
STRENGTH CHARACTERIZATION AND A BASIS FOR SAWCUTTING REQUIREMENTS FOR JOINTED CONCRETE PAVEMENTS

Jorge B. Soares

Texas Transportation Institute, Constructed Facilities
The Texas A&M University

Dan G. Zollinger

Texas Transportation Institute, Materials, Pavement, and Construction
The Texas A&M University

ABSTRACT

In the early life of a concrete pavement, the pavement's tendency to change its volume due to temperature gradients and/or shrinkage can cause significant stresses due to pavement restraint to movement. Stress concentration on the tip of initial flaws or cracks leads to the propagation of these cracks. To avoid the formation of random cracks, it is common practice in jointed concrete pavement systems to induce cracks by sawcutting at specific joint locations. The spacing and location of the joints are typically determined prior to sawcutting operations. Current sawcut depth and timing requirements are related to the strength characteristics of the concrete pavement. A deficiency in this regard is that strength is not a material constant and it varies with structure geometry and size. It is common knowledge among practitioners and researchers that different test specimens provide different strength values. Also, it is unknown whether the strength obtained from the small specimens is representative of the actual notched pavement strength. New approaches that take into account the geometry and size effects are needed. An approach is presented to characterize concrete susceptibility to crack propagation. Such characterization which makes use of fracture parameters is used to determine sawcut depth and timing requirements for both longitudinal and transverse joints. The effect of slab thickness and aggregate types on those requirements is investigated and is useful in explaining the performance of different methods of sawcutting.

Key Words: concrete strength, fracture, pavements, sawcutting

INTRODUCTION

The performance of a jointed concrete pavement greatly depends on the control of random cracking to desired crack locations. In a newly paved concrete slab, the temperature gradient through the structure depth caused by the hydration process and/or environmental temperature changes can cause significant stresses when the concrete slab is restrained from moving due to friction and/or tied adjoined slabs. Such conditions typically develop at the interior portions of a pavement slab.

By inducing cracks at certain locations at an early age, uncontrolled cracking can be prevented or minimized in jointed concrete pavement (JCP) systems. Crack formation releases energy reducing the stresses in the slab and consequently the potential for formation of undesired cracks. However, in order for stresses to be released, the initial crack (or sawcut notch) applied to form a joint needs to be deep enough so as to propagate to the bottom of the slab. Notch spacing is typically determined prior to sawcutting operations. Notch depth has typically been, with little technical merit, $h/3$ or $h/4$ (h being the slab depth) regardless of concrete properties or slab geometry (1).

Sawcut timing is related to the strength characteristics of the concrete pavement. A deficiency in this regard is that concrete strength is not a material constant since it varies with structure geometry and size. Practitioners and researchers have been aware for years that different laboratory test specimens provide different strength values (2, 3). It can also be questioned

whether the strength measured in these specimens is representative of the strength within the actual configuration of a pavement with a sawcut notch. Since size and geometry effects are present when measuring strength from small specimens, new approaches are needed to determine sawcutting requirements. The Texas Transportation Institute (TTI) has developed and applied fracture theory in both the laboratory and the field to obtain concrete properties that can serve as sawcutting depth and timing criteria. The criteria utilized can be used to evaluate the tensile strength of concrete accounting for slab size and geometry effects. Concrete fracture properties can be easily obtained from a modified split tensile test method based on the fracture mechanics theory (4).

Despite fracture mechanics consistent theory and applicability to concrete materials, fracture properties have still not yet been used to their potential in quality control and design of concrete pavements. One reason engineers have resisted to incorporate fracture theory into quality control and design procedures is its perceived complexity associated with fracture tests. Recent research results that greatly simplify fracture tests represent a step towards the incorporation of fracture properties into quality control and design of concrete pavements. The determination of such properties is also advantageous since it allows the representation of concrete pavement strength as a function of crack growth which is particularly important for sawcutting requirements.

CRACKING STRESSES

When dealing with sawcutting requirements, one is mainly concerned with the stresses developed in the pavement at early ages. Prior to traffic loading, stresses are primarily due to changes in temperature and in moisture content in the concrete. Thermal and shrinkage stresses can be analyzed together by analyzing strains caused by a temperature gradient between the top and bottom of the pavement slab. Although the temperature distribution through the pavement thickness is typically nonlinear, for the sake of simplicity, a linear distribution of temperature is assumed for discussion purposes. Recent studies have demonstrated how nonlinear temperature and/or shrinkage profiles can be simplified to an equivalent linear temperature variation (5). Analysis of the stresses induced by a linear temperature gradient in a restrained slab has been addressed by others (6, 7). In a long and wide pavement slab, the maximum curling stress (σ_o) away from the slab edge is the same in all horizontal directions:

$$\sigma_o = \frac{E \alpha_t dT}{2 (1 - \nu)} \quad (1)$$

where, E and ν are the Young's modulus of elasticity and Poisson's ratio, respectively, α_t is the thermal coefficient of expansion of the concrete, and dT is the difference in temperature from top to bottom of the slab.

For an infinitely long strip of slab (x -direction) with finite width b (y direction) the stresses (σ_x and σ_y , as indicated in Figure 1) at the top of the slab is given by Westergaard (6) as:

$$\frac{\sigma_y}{\sigma_o} = \left[1 - \frac{2 \cos \lambda \cosh \lambda}{\sin 2 \lambda + \sinh 2 \lambda} \cdot \left((\tan \lambda + \tanh \lambda) \cdot \cos \frac{y}{l \sqrt{2}} \cosh \frac{y}{l \sqrt{2}} + (\tan \lambda - \tanh \lambda) \sin \frac{y}{l \sqrt{2}} \sinh \frac{y}{l \sqrt{2}} \right) \right] \quad (2)$$

$$\sigma_x = \sigma_o + \nu (\sigma_y - \sigma_o) \quad (3)$$

where, σ_o is defined in Equation (1), $\lambda = b/\sqrt{8}$, and l is the radius of relative stiffness. For slabs of finite length L_x and width L_y , based on Westergaard's solutions, Bradbury (7) suggested correction factors which depend on the radius of relative stiffness l .

It is common practice to determine maximum thermal stresses on the slab from Equation (1) or Bradbury's equations. E is typically obtained from a relationship to the concrete's compressive strength, ν is often taken as 0.15 for concrete, α , depends on the aggregate type. Typical α , values are $6.0 \times 10^{-6}/^\circ\text{C}$ ($3.3 \times 10^{-6}/^\circ\text{F}$) for concretes with limestone and $10.8 \times 10^{-6}/^\circ\text{C}$ ($6.0 \times 10^{-6}/^\circ\text{F}$) for concretes with river gravel (2). Temperature gradients (dT/h) are typically 0.83 to 1.94°C per 25.4 mm of slab depth (1.5 to 3.5 F per inch). The stresses obtained are then compared to the concrete tensile strength which is obtained from indirect tensile tests performed on small laboratory test specimens.

Considering the concrete properties, temperature difference, and pavement information given in Table 1, from Equation (1), σ_o is 1.46 MPa (211.8 psi). The maximum stress σ_y is at the center of the slab and is approximately σ_o as indicated by Equation (2). If a crack develops along the center line, the maximum value of σ_y is at the quarter points ($\pm b/4$) of the pavement width and it is reduced to approximately $0.30 \sigma_o$. Therefore, it is clear that a longitudinal joint or crack is required to reduce stresses in the slab. The same rationale can be used for a transverse crack, although in this case the slab is not infinitely long in the transverse direction.

As a side note, in order to obtain significant stress relief as indicated above, a crack needs to propagate through the total slab thickness. If the joint or crack propagates only partially, stress is released, but the relief may not be enough to prevent random cracks from occurring somewhere else in the slab. Figure 2 presents finite element calculations of the stress relief for different crack length values (a/h being the crack length-to-slab depth ratio). As in Westergaard's paper (6), the stresses in the y -axis are plotted as a ratio over σ_o and the x -axis as a ratio of the horizontal distance from the center of the slab r over the radius of relative stiffness l . Because of symmetry, only half of the width of the pavement is shown in Figure 2 (from 0 to $b/2$). Note that for $a/h = 0.0$ the finite element stress values obtained match closely the values from Equation (2) indicated in Figure 2 as Westergaard's values (dashed line).

Although from Figure 2 it is clear that the crack (notch) should propagate all the way through the slab thickness, the emphasis of this paper is regarding the required depth of the sawcut notch that will ensure the crack propagation to the bottom. With little technical justification, notch depths of $h/4$ to $h/3$ have been assumed to be sufficient to induce the crack at desired locations (1). Moreover, such depth has been used regardless of concrete properties or slab thicknesses.

With respect to the discussion above, two important aspects are further investigated in this paper: (1) a technical justification for the appropriate sawcut notch depth considering fracture properties of the concrete slab; and (2) the determination of sawcut depth and timing based on the concrete strength as it is configured within a notched pavement and how these are affected by the pavement slab thickness and type of coarse aggregate.

BACKGROUND

In order to fully characterize the propensity for a crack to initiate in a concrete slab, fracture mechanics utilizes a parameter that can be obtained from laboratory tests and be related to structure behavior. One may think of this parameter as being a crack driving force. In fracture mechanics, this parameter is called the stress intensity factor. It is proportional to the applied load and depends on the initial notch length and slab thickness. Depending on the slab thickness considered, the application of fracture mechanics to concrete materials in pavements can employ either linear or nonlinear fracture models.

The use of linear elastic fracture mechanics (LEFM) in the analysis of crack development is based on the assumption that a crack initiates when sufficient energy is provided to cause an assumed pre-existing crack to propagate in an unstable manner. In mode I fracture (opening of crack faces), unstable crack growth occurs when the stress intensity factor K_I reaches a critical value called the fracture toughness K_{Ic} . This critical value expresses the ability of perfectly brittle materials to resist unstable crack growth in the presence of an initial crack. As long as $K_I < K_{Ic}$, the crack does not propagate. Within the LEFM approach, specimen or structure strength can be calculated as $\sigma_N = K_{Ic} / [(\pi a)^{1/2} F(\alpha)]$, where a is the crack length; α is the crack ratio a/d (d being a dimension characterizing the specimen size or slab thickness); and $F(\alpha)$ is a nondimensional geometry factor available from handbooks for basic geometries (8). Note that, since $a = \alpha d$, the strength expression above is a linear function of the structure size d (e.g., slab thickness) as indicated by the slanting line with slope $-1/2$ in Figure 3. The capacity to show the effect of specimen size or slab thickness on strength is a fundamental difference

between fracture mechanics and traditional strength theory which assumes strength to be constant with respect to structure size as indicated by the horizontal line in Figure 3.

Figure 4 demonstrates that in concrete structures, strength varies with size of the structure. However, the size effect in reality does not follow a straight line in concrete materials. Rather, it follows a transitional line that approaches the strength criterion for very small specimens and approaches the limit defined by linear elastic fracture mechanics for large specimens. This transitional zone encompasses the structure sizes of interest for most practical applications (9). The size effect in concrete is due to the presence of a fracture process zone of numerous microcracks that exist in front of a macroscopic crack. Formation and growth of these microcracks consume a large amount of energy in the cracking process serving as a mechanism of concrete toughening. Unstable crack growth does not occur until the fracture process zone develops a certain length. Several nonlinear fracture models modify LEFM by introducing an effective crack length. Among them is the generalized size effect law (9) which proposes that an infinitely large concrete structure or specimen cracks when the stress intensity K_I reaches the critical value K_{If} and the effective crack length reaches the critical length c_f simultaneously. The quantity c_f is referred to as the length of the process zone in an infinitely large structure, and c_f is small compared to the structure size which means that LEFM is sufficiently accurate for analysis purposes. Note that K_{If} and not K_{Ic} , is used as the critical value for the stress intensity factor K_I for materials with large fracture process zones such as in concrete. For these materials, K_{Ic} is known to be highly size dependent and can be calculated in terms of K_{If} which is independent of size (10). Further discussion on the relation between K_{Ic} and K_{If} can be found in references (9-11). It should only be noted here that for typical values of K_{If} and c_f for concrete at early age, the size dependent K_{Ic} was found to be approximately 15% smaller than K_{If} when considering a 304.8 mm (12 in) slab. Therefore, as indicated by Zollinger et al. (12), although the fracture parameters K_{If} and c_f refer to an infinitely large specimen, they can be conservatively applied to pavement slab dimensions.

In summary, the important aspect for considering fracture mechanics is the size effect which can be characterized through a comparison of geometrically similar structures of different sizes as indicated in Figure 3. Conventional stress analysis assumes a constant strength σ_N independent of specimen size. LEFM, by contrast, exhibits a strong size effect relative to the interpretation of concrete strength. The nonlinear models applicable to concrete structures represent a transitional behavior and will be used in the analysis subsequently described in this paper.

SAWCUT REQUIREMENTS BASED ON FRACTURE MECHANICS

One way to determine the necessary sawcut depth in a pavement slab is by comparing the stress intensity factor K_I caused by thermal stresses with the critical stress intensity factor obtained by LEFM, K_{Ic} , for a particular slab thickness considered. K_{Ic} can be obtained as a function of slab thickness d , and the size independent fracture parameters K_{If} and c_f which are obtained from fracture tests on small laboratory test specimens (10). Recent research developments had greatly simplified such tests by utilizing modified split tension cylinders (4).

In this paper, the stress intensity factor due to thermal stresses for the pavement strip with properties given in Table 1 was obtained for five different notch-to-slab thickness ratios $\alpha = a/d$ (0.08, 0.17, 0.25, 0.33, and 0.42). Three different temperature gradients were analyzed. The temperature differences between top and bottom of the slab, given as a mean temperature change of zero, were $\pm 2.78^\circ\text{C}$, $\pm 5.56^\circ\text{C}$, and $\pm 11.11^\circ\text{C}$ ($\pm 5\text{F}$, $\pm 10\text{F}$, and $\pm 20\text{F}$). The temperature distribution was then used in the thermal stress analysis. Both heat transfer and stress analysis were performed with the finite element package ABAQUS (13). The stress intensity factors for different α ratios were obtained using the crack surface displacement method (14).

Note that an important input in the thermal stress analysis is the concrete thermal coefficient of expansion α_t . Analysis using two different α_t values (representing concretes with limestone and river gravel) indicated that the stress intensity factors for concrete with river gravel can be 45% higher due to the increase in the thermal coefficient of expansion. Plots of stress intensity factors versus α ratios are shown in Figures 4 and 5. Temperature differences dT between top and bottom of the slab are indicated in the plots. Note that K_I increases with dT and α .

As previously indicated, in LEFM, the stress intensity factor can always be expressed as $K_I = \sigma(\pi a)^{1/2} F(\alpha)$. The geometry factor $F(\alpha)$, which indicates how K_I varies with α , can be determined from plots of $K_I/\sigma(\pi a)^{1/2}$ versus α using regression analysis. However, the variation of K_I with α for the pavement slab configuration analyzed was shown to be comparable to the known geometry factor for a double edge notch specimen as suggested by Zollinger et al. (12). The geometry factor for that configuration is as follows (8):

$$F(\alpha) = 1.12 + 0.203 \alpha - 1.197 \alpha^2 + 1.930 \alpha^3 \quad (4)$$

By using σ from Equation (1) and the $F(\alpha)$ above, the variation of K_I with α for the case of the double edge notch specimen was obtained and plotted with a dashed line in Figures 4 and 5. Note from these figures that there is not a great discrepancy between how K_I varies with α in the double edge notch configuration and the finite width concrete slab.

Once it is known how K_I varies with α for different temperature gradients, the sawcutting depth can be determined. The critical stress intensity factor determined from fracture tests in the laboratory, K_{Ic} , can be plotted in the curve so as to find the appropriate α for that concrete. As previously noted, K_{Ic} depends on the structure size (slab thickness in this case) and that the size-independent quantity K_{If} can be used to determine the appropriate notch depth-to-slab thickness ratio α . Also, K_{If} is larger than K_{Ic} , and therefore would conservatively require a larger α where the required notch depth is $a = \alpha d$. Therefore, the fracture parameter K_{If} can be used to determine the appropriate sawcut timing relative to the age of the concrete as indicated by Zollinger et al. (15).

Consider the fracture tests performed in a concrete with limestone as coarse aggregate and 12 hour fracture properties as indicated in Table 2. Assuming a temperature gradient of 0.46°C per 25.4 mm of slab thickness (0.83 F/in of slab) equivalent to a $dT = 5.56^\circ\text{C}$ (10 F) for a 30.48 cm (12 in) slab, from Figure 4, α is found to be approximately 0.2. Therefore the appropriate sawcutting depth is 6.1 cm (2.4 in). A smaller notch would be required if K_{Ic} had been considered. If K_{If} is known at different points in time, the appropriate sawcut depth for different times can be obtained.

As previously mentioned, the effect of the pavement slab thickness on the sawcut requirements can be accounted for by considering the size-dependent K_{Ic} instead of K_{If} . Therefore, K_{Ic} can be used as the criterion to determine sawcut notch depth and timing accounting for the effect of aggregate type and slab thickness. In order to use a method more readily associated with common design practices in pavement engineering, this paper presents an approach that considers the effect of pavement thickness on the concrete tensile strength. The approach utilizes the size-independent fracture parameters K_{If} and c_f .

One can utilize fracture parameters for strength determination as a function of notch depth-to-slab thickness ratio. This procedure is discussed with greater detail by Soares and Zollinger (4) where the relationship between the true tensile strength of small test specimens and a concrete pavement is presented. Based on K_{If} and c_f for any specimen of finite size with a characteristic dimension d (diameter of a cylinder or thickness of a slab), the nominal strength can be determined by the size effect law originally proposed by Bazant (16) :

$$\sigma_N = c_n \frac{K_{If}}{\sqrt{g'(\alpha_o)c_f + g(\alpha_o)d}} \quad (5)$$

where σ_N is the true concrete tensile strength, c_n is a coefficient introduced for convenience (4, 9, 17), α_o is the initial ratio (a_o/d), $g(\alpha_o)$ and its derivative $g'(\alpha_o)$ are functions of specimen geometry evaluated where $\alpha = \alpha_o$, $g(\alpha) = \pi \alpha c_n^2 F^2(\alpha)$, in which $F(\alpha)$ is the previously mentioned geometry factor. A combination of the two material factors, K_{If} and c_f and the geometry functions evaluated at $\alpha = \alpha_o$, $g(\alpha_o)$ and $g'(\alpha_o)$, control both the geometry effect and the size effect on the nominal strength.

In order to determine the concrete strength as it is configured within a notched pavement slab from small test specimens, it is necessary to have the geometry factors for both the test specimen and the notched pavement. The geometry factors of the modified cylinders necessary to determine K_{If} were presented by Tang et al. (18). The geometry factor which provides a relationship between K_I and α for the notched slab under thermal stresses was shown to be similar to the geometry factor available for the case a double edge notch specimen and it is given in Equation (4).

Equation (5) is next applied to obtain the true tensile strength of the concrete within a notched slab configuration. In this configuration, the maximum tensile stress is given from Equation (1). The coefficient c_n can be taken as unit and therefore $g(\alpha) = \pi \alpha F^2(\alpha)$.

To illustrate the procedure, the strength of a concrete mixture with a river gravel coarse aggregate at two different ages is considered. The fracture properties at the two different ages are indicated in Table 3. Note that lower fracture parameters are characteristic of concretes which are more brittle (15) and more susceptible to cracking. With $c_n = 1$, $g(\alpha)$ obtained from the $F(\alpha)$ given in Equation (4), and the fracture parameters provided, the true strength, σ_N , can be calculated

from Equation (5) as a function of the ratio α . Figure 6 shows how the true strength of the concrete varies with the notch depth-to-slab thickness ratio. Note that the strength decreases dramatically as the crack ratio approaches 1. A crack ratio of 1 indicates that the notch depth is equal to the slab thickness.

Effect of Pavement Thickness in Sawcut Requirements

In order to account for the effect of pavement thickness in sawcut requirements, the approach presented will compare typical stress levels in pavement slabs to pavement strength as it varies with time and notch-to-depth ratio α . This comparison is illustrated in the form of a chart for a range of slab thicknesses and sawcut times (i.e., concrete ages) in such a way that other thickness and age combinations may be interpolated from the chart data. The fracture parameters considered were obtained at 12 hours for concretes prepared with two types of aggregates, river gravel and limestone. A linear extrapolation was done to obtain the values at 6 hours.

First, consider a concrete with river gravel as the coarse aggregate ($\alpha_t = 10.8 \times 10^{-6}/^{\circ}\text{C}$). The fracture properties obtained at two different ages are indicated in Table 4. The strength as a function of crack length-to-slab thickness ratio as it is configured in a notched pavement can be obtained from Equation (5). The results for the different concrete ages and slab thicknesses are illustrated in Figure 7. The four straight lines in the figure correspond to maximum stresses from Equation (1) for different conditions according to Table 5.

To determine the required sawcut depth, one needs to determine the point where the maximum stress line crosses the strength line under consideration. Table 6 shows the required notch depth for the stress levels considered in Table 5. Note that since the thinner slab has a greater true tensile strength because of the size effect, it always requires a larger α . It is obvious that a 12-hour slab requires a larger α than a 6-hour slab. From Figure 7 and Table 6, note that lower stress levels accentuate the effect of slab thickness and age on the notch depth-to-slab thickness ratio α .

Figure 8 is equivalent to Figure 7 but for a concrete with limestone as the coarse aggregate. Table 5 has the maximum stress levels considering the same temperature differences and ages as for the concrete with river gravel, and assuming the same modulus of elasticity. Due to the lower coefficient of thermal expansion, the maximum stress levels for concrete with limestone are 45% lower than the maximum stresses obtained for concrete with river gravel. As previously mentioned, lower stress levels increase the effect of both age and slab thickness on the required notch-to-depth ratio. This is confirmed in Figure 8 and the results are also contained in Table 6. It is important to observe that when very low stresses are encountered such as in the cases of 0.15 MPa (22 psi) and 0.20 MPa (29 psi) in Table 6, notch-to-depth ratios greater than $h/3$ may be required for the crack to propagate through the slab thickness to avoid random cracking.

Previous research has indicated that the earliest time after pavement placement joint sawcutting can be done (called the near limit of the sawcutting window of opportunity) is associated, relative to the conventional method of sawcutting, to the propensity to cause unacceptable concrete edge ravelling (1). It should be clear that the approach so far described is independent of the method of notch placement and utilizes concrete and slab properties (coefficient of thermal expansion, true tensile strength based on fracture properties, and slab thickness) and maximum stresses in the slab to determine the required sawcut notch depth. However, the data presented in this paper in conjunction with previous research results can be used to determine the appropriate method of sawcutting most suitable to control random cracking.

Maintaining $\alpha = 0.1$ (i.e., $h/10$) for the two previously described cases, the size effect will be reflected in the timing requirement to introduce the notch. Using the tensile strength as the factor to decide the near sawing limit, if the sawcut notch is to be applied at or before the concrete (with river gravel) tensile strength reaches a certain level, say 0.314 MPa (45.6 psi), sawcutting would need to be done before 12 hours for the thicker slab. This is illustrated in Figure 9 (same as Figure 7) where it is seen that the point $\alpha = 0.1$ and $\sigma_N = 0.314$ MPa intercepts the thicker slab strength line at 12 hours. For $\alpha = 0.1$, the thinner slab strength at 12 hours is 0.46 MPa (66.7 psi). Therefore, if one wants to sawcut at 0.314 MPa (45.6 psi), the sawcut would need to be placed before 7.5 hours as indicated by the dashed line which interpolates the 6 and 12 hour lines for the thinner slab.

Maintaining a constant time of placement for the two cases (for a river gravel aggregate), α for the thinner slab will need to be increased to insure proper crack control. If the sawcut notch is to be applied at 12 hours with a concrete tensile strength of 0.314 MPa (45.6 psi), α could still be 0.1 for the thicker slab and would need to approximately 0.3 for the thinner slab as indicated in Figure 9. These results are also summarized in Table 7.

If a concrete with limestone as the coarse aggregate is considered, Figure 8 should be used. Keeping $\alpha = 0.1$ and using again the tensile strength to control the near sawing limit, if the sawcut notch is to be applied while the concrete tensile

strength is 0.314 MPa (45.6 psi), sawcutting would need to be done before 9 hours for the thicker slab, and would need to be done before 6 hours for the thinner slab. If the sawcut time is fixed at 12 hours, α would need to be 0.2 and 0.5 for the thinner and thicker slabs, respectively. These results are also presented in Table 7.

Note that the required sawcut time is earlier for the concrete with limestone than for the concrete with river gravel when α is fixed at 0.1. Note also that when sawcut time is fixed at 12 hours, the notch depth needs to be larger for the concrete with limestone. These results are due the fact that concrete with limestone has greater toughness, i.e., resistance to cracking.

Comparison to Sawcut Field Data

Pavement test sections in Texarkana, Texas (17), where the performance of conventional and early-entry sawcut methods were compared, provided data to validate the analysis described in this paper. The two sawcut methods yielded approximately similar results with respect to the percentage of cracked sawcuts. The sections were 330 mm (13 in) thick doweled plain jointed concrete pavements resting on subgrade. Five different concrete mixes were used in the referred sections: mixes 1, 3, and 4 had river gravel as coarse aggregate while mixes 2 and 5 had limestone. The development of cracks at the sawed joints is illustrated in Figure 10 (combined results for conventional and early-entry sawcut methods) in terms of percentage of sawcuts which cracked. Conventional sawcut notches were $h/4$ deep. Early-aged sawcut notches were placed 25.4 mm (1 in) deep and applied 4.5 to 5 hours after placement of the concrete. No apparent ravelling was observed along the notches by the early-entry sawcut technique. Crack development from the sawcut notches could be clearly observed from the pavement edge prior to pavement opening.

Figure 10 shows the results of both methods of sawcutting relative to the different test sections with their corresponding concrete mixtures and paving dates (in parenthesis). For each test section, the vertical bars, related to the survey date when cracked sawcuts were observed, provide the percentage of cracked sawcuts. All surveys were conducted before the sections were opened to traffic on 7/17/92. For example, for mix 3 (poured on 10/14/91), a little over 10% of cracked sawcuts were observed on 11/10/91 and 2/20/92 (i.e., 27 and 129 days after concrete placement, respectively), and 100% cracked sawcuts were observed on 6/4/92 and consequently on 7/13/92 (after 234 and 273 days).

For the six sections in Figure 10, the early-entry sawcuts tended to initiate cracking earlier than the conventional method. However, after a period of time, the performance between the two tended to be equivalent (17). Not all notches initiated cracking, as it is seen in mixes 1, 2, and 5, 100% cracked sawcuts were not obtained even after 8 or 9 months. Weather conditions during and after paving are believed to have played a critical role in suppressing the development of stresses in the pavement. Starting on 10/27/91, it rained for ten days in Texarkana, which is believed to have reduced temperature change, blocked drying of concrete, and delayed stress development in the concrete. As a result, small stresses were induced (17). As it was shown in Figures 7-9, for low stress levels, deeper sawcut notches are required for a crack to propagate through the slab thickness to induce earlier cracking.

It is interesting to note that, even where conventional sawcutting was used ($h/4$) in Texarkana, there were several cases in which the sawcut notches did not crack. Therefore, engineers should not assume that sawcut joints will necessarily break for large notch depth-to-slab thickness ratios, α , unless requisite stresses are present to induce the cracks. As the analysis presented in this paper indicated, for very low stress levels, an α of 0.5 may have been required to insure crack initiation, which suggests that material and weather combinations need to be carefully considered to insure that the desired sawcut depths will in fact initiate cracking in a timely manner.

As for the effect of coarse aggregate type, the analysis presented in this paper is in agreement with the field data in Texas (17), where it was found that, for the same early-entry sawcut timing, transverse joints showed a greater likelihood of formation for concrete using river gravel as coarse aggregate than for concrete using limestone.

CONCLUSIONS

A characterization of concrete in terms of crack induction at a sawcut notch by the use of fracture properties is presented. The concrete fracture parameters extracted from recently developed fracture tests can be compared to the stress intensity factors caused by thermal stresses in pavement systems. Since a crack will only propagate to the bottom of the slab when $K_I > K_{Ic}$, this fracture parameter was shown to be a pertinent criterion to determine the necessary sawcutting depth. By obtaining the fracture parameter at different points in time, sawcut timing can also be determined. The fracture parameters were also shown to be useful in determining the true tensile concrete strength as it is configured in a notched concrete

pavement. By considering the size effect in the determination of strength, the effect of slab thickness and aggregate type in the sawcut depth and timing requirements can be properly evaluated. The data provided in this paper can be used by project engineers to determine appropriate sawcut timing requirements for concrete paving, and to minimize the placement of late sawcuts relative to the method of sawcutting. This also implies, consequently, the appropriate choice for the method of notch placement. The results of the analysis of the effect of coarse aggregate type is in agreement with available data on sawcut timing. It is evident that not only is the control of random cracking important but also the initiation of cracking such that all sawcut joints are broken. In this respect, sawcut timing is more critical than sawcut depth. It is anticipated that the fracture parameters discussed will eventually be the primary factors in controlling the timing of sawcutting operation to insure the construction of quality concrete pavements.

LIST OF TABLES

TABLE 1.	Inputs for stress calculation
TABLE 2.	Fracture properties of concrete with limestone
TABLE 3.	Fracture properties of concrete with river gravel
TABLE 4.	Fracture properties of concrete for analysis of pavement thickness effect
TABLE 5.	Maximum stresses for concrete with river gravel (RG) and limestone (LS)
TABLE 6.	Required notch depth for concrete with river gravel and limestone
TABLE 7	Sawing limit for different slab thicknesses, notch depths, and aggregate

TABLE 1 INPUTS FOR STRESS CALCULATION

E - CONCRETE MODULUS OF ELASTICITY (MPa)	20,685
A_T - CONCRETE COEFFICIENT OF THERMAL EXPANSION (1×10^{-6})	10.8
ΔT - TEMPERATURE DIFFERENCE FROM TOP TO BOTTOM OF	11.11
B - PAVEMENT WIDTH (M)	7.7
H - PAVEMENT DEPTH (CM)	30.48
K_{sg} - MODULUS OF SUBGRADE REACTION (KN/M³)	27,130
L - RADIUS OF RELATIVE STIFFNESS (CM)	124

1 MPa = 145 PSI
 $T(^{\circ}\text{C}) = [T(^{\circ}\text{F}) - 32] / 1.8$
 1 M = 39.4 INCHES
 1 CM = 0.4 INCH
 1 KN/M³ = 0.0037 PSI

TABLE 2 FRACTURE PROPERTIES OF CONCRETE WITH LIMESTONE

COARSE AGGREGATE	LIMESTONE
AGE OF TESTING	12 HOURS
K_{If} (MPa.M^{1/2})	0.187 MPa.M^{1/2}
C_r (MM)	12.7 MM
K_{Ic} FOR 45.72 CM SLAB	0.157 MPa.M^{1/2}

1 MPa.M^{1/2} = 9.1×10^2 PSI.IN^{1/2}
 1 MM = 0.04 INCH

TABLE 3 FRACTURE PROPERTIES OF CONCRETE WITH RIVER GRAVEL

AGE OF TESTING	K_{If} (MPA.M ^{1/2})	C_F (MM)
12 HOURS	0.144	9.4
28 DAYS	0.928	16.3

1 MPA.M^{1/2} = 9.1×10² PSI.IN^{1/2}

1 MM = 0.04 INCH

TABLE 4 FRACTURE PROPERTIES OF CONCRETE FOR ANALYSIS OF PAVEMENT THICKNESS EFFECT

COARSE AGGREGATE	AGE OF TESTING (HOURS)	K_{If} (MPA.M ^{1/2})	C_F (MM)
RIVER GRAVEL	6	0.072	4.7
	12	0.144	9.4
LIMESTONE	6	0.093	6.4
	12	0.187	12.7

1 MPA.M^{1/2} = 9.1×10² PSI.IN^{1/2}

1MM = 0.04 INCH

TABLE 5 MAXIMUM STRESSES FOR CONCRETE WITH RIVER GRAVEL (RG) AND LIMESTONE (LS)

DT (°C)	5.56	5.56	11.11	11.11
TIME (HOURS)	6	12	6	12
E (MPA)	7,590	10,340	7,590	10,340
Σ_{MAX} (MPA) - RG	0.27	0.37	0.54	0.73
Σ_{MAX} (MPA)- LS	0.15	0.20	0.30	0.40

1 MPA = 145 PSI

 $T(^{\circ}C) = [T(F) - 32] / 1.8$

TABLE 6 REQUIRED NOTCH DEPTH FOR CONCRETE WITH RIVER GRAVEL AND LIMESTONE

Σ_{MAX} (MPa) - RG	0.27		0.54		0.37		0.73	
TIME (HOURS)	6		6		12		12	
SLAB DEPTH (CM)	15.24	45.72	15.24	45.72	15.24	45.72	15.24	45.72
REQUIRED A	0.10	0.07	0.03	0.02	0.20	0.09	0.04	0.03
NOTCH DEPTH (CM)	1.6	3.2	0.5	1.0	3.2	4.2	0.7	1.4
Σ_{MAX} (MPa) - LS	0.15		0.30		0.20		0.40	
TIME (HOURS)	6		6		12		12	
SLAB DEPTH (CM)	15.24	45.72	15.24	45.72	15.24	45.72	15.24	45.72
REQUIRED A	0.50	0.20	0.12	0.08	>0.50	0.43	0.27	0.10
NOTCH DEPTH (CM)	7.7	9.2	1.9	3.7	>7.7	19.7	4.2	4.6

1 MPa = 145 PSI

1CM = 0.4 INCH

TABLE 7 SAWING LIMIT FOR DIFFERENT SLAB THICKNESSES, NOTCH DEPTHS, AND AGGREGATE

COARSE AGGREGATE	SLAB THICKNESS (CM)	NOTCH DEPTH/SLAB THICKNESS	NEAR SAWING LIMIT (HOURS)
RIVER GRAVEL	15.24	0.1	7.5
	45.72	0.1	12
	15.24	0.3	12
	45.72	0.1	12
LIMESTONE	15.24	0.1	6
	45.72	0.1	9
	15.24	0.5	12
	45.72	0.2	12

1CM = 0.4 INCH

LIST OF FIGURES

- FIGURE 1.** STRESSES IN AN INFINITELY LONG PAVEMENT STRIP
- FIGURE 2.** MAXIMUM STRESSES IN AN INFINITELY LONG PAVEMENT STRIP
- FIGURE 3.** SIZE EFFECT FOR GEOMETRICALLY SIMILAR STRUCTURES OF DIFFERENT SIZES
- FIGURE 4.** STRESS INTENSITY FACTOR VERSUS CRACK RATIO (LIMESTONE AGGREGATE)
- FIGURE 5.** STRESS INTENSITY FACTOR VERSUS CRACK RATIO (RIVER GRAVEL AGGREGATE)
- FIGURE 6.** TRUE STRENGTH OF NOTCHED PAVEMENT AT DIFFERENT AGES (RIVER GRAVEL AGGREGATE)
- FIGURE 7.** SAWCUT DEPTH AND TIMING REQUIREMENTS CONSIDERING MAXIMUM STRESSES (RIVER GRAVEL AGGREGATE)
- FIGURE 8.** SAWCUT DEPTH AND TIMING REQUIREMENTS CONSIDERING MAXIMUM STRESSES (LIMESTONE AGGREGATE)
- FIGURE 9.** SAWCUT DEPTH AND TIMING REQUIREMENTS CONSIDERING STRENGTH GAIN (RIVER GRAVEL AGGREGATE)
- FIGURE 10.** PERCENTAGE OF CRACKED SAWCUTS OBSERVED ON DIFFERENT DATES (17)

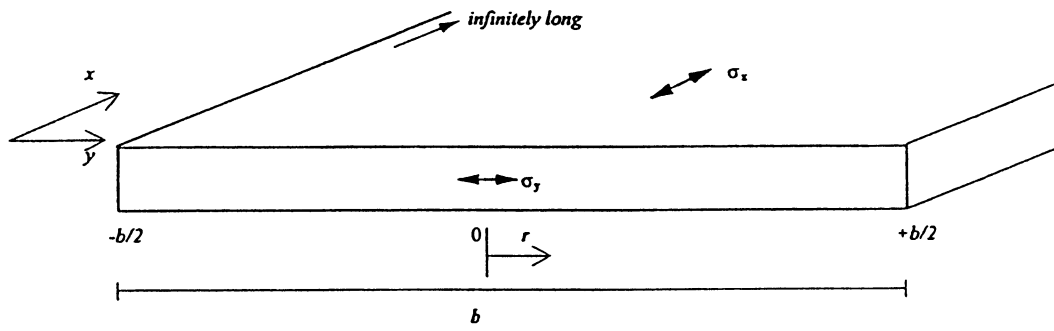


FIGURE 1. Stresses in an infinitely long pavement strip

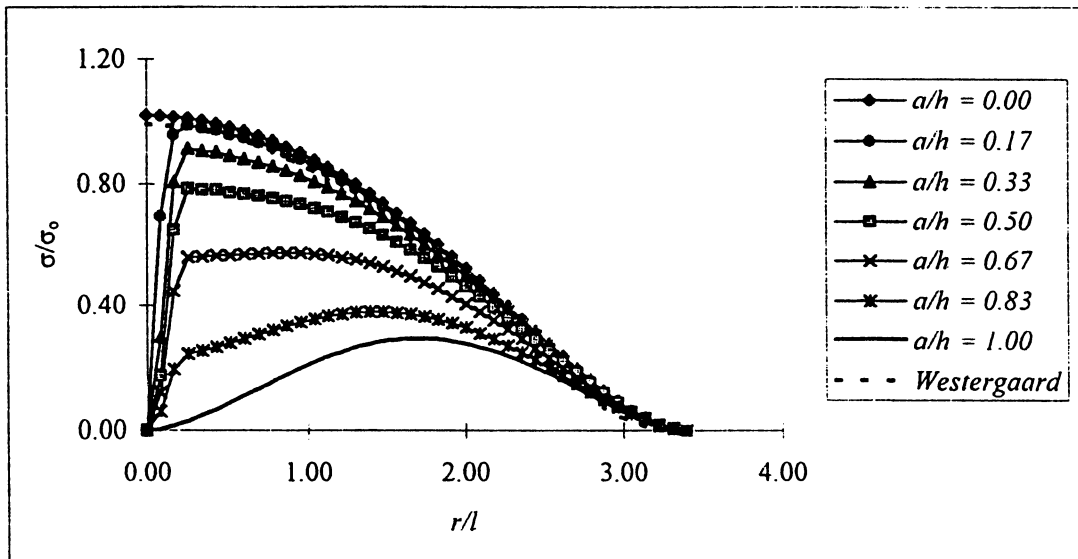


FIGURE 2. Maximum stresses in an infinitely long pavement strip

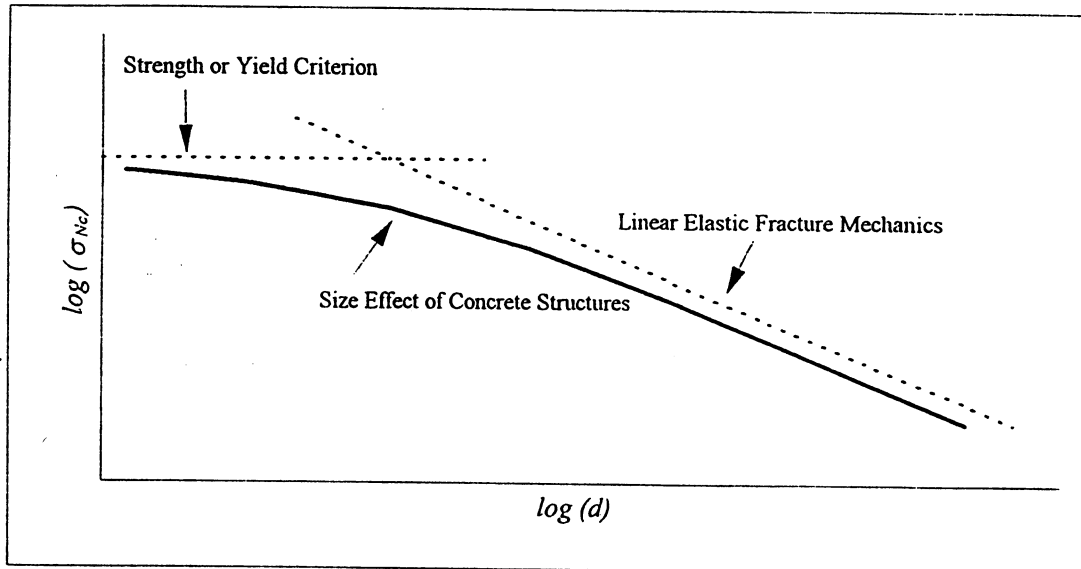


FIGURE 3. Size effect for geometrically similar structures of different sizes

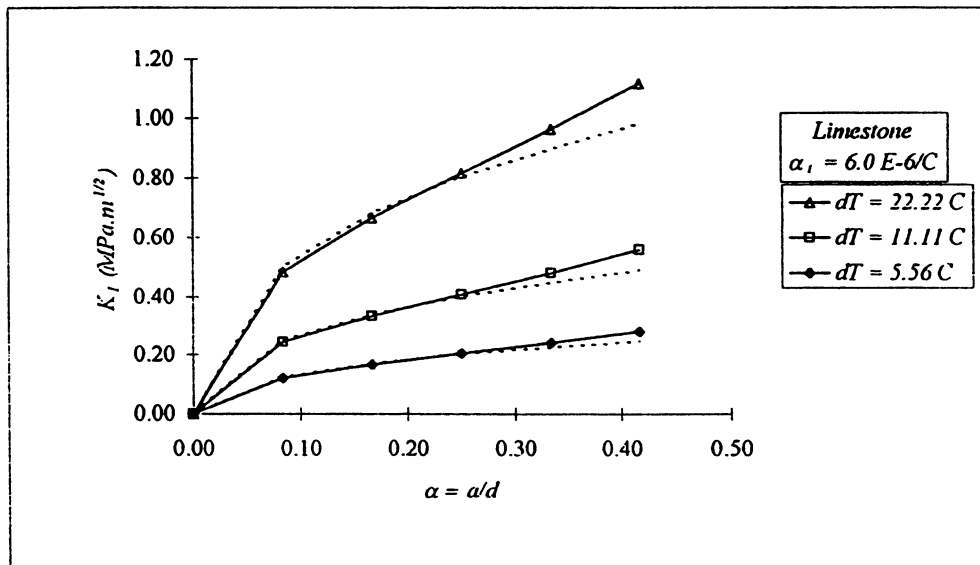


FIGURE 4. Stress intensity factor versus crack ratio (limestone aggregate)

$$1 \text{ MPa.m}^{1/2} = 9.1 \times 10^2 \text{ psi.in}^{1/2}$$

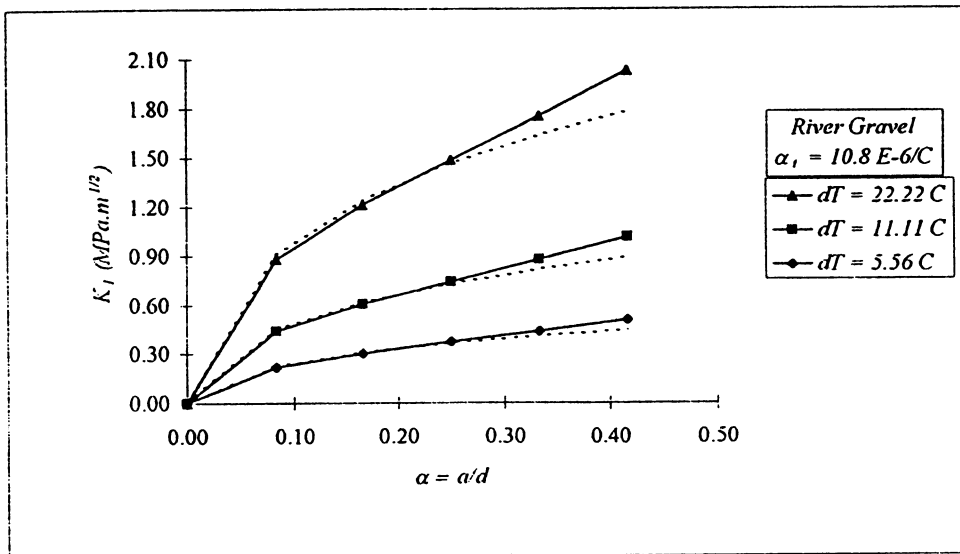


FIGURE 5. Stress intensity factor versus crack ratio (river gravel aggregate)

$$1\text{ MPa}\cdot\text{m}^{1/2} = 9.1 \times 10^2\text{ psi}\cdot\text{in}^{1/2}$$

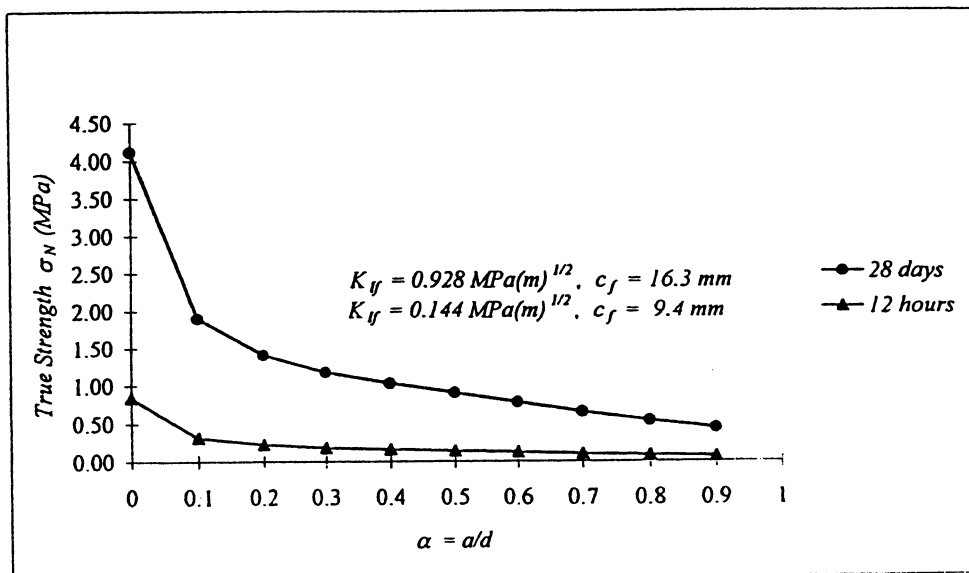


FIGURE 6. True strength of notched pavement at different ages (river gravel aggregate)

$$1\text{ MPa} = 145\text{ psi}$$

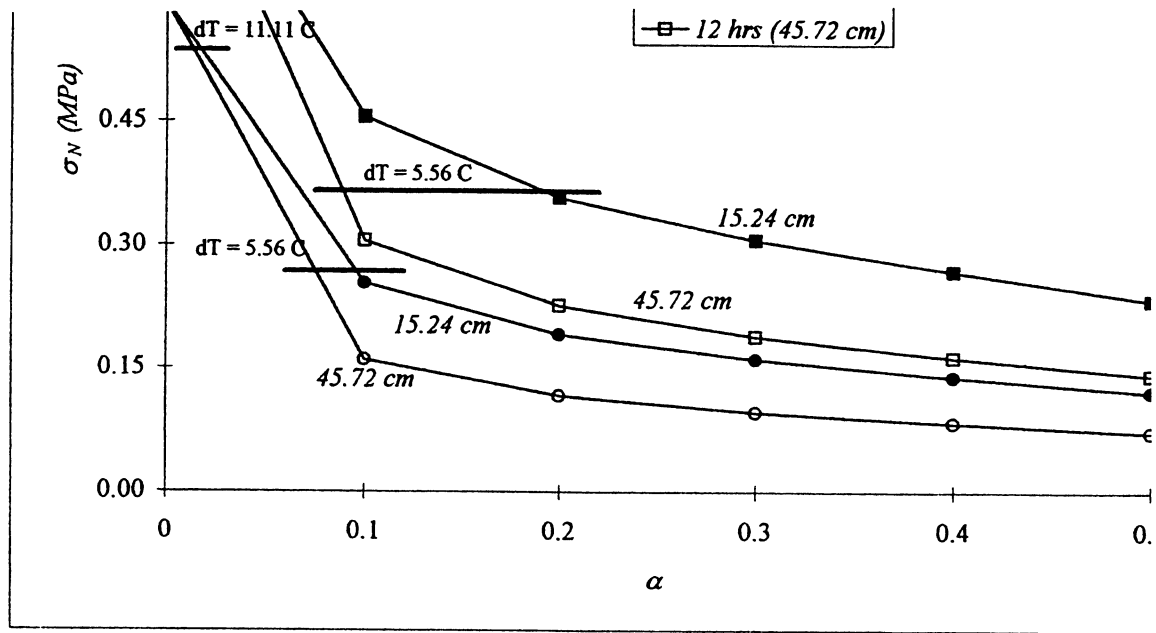


FIGURE 7. Sawcut depth and timing requirements considering maximum stresses (river gravel aggregate)

$$1 \text{ MPa} \cdot \text{m}^{1/2} = 9.1 \times 10^3 \text{ psi} \cdot \text{in}^{1/2}$$

$$1 \text{ MPa} = 145 \text{ psi}$$

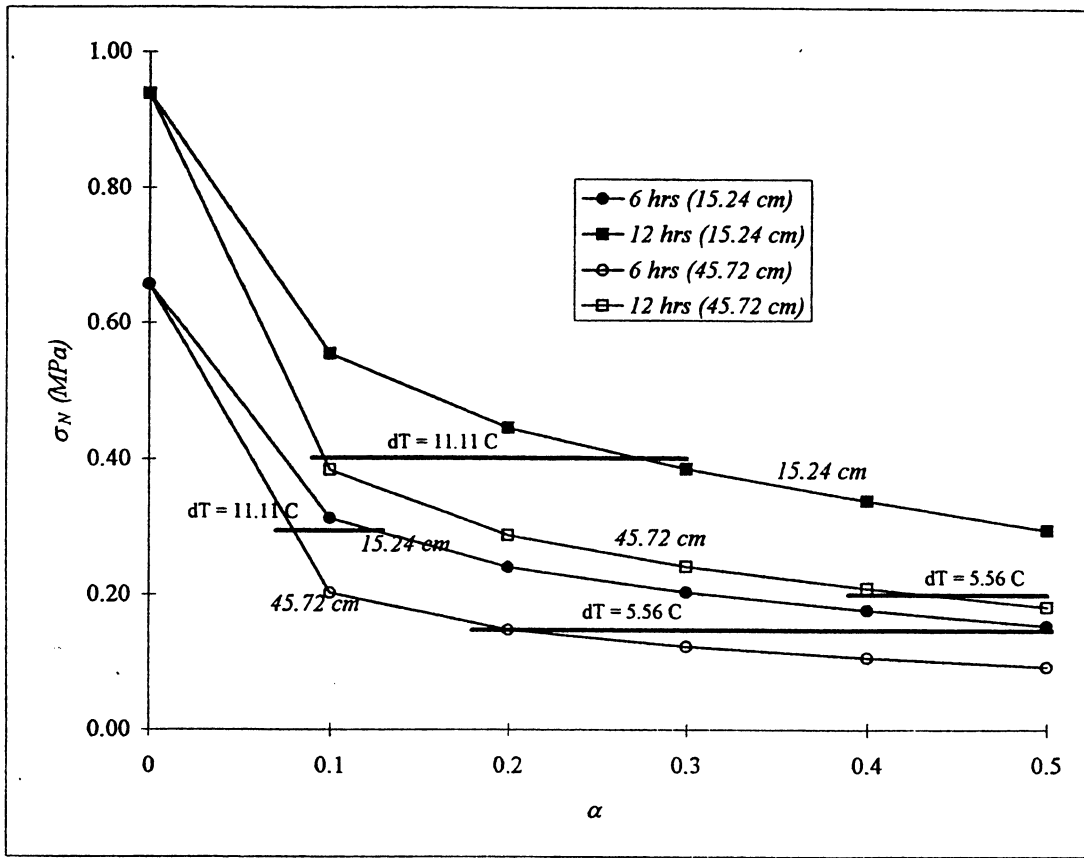


FIGURE 8. Sawcut depth and timing requirements considering maximum stresses (limestone aggregate)

$$1 \text{ MPa}\cdot\text{m}^{1/2} = 9.1 \times 10^2 \text{ psi}\cdot\text{in}^{1/2}$$

$$1 \text{ MPa} = 145 \text{ psi}$$

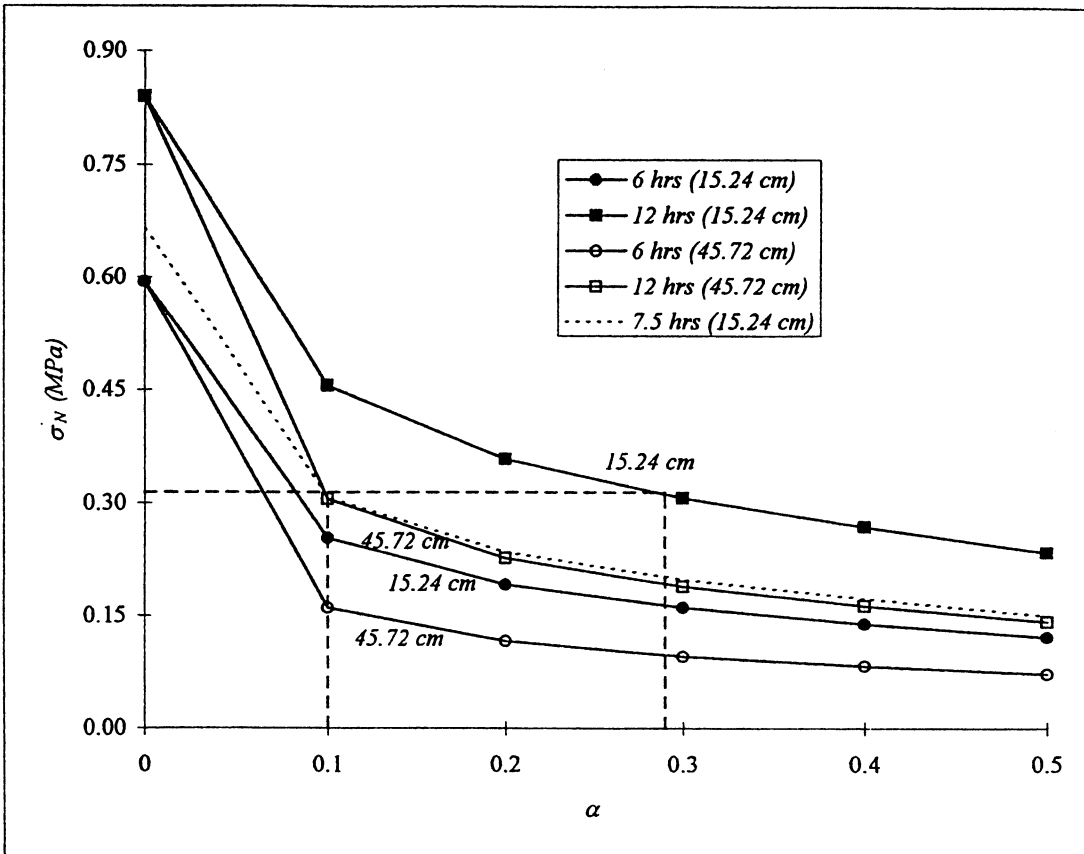


FIGURE 9. Sawcut depth and timing requirements considering strength gain

(river gravel aggregate)

1 MPa.m^{1/2} = 9.1 × 10² psi.in^{1/2}

1 MPa = 145 psi

Percentage of Cracked Sawcuts (%)

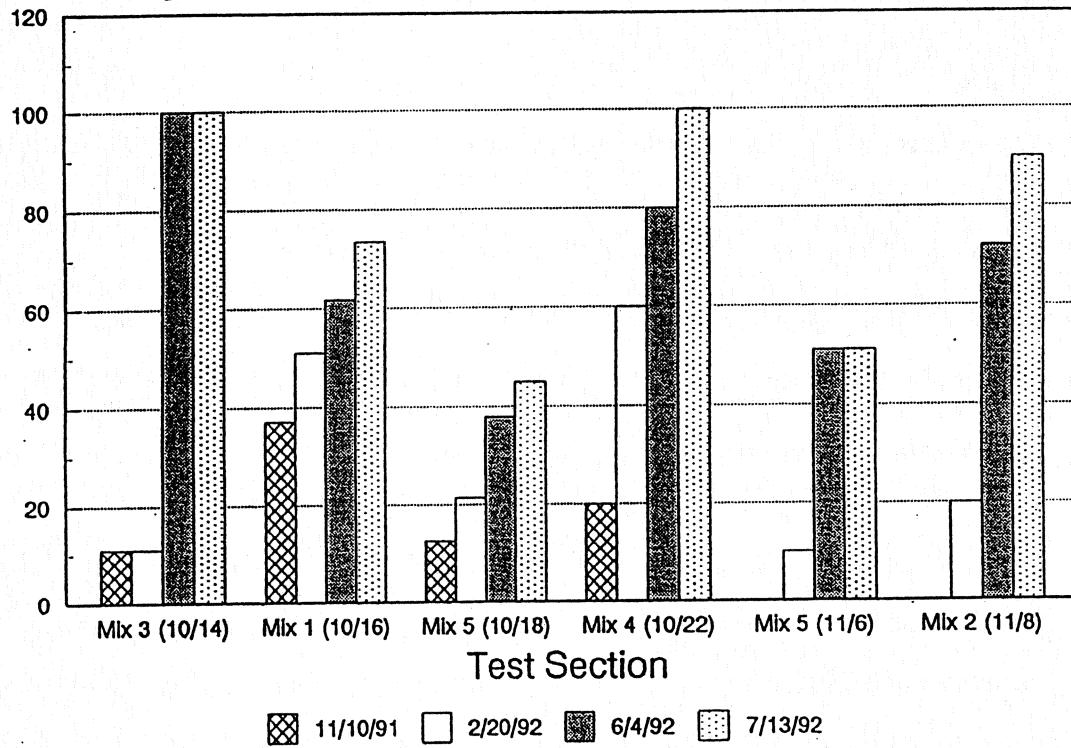


FIGURE 10. Percentage of cracked sawcuts observed on different dates (17).

REFERENCES

1. OKAMOTO, P.A., NUSSBAUM, P.J., SMITH, K.D., DARTER, M.I., WILSON, T.P., WU, C.L., AND TAYABJI, S.D. *GUIDELINES FOR TIMING CONTRACTION JOINT SAWING AND EARLIEST LOADING FOR CONCRETE PAVEMENTS*. FHWA-RD-91-079, 1991.
2. MINDESS, S.; AND YOUNG, J.F., *CONCRETE*. PRENTICE-HALL, INC., 1981, ENGLEWOOD CLIFFS, NJ.
3. WRIGHT, P.J.F.; AND GARWOOD, F. *THE EFFECT OF METHOD OF TEST ON THE FLEXURAL STRENGTH OF CONCRETE*. MAGAZINE OF CONCRETE RESEARCH, VOL. 11, OCTOBER 1952, PP. 67-76.
4. SOARES, J.B., AND ZOLLINGER, D.G. *A COMPATIBLE APPROACH TO CONCRETE STRENGTH CHARACTERIZATION IN THE DESIGN AND CONSTRUCTION OF CONCRETE PAVEMENTS*. TRANSPORTATION RESEARCH BOARD, 76TH ANNUAL MEETING, WASHINGTON, D.C., 1997.
5. MOHAMED, A.R., AND HANSEN, W., *EFFECT OF NONLINEAR TEMPERATURE GRADIENT ON CURLING STRESSES IN CONCRETE PAVEMENTS*. TRANSPORTATION RESEARCH BOARD, 76TH ANNUAL MEETING, WASHINGTON, D.C., 1997.
6. WESTERGAARD, H.M. *ANALYSIS OF STRESSES IN CONCRETE PAVEMENTS DUE TO VARIATIONS OF TEMPERATURE*. PROCEEDINGS OF HIGHWAY RESEARCH BOARD, 1926, PP. 201-215.
7. BRADBURY, R.D. *REINFORCED CONCRETE PAVEMENTS*. WIRE REINFORCEMENT INSTITUTE, WASHINGTON, D.C., 1938.
8. TADA, H., PARIS, P.C., AND IRWIN, G.R. *THE STRESS ANALYSIS OF CRACKS HANDBOOK*. 2ND EDITION, PARIS PRODUCTIONS INC., ST. LOUIS, MO, 1985.
9. BAZANT, Z.P., AND KAZEMI, M.T. *DETERMINATION OF FRACTURE ENERGY, PROCESS ZONE LENGTH AND BRITTLINESS NUMBER FROM SIZE EFFECT, WITH APPLICATION TO ROCK AND CONCRETE*. INTERNATIONAL JOURNAL OF FRACTURE, 44, No. 2, 1990, PP. 111-131.
10. BAZANT, Z.P., AND KAZEMI, M.T. *SIZE EFFECT IN FRACTURE OF CERAMICS AND ITS USE TO DETERMINE FRACTURE ENERGY AND EFFECTIVE PROCESS ZONE LENGTH*. JOURNAL OF AMERICAN CERAMIC SOCIETY, 73, No. 7, 1990, PP. 1841-1853.
11. SHAH, S.P., SWARTZ, S.E., AND OUYANG, C. *FRACTURE MECHANICS OF CONCRETE*. JOHN WILEY & SONS, INC., 1995, NEW YORK
12. ZOLLINGER, D.G., TANG, T., AND DAPENG, X. *SAWCUT DEPTH CONSIDERATIONS FOR JOINTED PAVEMENT BASED ON FRACTURE MECHANICS ANALYSIS*. TRANSPORTATION RESEARCH RECORD 1449, 1993, PP. 91-100.
13. HIBBITT, H.D., KARLSSON, B., AND SORENSEN, E.P. *ABAQUS USER'S MANUAL*. HIBBITT, KARLSSON, AND SORENSEN INC., PAWTUCKET, RI, 1994.
14. CHAN, S.K., TUBA, I.S., AND WILSON, W.K. *ON THE FINITE ELEMENT METHOD IN LINEAR FRACTURE MECHANICS*. ENGINEERING FRACTURE MECHANICS, VOL. 2, 1970, PP. 1-17.
15. ZOLLINGER, D.G., TANG, T., AND YOO, R.H. *FRACTURE TOUGHNESS OF CONCRETE AT EARLY AGES*. ACI MATERIALS JOURNAL, 90, No.5, 1993, PP. 463-471.
16. BAZANT, Z.P. *SIZE EFFECT IN BLUNT FRACTURE: CONCRETE, ROCK, AND METAL*. JOURNAL OF ENGINEERING MECHANICS, VOL. 110, No. 4, 1984, PP. 518-535.
17. BAZANT, Z.P., AND KAZEMI, M.T. *SIZE EFFECT IN FRACTURE OF CERAMICS AND ITS USE TO DETERMINE FRACTURE ENERGY AND EFFECTIVE PROCESS ZONE LENGTH*. JOURNAL OF AMERICAN CERAMIC SOCIETY, 73, No. 7, 1990A, PP. 1841-1853.

18. TANG, T., BAZANT, Z.P., YANG, S., AND ZOLLINGER, D.G. *VARIABLE-NOTCH ONE-SIZE TEST METHOD FOR FRACTURE ENERGY AND PROCESS ZONE LENGTH*. ENGINEERING FRACTURE MECHANICS, VOL. 55, NO. 3, OCTOBER 1996.
19. TANG, T., ZOLLINGER, D.G., AND MCCULLOUGH, B.F. *FIELD TEST AND ANALYSIS OF CONCRETE PAVEMENT IN TEXARKANA AND LA PORTE, TEXAS*. RESEARCH REPORT 1244-7, TEXAS TRANSPORTATION INSTITUTE, COLLEGE STATION, TX, 1996.