Research Report
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CRACKING IN CONCRETE PAVEMENTS

by

Robert C. Deen[1]
Assistant Director

James H. Havens[2]
Director

Assaf S. Rahal[3]
Research Engineer Chief

and

W. Vernon Azevedo[4]
Research Engineer Senior

Division of Research
Bureau of Highways
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

Accepted for Publication
by the
American Society of Civil Engineers

October 1979

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By Robert C. Deen[1], F. ASCE, James H. Havens[2], M. ASCE,
Assaf S. Rahal[3], and W. Vernon Azevedo[4]
(Reviewed by the Highway Division)

INTRODUCTION

Cracking of portland cement concrete pavements is a common, even expected, occurrence. To minimize potentially detrimental effects, it is necessary to understand mechanisms that might be used to explain such behavior so that measures may be developed and implemented to control the occurrence of such cracking. Several "theories" are discussed that might provide the mechanisms of pavement cracking. An alternate meachanism, based on fatigue loading of the concrete by temperature expansions-contrations, for the development of so-called D-cracking is offered.

CRACKING OF PLAIN AND JOINTED CONCRETE PAVEMENTS

Cracking of plain concrete pavements occurs when contraction or distension creates forces equal to the compressive or tensile strengths of the concrete. For example, very young pavement concrete may crack during the first drying (and) or cooling, if the conditions are critical. The resisting forces are due to friction between the pavement and earth. According to the frictional drag theory,

$$F = fW, \qquad 1$$

where F = longitudinal drag force,

f = coefficient of friction between the pavement and earth (usually equal to unity), and

W = normal force (or weight of pavement).

W may be represented by an element of the body of height D and width w, having a length L, multiplied by the density (unit weight), γ_m, of the material. The force, F, may be represented by the product of stress, σ, and cross-sectional area, A; and A may be defined as the product of D and w. Thus,

$$\sigma \times D \times w = \gamma_m \times D \times w \times L \times f \qquad 2$$

and

$$\sigma \quad = \quad \gamma_m \times L, \qquad 3$$

where f is taken as unity. For cracking to occur, σ must be critical. Let σ_{cr} equal the modulus of rupture of the concrete -- say 600 psi (4.1 MPa); assume the density of the concrete to be 144 lbs/ft^3, or 0.0833 lbs/in^3 (9.5 Mg/m^3). Finally,

$$L \approx 600 \text{ lineal feet (183 m).} \qquad 4$$

It is seen that the tensile strength of the concrete in pounds per square inch is equal to the length of pavement in feet which can be mobilized (brought into traction) in both directions about an assumed point (cross section) of reference during contraction. The natural interval between tension cracks, therefore, will be approximately 2L. There would be at least one crack in each 1,200-feet (365-m) length of pavement. Usually, critical drying-shrinkage or cooling occurs before the concrete achieves ultimate strength. For instance, if new concrete achieves 30 psi (0.2 MPa) tensile strength during the first few hours and undergoes further drying and cooling at night, there may be cracks at 60-foot (18-m) intervals. This is a basis for spacing contraction joints at 50 feet (15 m) or less and for sawing joints before the concrete is 16 hours old. Figure 1 shows a shrinkage crack which was well controlled by a sawed joint. Figure 2 shows a crack near a contraction joing which was sawed too late or else the dowels were skewed or were otherwise locked.

Concrete having a compressive strength of 3,000 psi (21 MPa) would be capable of pushing 3,000 lineal feet (915 m) of pavement in both directions from a given point. The normal interval between blowups or severely crushed joints would be between 6,000 and 9,000 feet (1,820 and 2,740 m) (considering compressive strength to be 4,500 psi (31 MPa)) if there were no reductions in cross-sectional bearing such as those made by sawed joints. A 25-percent reduction in cross section would shorten the interval to between 4,500 and 6,750 feet (1,370 and 2,060 m). This is a basis for spacing expansion joints approximately one-half mile (0.8 km) or less apart.

It is seen that the ability of concrete to "push" is about 10 times greater than its ability to "pull". This "push-pull" cycling is a daily and seasonal happening, which may or may not involve freeze-thaw phenomena. Even apart from any freezing and thawing, push-pull stresses are fatiguing if they exceed approximately 50 percent of their failure values. Fatigue is an insidious type of failure which eventually becomes outwardly and visibly manifested in cracks where the stresses have been most concentrated. This type of deterioration has been identified extensively with sawed- or reduced-section-type joints and called "D-cracking" -- but has been attributed by others entirely to unsound aggregate and to freeze-thaw actions *(2, 7)*.

Whereas weakened-plane-type joints effectively discipline the close-interval cracking due to contraction, the pushing stresses accompanying temperature expansion often exceed 50 percent of the critical compressive stress. The thrust force borne by a cross section reduced 25 percent intensifies the stress 1.33 times. Near-critical stresses cause severe fatigue damage. Of course, critical stresses cause explosive shattering or buckling. An increase of 100° F (56° C) in pavement temperature suffices to generate severe compression in a pavement.

FATIGUE OF CONCRETE

Concrete may withstand an unlimited number of applications of stress up to 50 percent of its ultimate strength -- some authorities say up to 55 percent. From the standpoint of pavement slabs, fatigue is usually inferred to mean overstressing due to bending (flexure). Whereas bending due to live loads and warping due to temperature are normally considered in the design of pavement slabs, overstressing (fatigue) in the axial direction due to thermal expansion (compression) has not been regarded heretofore as being significant to the performance of pavements. Unfortunately, progressive deterioration at sawed joints may have been mistakenly attributed by others to freezing-and-thawing and blamed on poor quality of aggregate -- under the name of D-cracking.

Strength of concrete varies somewhat with the mode and rate of loading and with moisture conditions. A given stress may be slightly more (or less, as the case may be) fatiguing at some times than it is at others. A stress near criticality at a given time and in given conditions may cause catastrophic failure at another time under slightly different conditions. Nevertheless, fatigue tests in the compression mode have yielded typical S-N diagrams. Antrim and McLaughlin *(1)* found good correlation between the stress, S, expressed as a percentage of compressive strength, and log N, the logarithm of the number of load applications; their equation for air-entrained concrete was as follows:

$$\log N = 20.501 - 0.214\ S. \qquad 5$$

It is interesting to note that when N = 1, S = 96; when S is in the range of 50 to 55, N is large.

Fatigue damage generally progresses in three or more stages: the first is insidious (or unseen) and continues until a crack develops (crack initiation); the second involves crack propagation through a subcritical to a critical stage; the third stage is the final fracture. The total fatigue life embraces all stages.

THE D-CRACKING PHENOMENON

Whereas some commentaries imply that the term D-cracking in portland cement concrete pavements merely alludes to an early, recognizable symptom of deterioration usually associated with a joint or an edge, others use the term more definitively: that is, to describe D-shaped cracking patterns conjunctive to a joint or edge. From the mid-1920's until the late-1940's, both expansion and contraction joints were considered to be necessary to the satisfactory performance of concrete pavements. Several experimental pavements were built without any joints; the natural intervals of cracking were about 50 feet (15 m) when the concrete contained limestone aggregates and less than 50 feet (15 m) when the aggregate was gravel. The cracking was adjudged to be due, principally, to shrinkage and thermal contraction. Current practices for spacing contraction joints evolved from those experiments. The conception of a contraction joint was a reduced cross section, with or without dowel bars. The depth of the saw cut became D/4 + 1/4 inch (6.3 mm), where D is the thickness of the slab. The thinking was that expansion would somehow take care of itself. Subsequent performance indicated that expansion joints did not prevent blowups. The joints filled with incompressible material and ceased to function. Wooley *(8)* credited this explanation to Griffin *(4)*.

Whereas the experimental pavements without joints contracted and cracked, thermal expansion and closure brought the total cross section into bearing. On the other hand, the sawed contraction joint reduced the bearing area and consequently intensified bearing stresses 1.33 times.

Assuming the coefficient of thermal expansion of concrete to be 5.5 x 10^{-6} per degree F (9.9 x 10^{-6} per degree C) and the modulus of elasticity to be 5 x 10^{6} psi (34.5 GPa), the stress rise for a 100°F (56°C) temperature change, when fully constrained, would be given by $C_c \Delta T E_c$, or 2,750 psi (19.0 MPa). The stress concentrated on the bearing area would be 2,750 x 1.33, or nearly 3,800 psi (26.2 MPa). Normally, pavement concrete is required to have a compressive strength of 3,500 psi (24.1 MPa). Fortunately, most concrete exceeds the strength specified; fortunately, too, the concrete is not fully constrained unless the contraction joints have become filled. Nevertheless, it seems evident that thermal stressing can approach the compressive strength of the concrete at typical contraction joints. Blowups observed have occurred at joints -- not in the slab itself. Blowups most likely occur at joints where the concrete is the weakest.

Others *(2, 7)* have attributed D-cracking at joints principally to freezing and thawing -- and, therefore, to qualities of aggregates. This theory is based on the presence of disintegrated concrete under the joint together with the apparent tendency for the deterioration to begin at the bottom and progress upward. Several observations seem to conflict with this theory. For instance, in no case excavated and examined was there any comparable deterioration at the pavement edge or any unsoundness elsewhere in the slabs. On the other hand, there were overwhelming evidences of diagonal shear and near-horizontal splitting of the slabs *(6)*.

It is hypothesized, here, that the principal mechanism of deterioration of these situations is overstressing of the bearing area at the joint. The principal thrust is generated by thermal expansion; the eccentricity or direction of the thrust at the bearing surface determines the mode or angle of failure. Warping of the slab or intrusions into the joint may cause uneven bearing. Fatigue is believed to be a contributory cause. Indeed, cracking and deterioration tends to begin at the bottom and is insidious in that way. Fatigued and fractured concrete may be affected by freezing and thawing; however, freezing and thawing is not an essential part of the mechanism hypothesized.

Contraction joints should not be sawed (should not have a reduced cross section); they should consist of a parting or a non-adhering separating panel extending full depth and width and should have dowel bars. This type of joint may be kept in a compressed condition, or nearly so, by the proper design of expansion joints or dual-purpose contraction-expansion joints. The compressible material in the joint should provide enough resistance to closure to push back and restore several slabs in each direction, from a given joint, to their equilibrium or "null" position. Limiting the back-pressure of the joint filler to approximately 50 percent of the compressive strength of the concrete would avoid unnecessary overstressing and fatigue of the concrete.

CRACKING OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

Cracking of continuously reinforced pavements has not been fully explained heretofore. There is a pattern or regular interval of cracking which is characteristic of the first cool-down after the maximum heat of hydration has passed. This crack interval may range between 30 and 60 feet (9 and 18 m) and may occur within 24 hours after construction. A second pattern appears during the first significant rise in temperature during or after curing. This interval depends on the strength of the concrete at the time and the percentage of steel. This interval generally ranges between 2 and 6 feet (0.6 and 2 m). A third pattern develops in the end zone -- that is, from the very ends of the concrete and steel inward and into the slab. To the unaided eye, it may appear that the distance to the first crack is 50 to 100 feet (15 to 30 m). Close inspection affirms the presence of close-spaced (2 to 6 feet (0.6 to 2 m)) cracking as found farther inward and more remote from the end. The mechanism by which these cracks are obscured there, but revealed elsewhere, is explained by the so-called drag theory and yielding of the steel.

Rising-Temperature Theory. -- For a temperature change of ΔT, the steel is strained $C_s \, \Delta T$, where C_s is the coefficient of thermal expansion of steel. Likewise, the strain in the concrete is $C_c \, \Delta T$, where C_c is the coefficient of expansion of concrete. Assuming continuity of strains,

$$C_s \, \Delta T + \frac{\Delta \sigma_s}{E_s} = C_c \, \Delta T + \frac{\Delta \sigma_c}{E_c} \qquad 6$$

where E_s and E_c are the moduli of elasticity of steel and concrete, respectively. For a balance of forces,

$$\Delta \sigma_s = \frac{- \Delta \sigma_c}{A_s} \qquad 7$$

where A_s is the area of steel per unit of cross-sectional area of pavement. Substituting Equation 7 into Equation 6 and integrating with respect to T, the stress rise in the concrete per unit length is found to be

$$\Delta \sigma_c = \frac{(C_s - C_c) \, \Delta T}{\left(\frac{1}{E_s A_s} + \frac{1}{E_c}\right)} \, . \qquad 8$$

Using Equation 7, the stress rise in the steel is

$$\Delta \sigma_s = \frac{(C_s - C_c) \, \Delta T}{\left(\frac{A_s}{E_c} + \frac{1}{E_s}\right)} \, . \qquad 9$$

The total stress rise, σ, can be found from

$$\sigma = \Delta \sigma \, \Delta L, \qquad 10$$

where ΔL is the length of pavement under consideration. Substituting Equation 10 into Equations 8 and 9 and integrating with respect to L, it is found that

$$L_c = \left(\frac{1}{E_s A_s} + \frac{1}{E_c}\right) \frac{\sigma_c}{(C_s - C_c)\,\Delta T} \tag{11}$$

and

$$L_s = \left(\frac{A_s}{E_c} + \frac{1}{E_s}\right) \frac{\sigma_s}{(C_s - C_c)\,\Delta T}\;. \tag{12}$$

Equations 11 and 12 can be used to estimate L for the following typical values. Whereas the extreme range of cycling of pavement surface temperatures is from -20°F to 145°F (-29°C to 63°C), the temperatures at the depth of the steel range about 100°F (56°C) between winter and summer. This value of ΔT is used as an approximation of the effective seasonal change in temperature. Estimates of parameters are as follows:

C_c = 5.5×10^{-6}/°F (9.9×10^{-6}/°C),

C_s = 6.5×10^{-6}/°F (11.7×10^{-6}/°C),

E_c = 5×10^{6} psi (34.5 GPa),

E_s = 30×10^{6} psi (206.8 GPa),

A_s = 0.00677, and

ΔT = 100°F (56°C).

Recognizing that the distance between cracks may approach 2L, it is found for concrete with a tensile strength of 600 psi (4.1 MPa) that L_c = 30.7 inches (0.8 m). L_c describes the condition for cracking of the concrete; L_s describes the condition for yielding of the steel. Both conditions are met, approximately simultaneously, when ΔT = 100°F (56°C). It may be noted that the maximum tensile force in the concrete just balances the maximum extension force in the steel. There is no historical basis for balancing these critical forces or for pre-selecting the percentage of steel in this way. Neither is there any historical basis for considering the yield strength of deformed-bar reinforcing steel to be 90,000 psi (620 MPa). Grade 40 or Grade 60 steel surely would have yielded in the 600-psi (4.1-MPa) concrete described above -- that is, 60,000 x 0.00677 = 406.2 psi (2.8 MPa). The analysis suggests that crack intervals typically should be 2.5 feet (0.8 m) or greater but less than 5 feet (1.5 m). This compares favorably with observed crack intervals. These intervals occurred with about equal frequencies. The frequency graphs do, in fact, appear to be truncated in a way suggesting the existence of bi-modal trends.

Bond-Strength Analogy. -- The force in a steel bar at the yield point is $\sigma_{sy}A_s$. To develop sufficient anchorage in concrete to utilize the full force of the bar without pulling out, the bar would have to be embedded a length equal to about 40 diameters. To derive the bond strength needed, let

$$\sigma_{sy} A_s = \mu \times \Sigma \times l_s \tag{13}$$

where μ = bond strength needed (psi),

Σ = perimeter of bar (in.),

l_s = length of embedment (in.),

A_s = $\pi d_s^2/4$, and

d_s = diameter of bar (in.).

When σ_{sy} is 60,000 psi (410 MPa), d_s is 0.625 inch (16 mm), and l_s is 40 diameters, then μ = 375 psi (2.6 MPa). When σ_{sy} = 90,000 psi (620 MPa), d_s is 0.625 inch (1.6 mm), and l_s is 40 diameters, then μ = 625 psi (4.3 MPa). Forty diameters is a rule-of-thumb figure. Contrarywise, the allowable bond stress is generally taken to be 350 psi (2.4 MPa), which in the above case could mobilize a stress of 56,000 psi (386 MPa) in the No. 5 bar. Forty diameters of a No. 5 bar is 25 inches (0.6 m).

This analogy implies that if a continuously reinforced pavement were pre-cracked at intervals of 40 x d_s and were floated and stretched -- that is, pulled uniformly at the tendons -- there would be a chance that yielding of the steel and widening of cracks could occur with great regularity. On the other hand, if the concrete were not pre-cracked and the pavement were stretched by pulling the steel, the steel (if capable of withstanding a greater force than the concrete) would transfer force into the concrete in relation to the depth of embedment. The force in the external portion of the tendon would equal the force in the internal portion of the tendon plus the force in the concrete. The first crack may occur at any point farther than l_s from each end. Further stretching under the sustained force will induce cracks at an eventual interval ranging between l_s and $2l_s$.

The bond-strength analogy parallels the rising-temperature theory described previously insofar as crack spacing is concerned. The motivating force is more readily visualized as arising from temperature distension of the steel -- in which case no external force is required.

Application of Drag Theory. -- Before close-interval cracking occurs, any thermal contraction due to down-cooling tends to shorten the pavement. If the contration were free and unrestrained, the steel would contract slightly more than the concrete; and this tightening of the tendons would bring the concrete into compression. A pavement resting on earth resists expansion and contraction, as described before. The drag resistance balances the combined tensile forces in the steel and concrete until the stress rises in the concrete to the critical level and the concrete cracks. The steel may or may not have yielded when the concrete cracks. When the concrete cracks, the forces tend to transfer to the steel and to induce yielding in the steel then and when further contraction ensues.

The length of pavement which generates a drag force equal to the yield strength of the steel is

$$L \text{ (in feet)} = \frac{\sigma_{sy} \times \text{percent steel}}{100}. \qquad 14$$

When σ_{sy} = 60,000 psi (414 MPa) and percent steel = 0.677, then L = 406.2 feet (104 m); when σ_{sy} = 90,000 psi (620 MPa), L = 609.3 feet (186 m).

If the concrete is already cracked at close intervals, as explained by rising-temperature situations, the steel alone determines the distance from an end where the drag forces cause yielding of the steel and widening of the cracks. For mild steel, this would be about 400 feet (120 m); for high-strength steel, this would be about 600 feet (185 m). This introduces an apparent mismatch of logic inasmuch as the observed distance to the first apparent crack has been in the order of 30 to 100 feet (9 to 30 m). Beyond the first crack, the apparent interval shortens progressively until the normal, short, regular interval becomes established. However, the first down-cooling or curing shrinkage may occur before bond develops between the steel and concrete. For instance, concrete having a tensile strength of 30 to 50 psi (0.2 to 0.35 MPa) would crack at 30- to 50-foot (9- to 15-m) intervals and crack later at 2.5- to 6.0-foot (0.8- to 1.8-m) intervals. Only the earliest shrinkage cracks would be at all apparent to the observer -- and they might be more so farther inward from the end.

The two mechanisms superimpose near the end of a slab. The close-interval cracking has been demonstrated experimentally to be independent of position in a pavement and to be dependent upon the characteristics of the steel and concrete.

Whereas the very first cracking may occur at intervals of 30 to 50 feet (9 to 15 m) and whereas close-spaced cracking must await the development of nearly full strength of the concrete and a critical rise in temperature, close-spaced cracks will not be readily apparent near the end of the slab. However, if the tendons at the end of a slab were pulled, the interval between cracks would be close-spaced; but the crack width would be greatest nearest the end and would diminish farther inward into the slab and as drag resistance rises in proportion to distance.

CRCP in Kentucky. -- Approximately 7.12 miles (11.5 km) [29 lane-miles (46 lane-km)] of continuously reinforced concrete pavement were placed using slip forms on I 71 in Henry, Trimble, and Carroll Counties. Paving was completed there in late 1968 *(3)*. The slab was 8 inches (203 mm) thick and contained 0.677 percent longitudinal steel placed at a nominal depth of 3 ± 0.5 inches (76 ± 13 mm) below the surface. No. 5 deformed bars were spaced transversely at 5.5 inches (140 mm). Two inner lanes of mainline pavement extending 4.7 miles (7.6 km) westbound on I 275 from I 75 in Boone and Kenton Counties were built in 1971; the outer lane and a ramp from I 75 southbound onto I 275 westbound were completed in 1972. No. 5 bars were used at intervals of 4.75 inches (121 mm) in the 9-inch (229-mm) slabs. This provided 0.677 percent longitudinal steel for both slab thicknesses. The minimum expected 28-day compressive strength for pavement concrete was 3,500 psi (24 MPa), and the minimum expected 28-day modulus of rupture was 550 psi (3.8 MPa) when tested in accordance with ASTM C 39 and ASTM C 78.

Crack surveys were made periodically on I 71 and I 275; 200 feet (61 m) in each 1,000 feet (305 m) were sampled on I 71; 100 feet (30 m) in each 1,000 feet (305 m) were sampled on I 275. Any crack, regardless of size, was counted and recorded. The survey made in July 1969 on I 71 gave an average interval of 5.78 feet (1.76 m) between cracks. In July 1970, another survey was conducted, and the average interval between cracks was 4.14 feet (1.26 m). There was a strong tendency for cracks to occur at intervals at or between 2.5 and 5 feet (0.8 and 1.5 m).

The first cracking observed on I 275 was at a regular interval of 30 feet (9.1 m) and occurred during an onset of winter temperatures soon after construction. Surveys of the mainline section of I 275 in February 1972 (4 months after construction) indicated a dominant interval of 3 to 6 feet (0.9 to 1.8 m). By October 1972 (1 year after construction), the dominant interval had decreased to 2 to 5 feet (0.6 to 1.5 m). The mature spacing of cracks on the mainline section of I 275 is shown in Figure 3.

To develop a time-to-cracking relationship, surveys of ramp sections of I 275 were begun immediately after construction (April 1972) and made at short time intervals. Most of the cracking occurred within the first 2 weeks after construction; this is shown in Figure 4. The spacing between cracks on the ramps later in 1972 is given in Figure 5.

Cores from the I-71 project approximately 3.5 years after construction showed that the cracks extended through the full depth of the slab. The steel showed slight traces of corrosion.

The crack patterns approaching the terminal joints are obscured. In the July 1970 survey (I 71), there was no apparent cracking in the first 85 feet (26 m) from the beginning station and no obvious cracking in the last 25 feet (8 m) approaching the ending station. Several other end situations existed within this project, and all conformed more or less to this pattern. Ends, of course, are free to expand and contract to some extent.

Apparently, traffic had little influence on the crack interval on I 275. The incidence of cracking in the outside lane was slightly greater than in the two inside lanes on the mainline.

The construction season and the early curing temperatures may have affected early crack development but had little or no affect on the ultimate crack intervals. I 71 and the mainline inner lanes of I 275 were paved in the fall; the outer lane and a ramp on I 275 were paved in early spring. The mainline inner lanes of I 275 which were exposed to cold temperatures at an early curing age first developed cracks at a regular interval of about 30 feet (9 m); otherwise, the majority of cracking occurred at an early age; and all pavements seemed to reach an equilibrium in cracking after approximately 1 year.

BUCKLING

Thermal compression in a pavement occurs when the temperature rises and when free expansion is not permitted because of natural or imposed constraints. Constraints may be partial or complete. The actual distension or elongation together with any further elongation which would occur upon removal of any constraints is equal to the free expansion. That portion of the expansion which would occur if constraints were removed is virtual expansion, and that portion of the strain is virtual strain. The product of this strain and the modulus of elasticity is stress.

A plain concrete pavement undergoing a 100°F (56°C) rise would have a free expansion of about 3 feet ($5.5 \times 10^{-6} \times 100°F \times 5{,}280$) per mile (0.57 m per km). If completely constrained, the stress rise would be about 2,750 psi (19 MPa) -- that is, $5.5 \times 10^{-6} \times 100°F \times 5 \times 10^{6}$. Indeed, this stress rise alone is likely to exceed 50 percent of the compressive strength of the concrete and is, therefore, fatiguing to the concrete. Extreme effects at points of weakness and (or) reduced sections have been described previously. An overthrust of 3 feet (0.9 m) or an upthrust or arching of slabs could occur. This is a form of buckling.

A continuous strand of reinforcing steel undergoing a 100°F (56°C) rise would have a free expansion of 3.4 feet ($6.5 \times 10^{-6} \times 100°F \times 5{,}280$) per mile (0.6 m per km) -- that is, approximately 0.4 feet per mile (76 mm per km) greater than the expansion of concrete. S-shaped bends or laps of several inches (mm) have been noted in instances where continuously reinforced pavements have blown up or shattered.

Partial-width patches, especially those made with bituminous concrete, tend to invite buckling and blowups to occur later. Perhaps it would be more appropriate to state that any reduction in cross section or bearing area of the cross section, such as may be caused by deterioration and removal of concrete and the concentration of compressive forces onto the remaining area may lead to blowups or buckling. Partial-width patching, therefore, should include a compression relief joint extending the full width of the pavement.

SIMULATIONS

Inasmuch as the theory attributing close-spaced cracking in continuously reinforced concrete to the greater thermal expansion of the steel seemed to evoke disbelief when first proffered *(5)*, a demonstration was designed to induce cracking without applying external force. First, a beam, 4 inches by 4 inches by 12 feet (100 mm by 100 mm by 3.6 m) was cast about a No. 5 re-bar (Grade 40, σ_{sy} = 50,900 psi (351 MPa)) (Figure 6) and cured until strengths of 703 psi (0.485 MPa) and 4,931 psi (3.4 MPa.) were reached in tension and in compression. The beam was enveloped and warmed by heated air. To monitor temperatures, thermocouples were installed in the concrete at various depths; and to monitor cracking of concrete, yielding of steel, and mere slip, Dunegan acoustic emission instrumentation was attached to the steel and concrete. The warming proceeded slowly to about 180°F (82°C), and only weak emissions were detected. Later, the re-bar was heated electrically by attaching arch-welding equipment to the exposed ends. It was found that 250 to 300 amperes produced a rate of heating which would achieve 180°F (82°C) in less than an hour. The heating produced intensive emissions at times from detectors mounted on the steel and mild outbursts from the concrete. The cracks are lined in Figure 7, Beam 1.

A second beam was made using Grade 60 steel (σ_{sy} = 67,727 psi (466 MPa)). When heated to about 180°F (82°C), the steel emitted very little "noise;" but the concrete had emitted several mild outbursts (see Figure 7, Beam 2). It appeared that the steel had yielded in the first beam but not in the second. This also appeared to be so from the standpoint of the strength of the concrete and σ_{sy} of the steel.

To further demonstrate the yielding situation, a third beam 4 inches by 6 inches by 12 feet (100 mm by 150 mm by 3.6 m) containing a 3/8-inch (9.5-mm) Grade 60 bar was made and tested as before. Strong outbursts of "noise" from the steel were again related to yielding of the steel; and milder noises were related to cracking of the concrete. The spacing of the cracks was as shown in Figure 7, Beam 3. Attenuations of noise rates correspond to relief of stress in the steel by cracking of the concrete. In the test in which the steel yielded, some 28,000 acoustic emission counts were recorded. The maximum re-bar temperature was 160°F (71°C). In the test which precluded yielding, only about 9,000 acoustic emission counts were recorded, though the maximum test temperature of the steel was 210°F (99°C). Subsequent acoustic emission tests on concrete beams and cylinders revealed that the quantity of acoustic emission from fracturing concrete depends on the strength and mode of loading. The results indicate that concrete is probably not a high emitter. Therefore, the major source of acoustic emissions in these tests was the reinforcing steel.

IMPLICATIONS FROM THEORIES

Deteriorations as witnessed and explained by the mechanisms or theories advanced here are not directly associated with live load, traffic, or use of the pavement. It is implied that cracking would have occurred even if there had been no hauling or travelling on the pavement. It is implied, moreover, that there are defects in design concepts -- which, if corrected, would surely extend the life and efficiency of concrete pavements generally and would yield service to the user on a better investment-recovery basis.

Beam action is severely reduced by close-spaced cracking. The short slabs tend to rotate when the load is over a crack; the steel undoubtedly provides some hinge action unless it becomes fatigued and is broken. Bearing pressures beneath the pavement are greater when the wheel loads are centered over the cracks.

Continuously reinforced pavements undergo numerous push-pull cycles; the steel yields; and the concrete fatigues. Buckling and blowups occur because expansion joints are not provided at intervals of one-half to three-quarters of a mile (0.8 to 1.2 km). To prevent close-spaced cracking, the steel would have to be smooth, lubricated bars. The bars might be threaded at the ends and be snugged by nuts when the curing temperature is highest. Snugging again after significant strength gain and at summer-high temperatures would bring the tendons into tension upon cooling and tend to reduce contraction cracking to some degree. The slab length, from this point of view, probably should not exceed B00 to 1,000 feet (240 to 300 m).

APPENDIX I. -- REFERENCES

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APPENDIX II. -- NOTATION

A_s = area of steel

C_c = coefficient of thermal expansion of concrete

C_s = coefficient of thermal expansion of steel

D = thickness of concrete slab (height of slab element)

d_s = diameter of steel bar

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

F = longitudinal drag force

f = coefficient of friction between the pavement and the subgrade

L = length of concrete slab element

L_c = change in length of slab due to stress rise in the concrete

L_s	=	change in length of slab due to stress rise in the steel
l_s	=	length of embedment of steel bar in concrete
N	=	number of load applications
S	=	stress level
W	=	normal force (or weight of pavement)
w	=	width of concrete slab element
ΔL	=	change in length or length of slab under consideration
ΔT	=	change in temperature
$\Delta\sigma_c$	=	stress rise in concrete per unit length
$\Delta\sigma_s$	=	stress rise in steel per unit length
γ_m	=	density (unit weight) of concrete
σ	=	total stress rise
σ_{cr}	=	critical stress
σ_{sy}	=	yield stress of steel
μ	=	bond strength (between concrete and steel)
Σ	=	perimeter of steel bar

CRACKING IN CONCRETE PAVEMENTS

KEYWORDS: Bond strength; Concrete pavements; Continuously reinforced concrete pavements; Cracking; D-cracking; Fatigue; Plain and jointed pavements; Temperature expansions and contractions

ABSTRACT: Several theories explaining the mechanisms and intervals of cracking of portland cement concrete pavements are reviewed. Crack and blowup intervals for plain and jointed pavements and continuously reinforced concrete pavements were predicted and verified by field observations.

REFERENCE: Deen, Robert C.; Havens, James H.; Rahal, Assaf S.; and Azevedo, W. Vernon; *Cracking in Concrete Pavements*

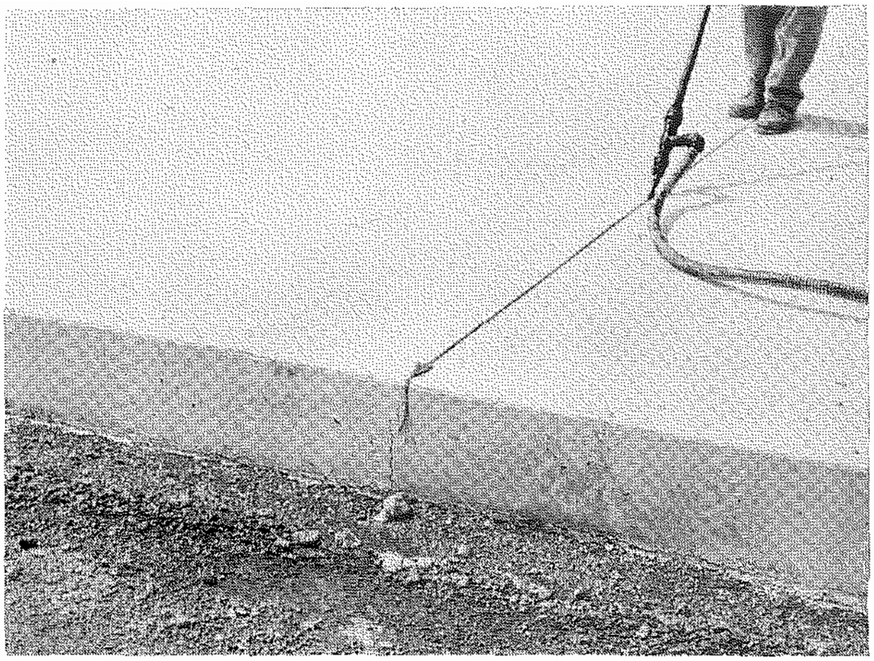
CRACKING IN CONCRETE PAVEMENTS

KEYWORDS: Bond strength; Concrete pavements; Continuously reinforced concrete pavements; Cracking; D-cracking; Fatigue; Plain and jointed pavements; Temperature expansions and contractions

ABSTRACT: Several theories explaining the mechanisms and intervals of cracking of portland cement concrete pavements are reviewed. For plain and jointed pavements, the cracking interval was found to be approximately twice (when expressed in feet) the strength (when expressed in psi) of the concrete. For new concrete with a 30-psi (0.2 MPa) tensile strength after only a few hours, the drying-shrinkage or cooling crack interval is approximately 60 feet (18 m). This is a basis for sawing joints at approximately 50-feet (15-m) intervals before the concrete is 16 hours old. For concrete with a compressive strength of 4,500 psi (31 MPa) and sawed joints, the interval between blowups or crushed joints would be about 1 mile (1.6 km). This is the basis for spacing expansion joints approximately 0.5 to 1 mile (0.8 to 1.6 km) apart. A theory of differential temperature expansion-contraction between concrete and steel explains the ultimate crack interval of continuously reinforced concrete pavements between 2 and 6 feet (0.6 and 2 m). This interval was verified by field observations.

REFERENCE: Deen, Robert C.; Havens, James H.; Rahal, Assaf S.; and Azevedo, W. Vernon; *Cracking in Concrete Pavements*

Figure 1. Normal Shrinkage Cracks, Controlled by Sawed Joint.



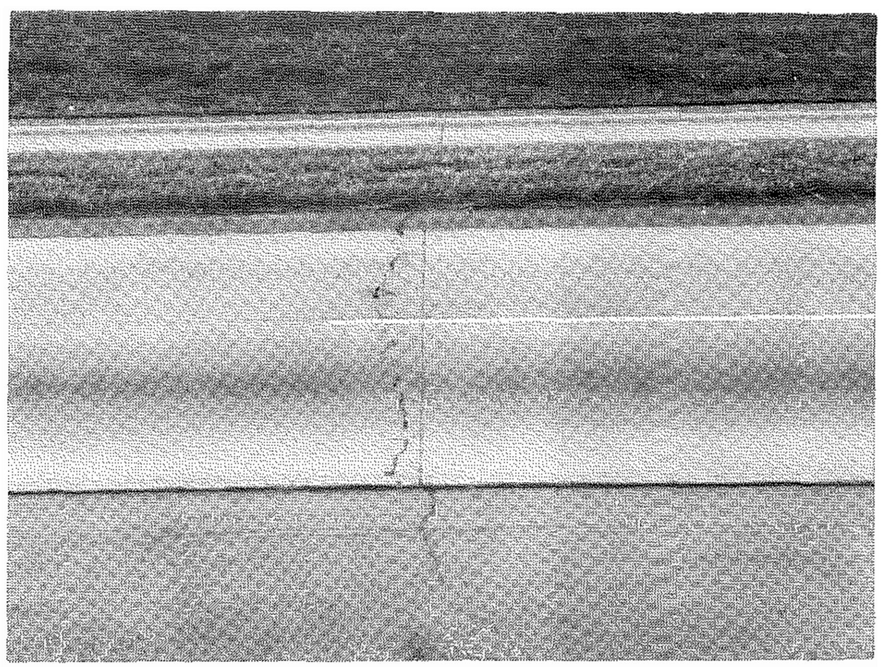


Figure 2. Shrinkage Crack; Either the Joint was Sawed too Late or the Dowel Bars Locked or Froze (Misaligned).

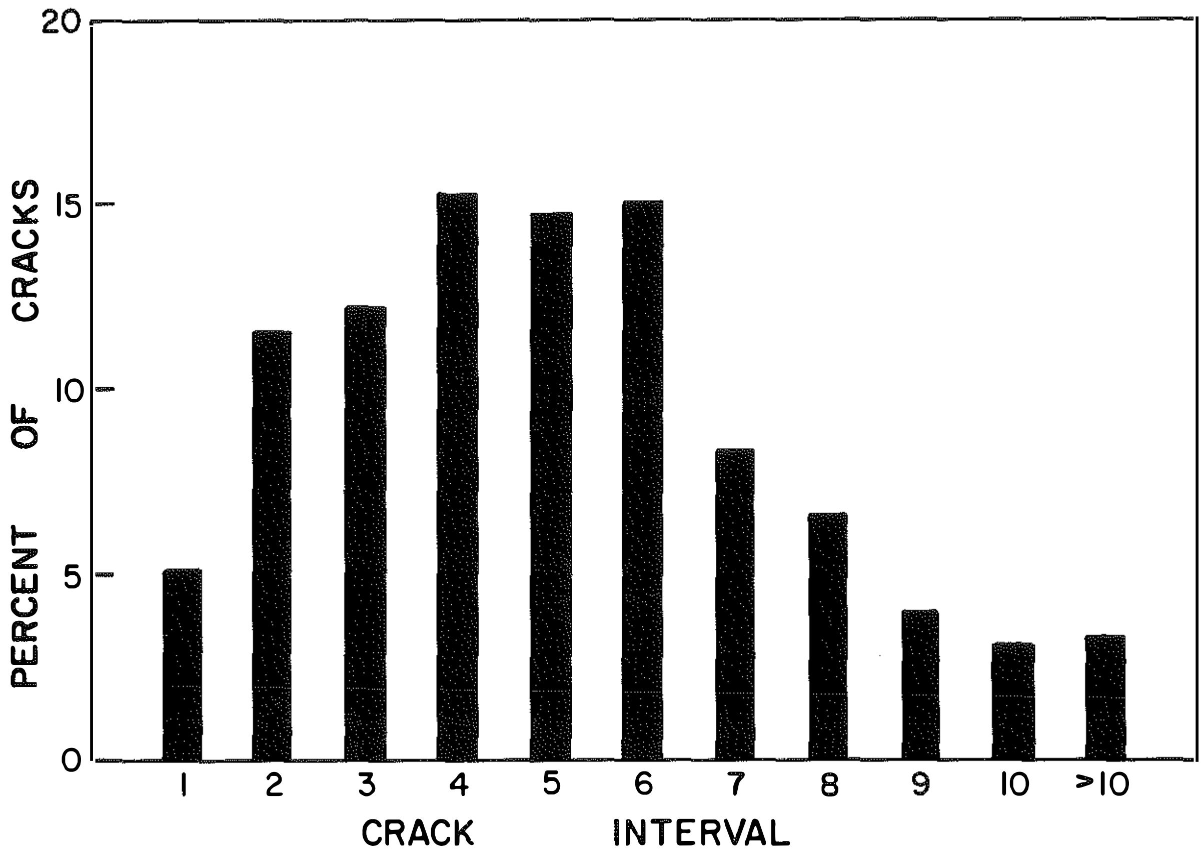


Figure 3. Mature Spacing of Cracks on Mainline Section of I 275.

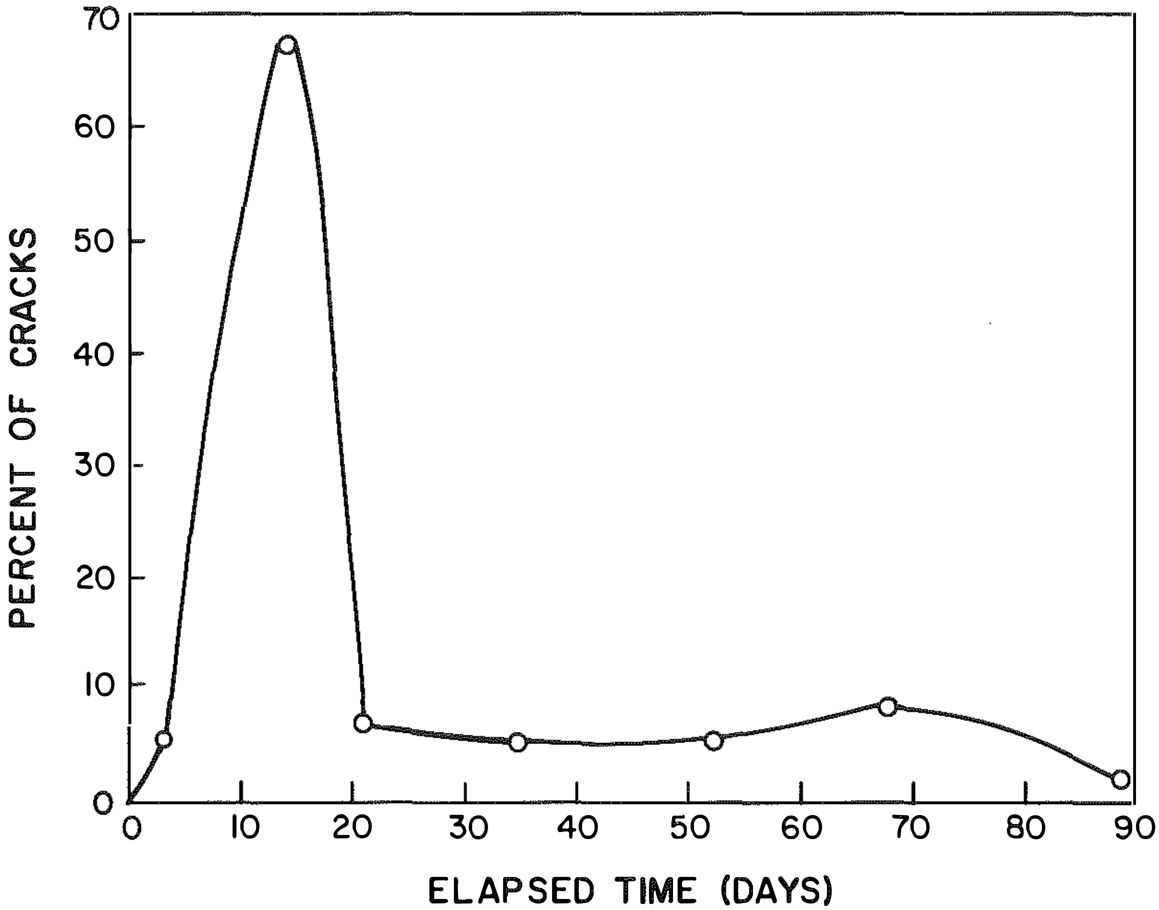


Figure 4. Spacing between Cracks in Relation to Time (Age of Concrete) on Ramp Section of I 275.

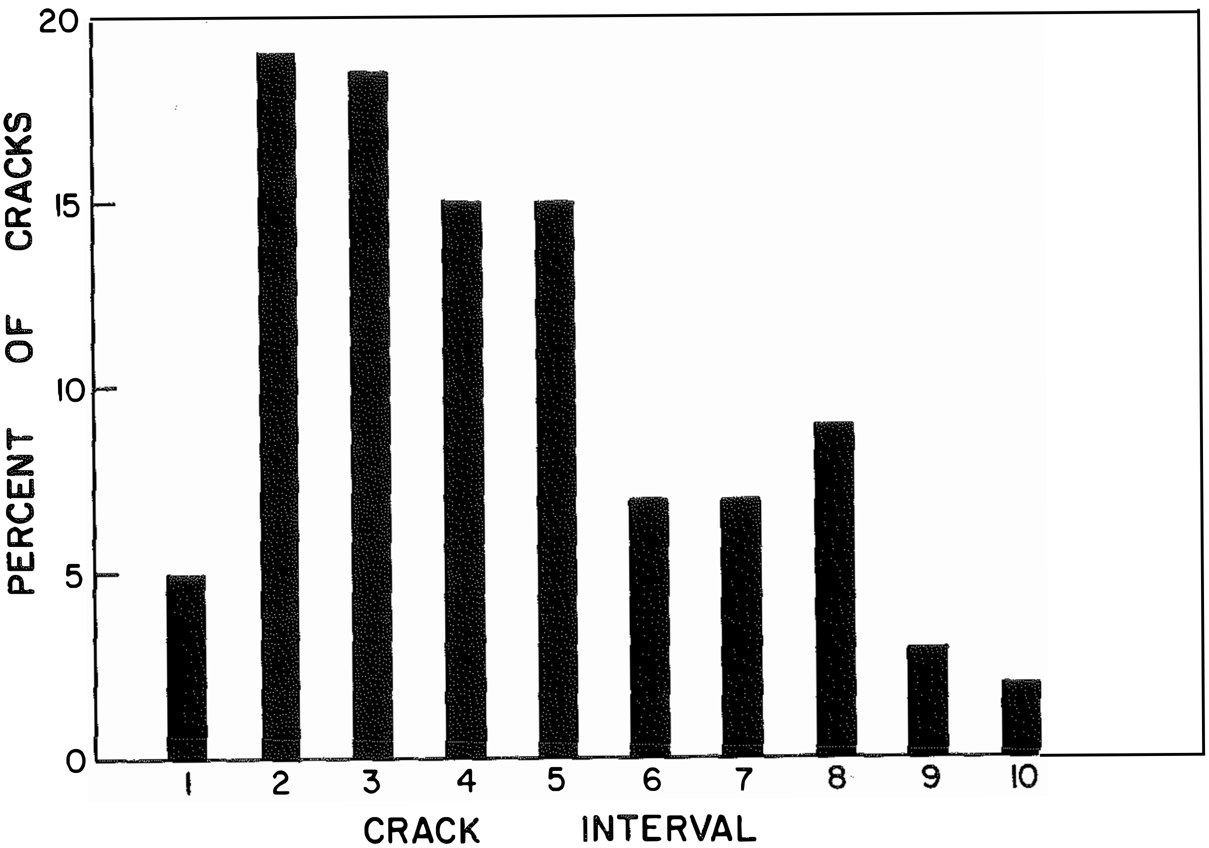


Figure 5. Mature Spacing of Cracks on Ramp Section of I 275.

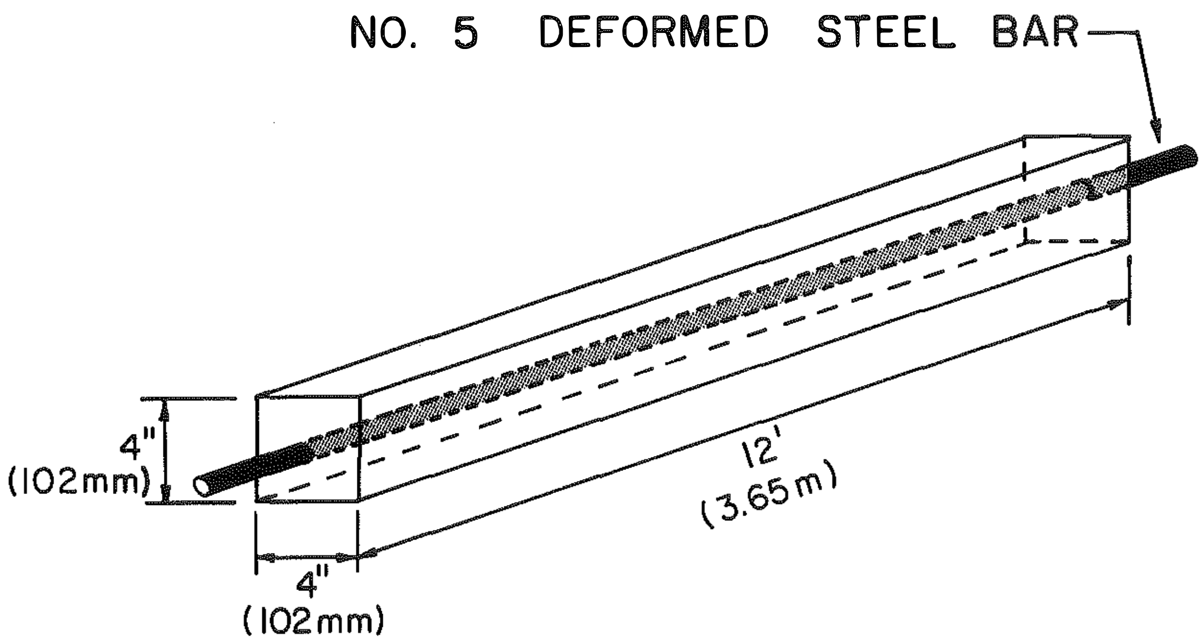


Figure 6. Test Specimen; Beam Used to Simulate Cracking Conditions in Reinforced Concrete.

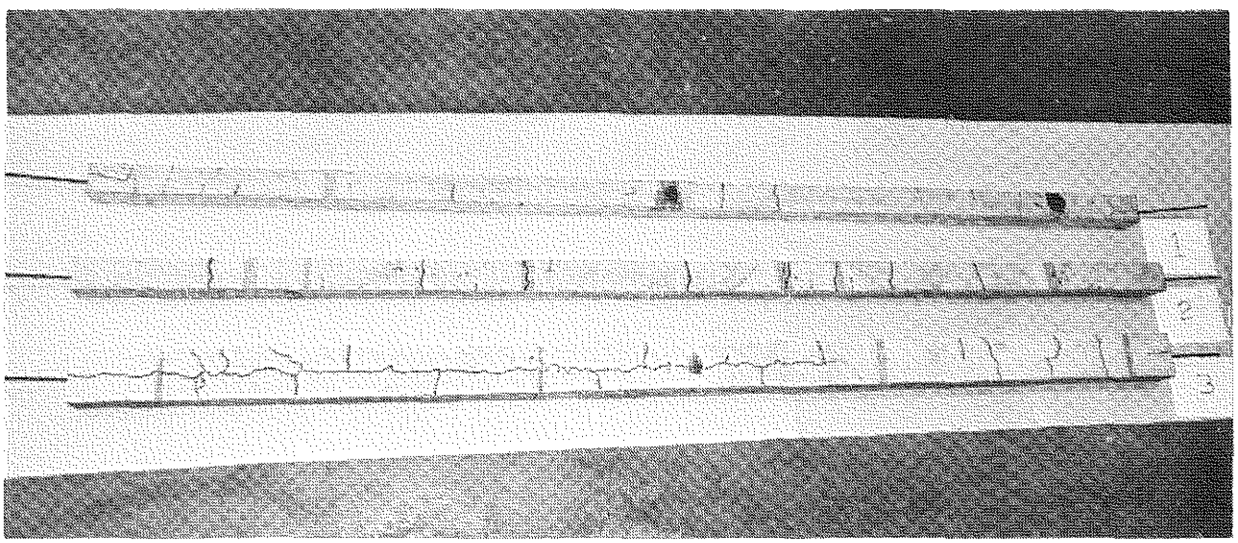


Figure 7. Test Beams Showing Cracks (Lined) Induced by Warming Reinforcing Steel (by Resistance Heating).