with the age of the pavement, increased wheel loads, and greater traffic density because of concrete fatigue and the accumulation of cracks. To determine this effect, the solutions in the reports by Kerr and Dallis (8) and Kerr and Shade (9) were evaluated for various values of $I = bh^3/12$ without changing the other parameters. The results are shown in Figure 10. Note the drop in T_L caused by the reduction of I. For example, for a continuous pavement with h = 20 cm, when the bending stiffness is reduced by 50 percent, T_L decreases by about 20 percent.

It is also of interest to establish the effect of shearing resistance at the pavement-soil interface on pavement blowups. This was achieved by evaluating the solutions in the work of Kerr and Dallis (8)

Figure 9. Procedure for choosing neutral temperature.



and Kerr and Shade (9) for different values of sliding frictional resistance (r_o) without changing the other parameters. The results are shown in Figure 11. Note the strong drop in T_L for decreasing r_o. This finding should be of interest in considering the effect of the type of drainage and subgrade over which the pavement was laid.

Many pavement engineers are of the opinion that the shearing resistance at the interface contributes to the formation of transverse cracks and thus should be minimized--for example, by placing plastic sheets at the interface. In this connection, it should be noted that a reduction of this resistance also reduces T_L and thus has an adverse effect on pavement blowups. To prevent blowups, the sliding frictional resistance should be as high as possible.

To show the effect of the coefficient of linear thermal expansion (α) on pavement blowups, the obtained solutions were evaluated for a range of α values. The results are shown in Figure 12. Note the drop of T_L with increasing α. This finding should be taken into consideration when the cement and the coarse aggregate for the pavement concrete are chosen.

Figure 10. Effect of pavement bending stiffness on safe temperature increase.

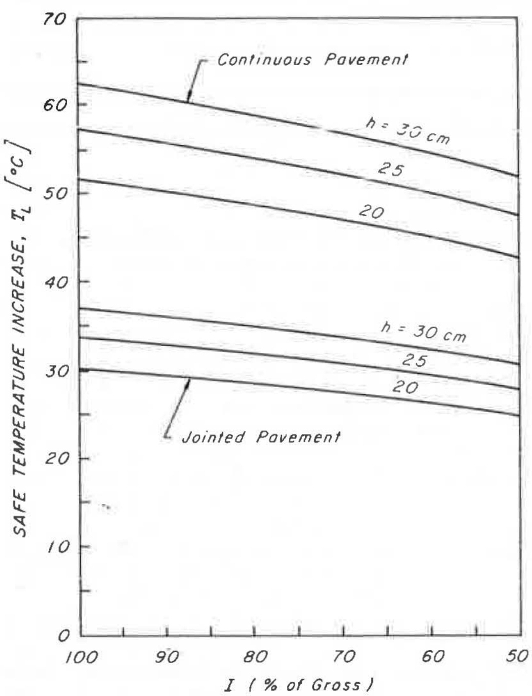
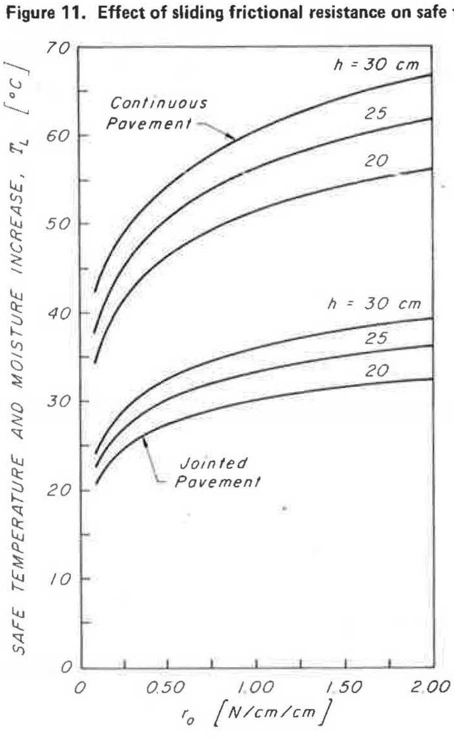


Figure 11. Effect of sliding frictional resistance on safe temperature increase.

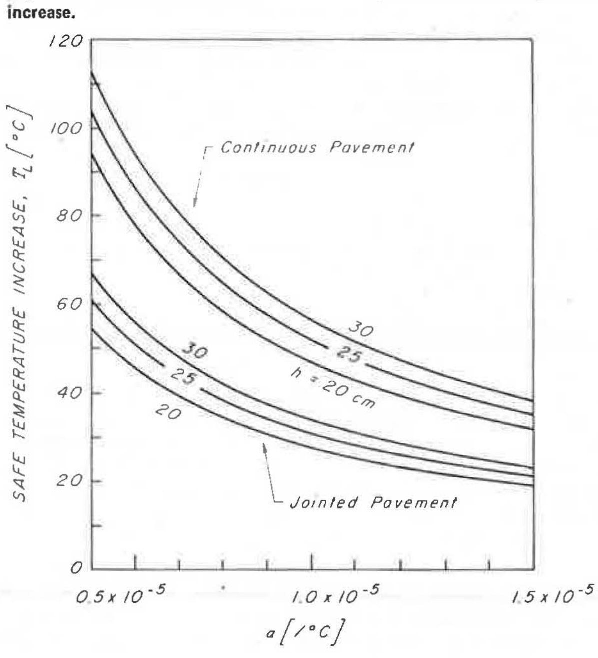


CONCLUSIONS AND RECOMMENDATIONS

A number of important parameters that may have an effect on concrete pavement blowups have been identified: (a) pavement thickness (h), (b) sliding frictional resistance (r_o) at the interface of the pavement and the soil, (c) effective flexural stiffness of the pavement $[EI/(1 - \nu^2)]$, (d) the coefficient of linear thermal expansion (α), and (e) the rotational and axial stiffness of joints and transverse cracks. Because of the importance of these parameters, it is recommended that their values (except h) be determined by using large-scale tests on actual pavements. The variations of these parameters with time, traffic density, magnitude of wheel loads, and type of subgrade and drainage are necessary to predict safe temperature increases in concrete pavements. Also of interest are the factors that alter the neutral temperature after the pavement is in service. In addition, a test program on actual pavements is needed to establish the validity of the safe temperature increase criterion for dynamic as well as for static situations.

The results presented in this paper cover only a few pavement cases. The analyses devloped by Kerr and Dallis (8) and Kerr and Shade (9) can also be used to solve other pavement situations related to blowups (for example, pavements with expansion joints and pavements adjoining bridge piers). Nevertheless, it is hoped that the results and discussion

Figure 12. Effect of coefficient of thermal expansion on safe temperature increase.



presented here will contribute to a better understanding of the mechanics of pavement blowups and will serve pavement engineers as guidelines for prescribing measures to reduce or totally eliminate the occurrence of blowups in concrete pavements.

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Publication of this paper sponsored by Committee on Rigid Pavements.

Overlay Thickness Design Using Rolling Wheel Deflection Ratio and CBR Thickness Formula

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The thickness design formula of the California bearing ratio method is used, in conjunction with the common inverse proportionality between pavement life and some power of the surface deflection recoverable under a rolling wheel, to determine the required overlay thickness for an existing flexible pavement. The proposed procedure is compared with other known methods. It has the advantage of including the effects of equivalent single-wheel load and tire inflation pressure as well as the important parameter of subgrade strength, which do not figure in the other methods.

In this paper, the thickness design formula of the California bearing ratio (CBR) method is used, in conjunction with the common inverse proportionality between pavement life and some power of the surface deflection recoverable under a rolling wheel, in determining the required overlay thickness for an existing flexible pavement.

CBR THICKNESS FORMULA

The design thickness (in millimeters) of a pavement on a subgrade soil with given CBR strength (up to 12 percent) for a given wheel load (P) (for highway pavements, usually 4000 kg on dual wheel) is given by the following equation:

$$t = 2.3 \log(4.5N) \sqrt{P[(1/0.582\text{CBR}) - (1/\pi p)]} \quad (1)$$

where p is the tire inflation pressure (kg/cm²) and N is the number of lifetime applications of the wheel load.

Equation 1 is fitted to the empirical curve of percentage design versus number of applications provided by the U.S. Army Corps of Engineers. A nomographic chart for approximating equivalent single-wheel load (ESWL) and total pavement thickness for a dual wheel, according to Equation 1, is shown in Figure 1. The CBR strength is usually obtainable in field and laboratory tests, or indirectly, in vibratory nondestructive tests that yield Young's modulus of the subgrade (E_s), by using the nonlinear dynamic theory relation--i.e., E_s = 100 CBR (kg/cm²) = 1500 CBR (psi) (1, Figure 10).

OVERLAY THICKNESS FORMULA

The overlay thickness formula, related to Equation 1, reads as follows:

$$t_o = t_2 - t_1 \quad (2)$$