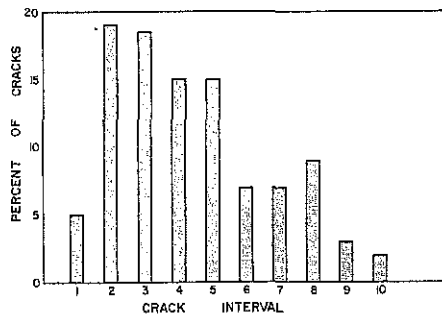RESEARCH
REPORT

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BUREAU OF HIGHWAYS
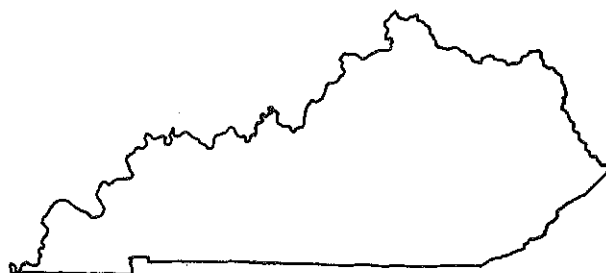
CRACKING IN CONTINUOUSLY
REINFORCED CONCRETE PAVEMENTS

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October 31, 1977

JULIAN M. CARROLL
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H.3.28

MEMORANDUM TO: G. F. Kemper
State Highway Engineer
Chairman, Research Committee

SUBJECT: Research Report No. 480; "Cracking in Continuously Reinforced Concrete Pavements;" KYP-71-28; HPR-PL-1 (11), Part III-B

Report No. 480 provides understanding of unwanted, detrimental mechanisms at work in continuously reinforced concrete pavements. These insidious, side effects interfere with the otherwise-expected long-and-enduring, perhaps maintenance-free service-life of this type of pavement.

Respectfully submitted,

Jas. H. Havens
Director of Research

gd
Enclosure
cc's: Research Committee

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16. Abstract

The objective of this study was to monitor placement and evaluate performance of continuously reinforced concrete pavement with an emphasis on determining the time of cracking and crack interval. This paper reports on the design and construction practice for CRCP in Kentucky. Data on crack frequency and time of cracking is also included in an attempt to further explain the cracking pattern associated with CRCP.

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Research Report
480

CRACKING IN CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

KYP-71-28; HPR-PL-1 (11), Part III-B

by

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Commonwealth of Kentucky

October 1977

INTRODUCTION

The first continuously reinforced concrete pavement (CRCP) was built by the Bureau of Public Roads in 1921 near Washington, D. C., on the Columbia Pike. In 1925, the Illinois Division of Highways used a continuously reinforced pavement over a section of peat bog. Another test road, which included several continuously reinforced test sections, was placed by the Indiana State Highway Commission in cooperation with the Bureau of Public Roads in 1938. Based on performance of those initial test roads, considerable interest was aroused in several highway departments, and additional experimental CRC pavements were constructed in Illinois and New Jersey in 1947 *(1)*, in California in 1949 *(2)*, and in Texas in 1951 *(3)*. By 1959, there had been about 100 miles (160 km) of equivalent, two-lane pavement constructed using CRC concepts. By 1969, the mileage of CRC pavement had increased to over 5,000 miles (8,000 km), and an increasing number of highway and airfield agencies were adopting the CRC pavement as a standard or alternative construction procedure *(4 through 16)*.

CRCP IN KENTUCKY

Approximately 7.12 miles (11.5 km) or 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 [I 71-2(15)37] (Figure 1). Paving was completed there in late 1968 *(17)*. 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 [I 275-9(23)8] were built in 1971; the outer lane and Ramp H from I 75 southbound onto I 275 westbound were completed in 1972 (Figure 2). Other ramps in the I 75 - I 275 junction were not studied. 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. Special provisions and standard drawings applicable to these projects are included in the Appendix.

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. The reinforcement could be deformed steel bars, deformed bar mats, or welded deformed wire fabric. The size, spacing, and laps were indicated on the plans for a specific job. Wide-flange beams and angles in terminal joints conformed to ASTM A 36 and were galvanized after fabrication in accordance with ASTM A 123.

Base and subgrade construction was the same as for conventional pavements. Reinforcement could be installed by pre-setting on chairs or continouous chair bars or be depressed into the fresh concrete -- that is, except when slip-forming. Transverse bars could be positioned either over or under the longitudinal bars, provided that the position was not varied and that a clearance of 3 ± 0.5 inches (76 ± 13 mm) from the top of the pavement to the steel was maintained. Concrete was placed in one lift. Finishing and curing of CRC pavement were the same as for conventional portland cement concrete construction. Transverse construction joints were installed at the end of each day's work, when work was terminated for 30 minutes or more, or where the project terminated in a free end. At construction joints, longitudinal bars were required to extend a minimum of 8 feet (2.4 m). Additional No. 5 bars, 3 feet (0.9 m) in length, were placed between the longitudinal bars at the joints. Construction joints were a minimum of 3 feet (0.9 m) beyond a lap, and all laps were a minimum of 8 feet (2.4 m) beyond construction joints. Terminal joints were a minimum of 125 feet (38 m) from any structure and were also designated where the pavement abutted another pavement. The terminal joint consisted of the necessary excavation, the sleeper slab, and the wide-flange beam and angle as then detailed in Standard Drawing RPC-020.
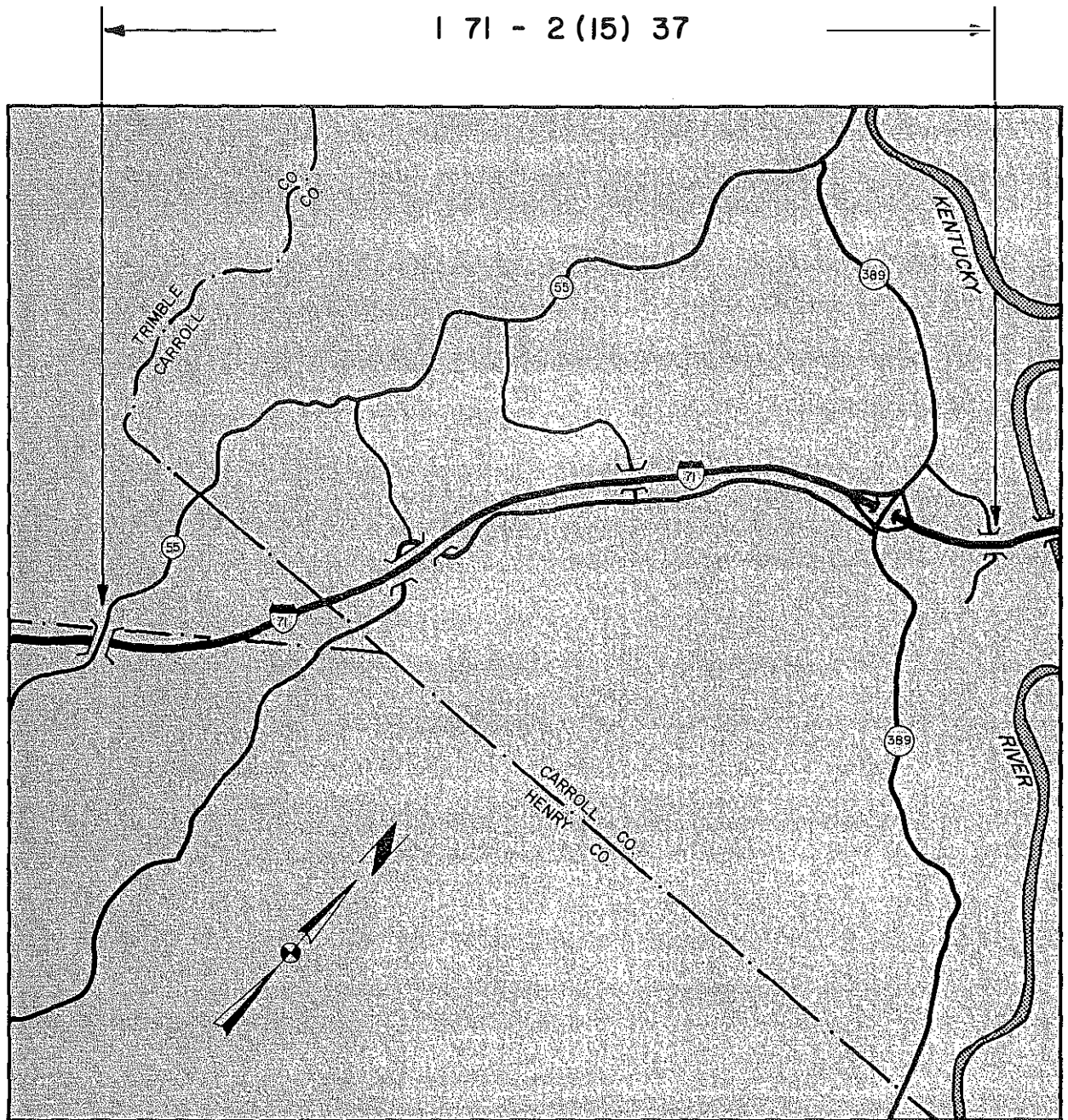
Figure 1.   Location of I 71-2(15)37. Completed in December 1968; 7.198 Miles (11.582 km), 8-inch (203-mm) Slab. Slip-Formed.
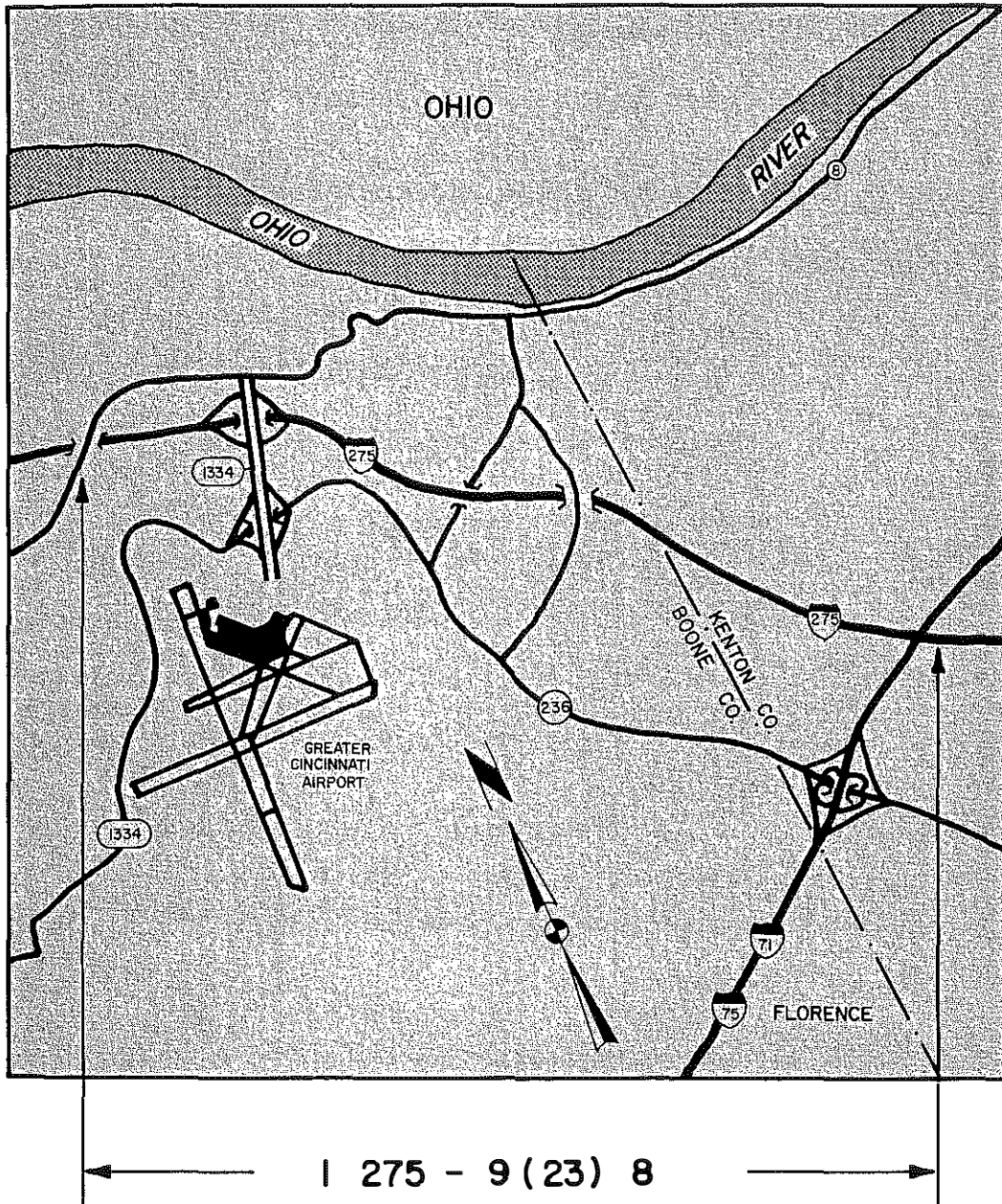
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Figure 2. Location of I 275-9(23)8. Extends 4.7 miles (7.6 km) Westward; Built in 1971; Outer Lane and Ramp Were Completed in 1972; 9-inch (229-mm) Slab.

PERFORMANCE

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.

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

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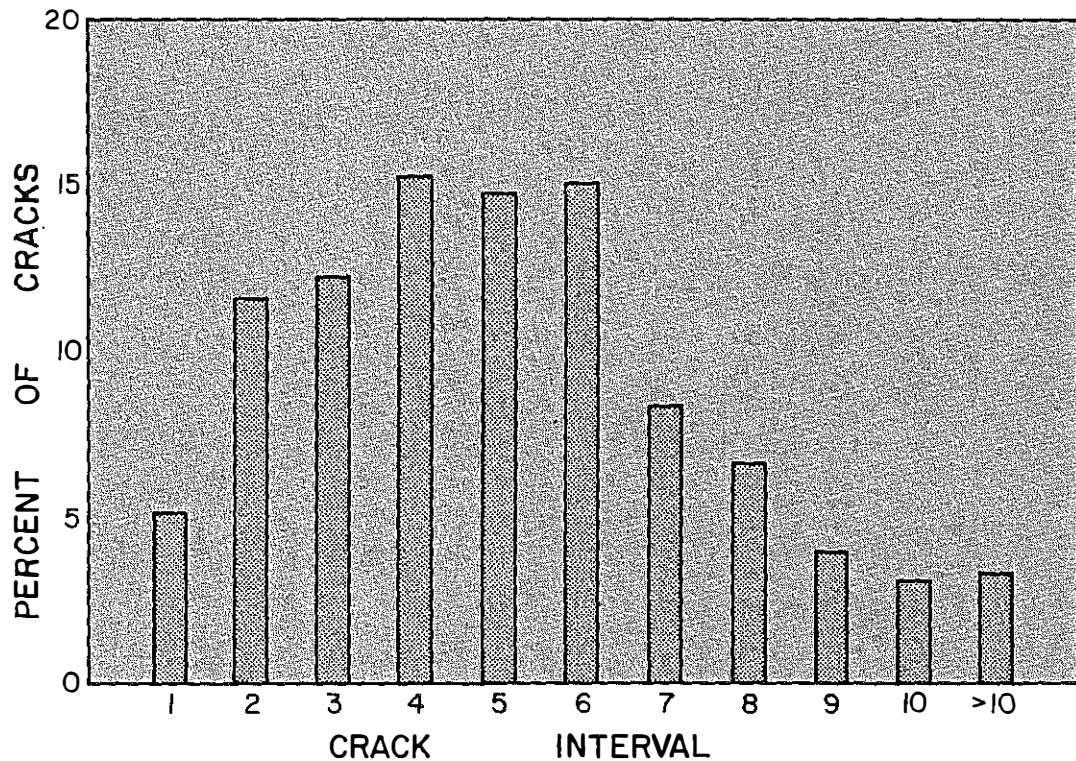
To develop a time-to-cracking relationship, surveys of ramp sections of I 275 were begun immediately after construction (April 1972) and made at short time intervals. Most of the cracking occurred within the first 2 weeks after construction; this is shown in Figure 4. The spacing between cracks on the 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 (1742 + 50) and no obvious cracking in the last 25 feet (8 m) approaching Station 1866. 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.
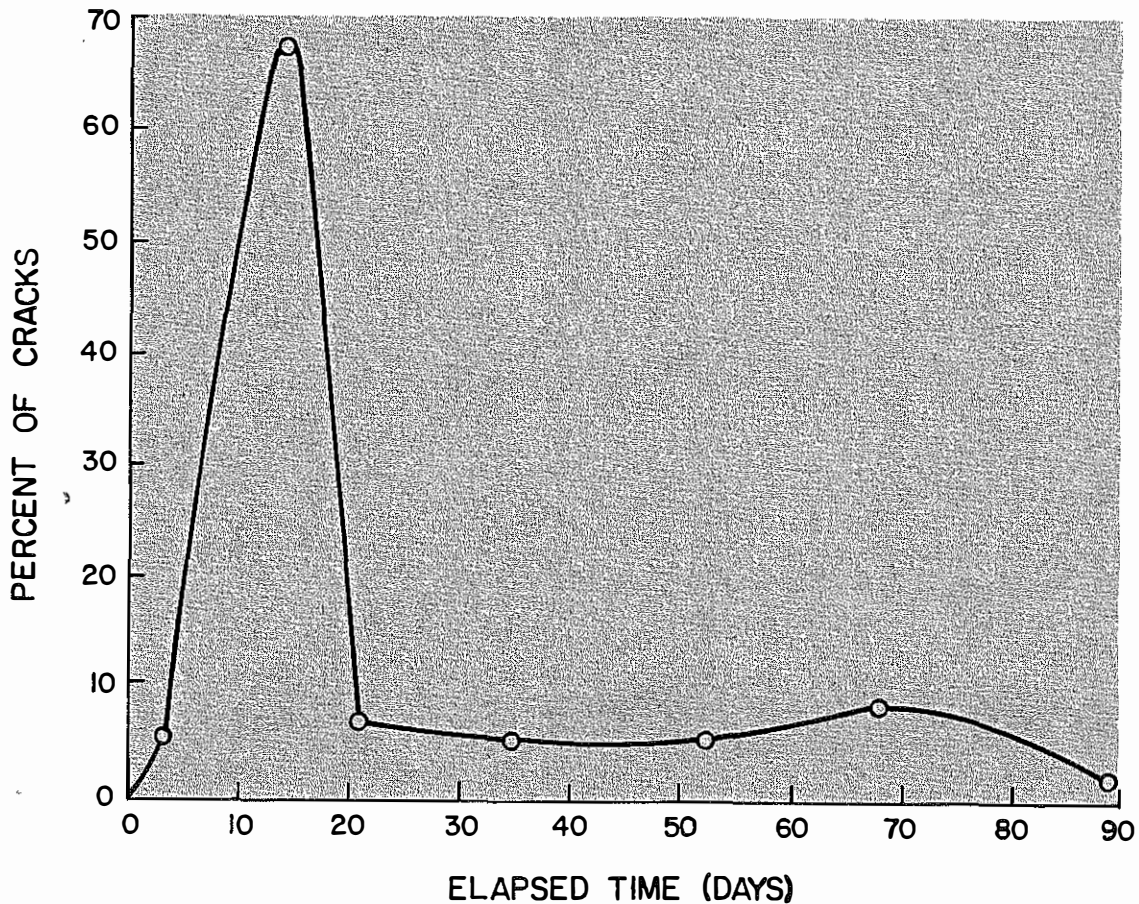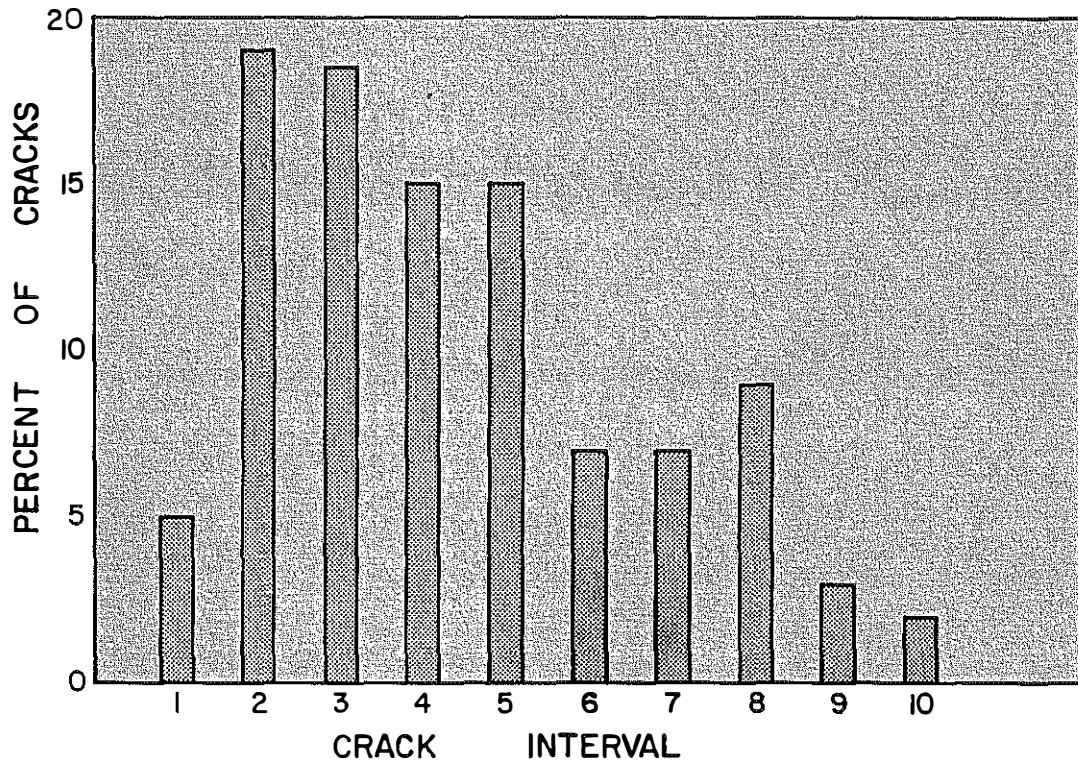

Figure 4.   Spacing between Cracks in Relation to Time (Age of Concrete); Ramp Section, I 275, Westward from Southbound I 75.

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Figure 5.   Mature Spacing of Cracks on Ramp Section; I 275, Westward from
Southbound I 75.

THEORIES OF CRACKING: PLAIN AND JOINTED CONCRETE

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 very 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, h, and width, w, having length, l, multiplied by the density (unit weight), γ_m, of the material. The force, F, may be represented by σA, and A may be defined as $h \times w$. Thus,

$$\sigma \times h \times w = \gamma_m \times h \times w \times l \times f \qquad 2$$

and

$$\sigma = \gamma_m \times l. \qquad 3$$

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,

600 lbs = 0.0833 x 12 x 1/12,
600 lbs = 1 lb/lineal ft x l, or
l = 600 lineal feet (183 m).

6

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 2 *l*. 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-feet (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 6 shows a shrinkage crack which was well-controlled by a sawed joint. Figure 7 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) is 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 (1370 and 2060 m). This is a basis for spacing expansion joints approximately one-half mile (0.8 km) (or less) apart.
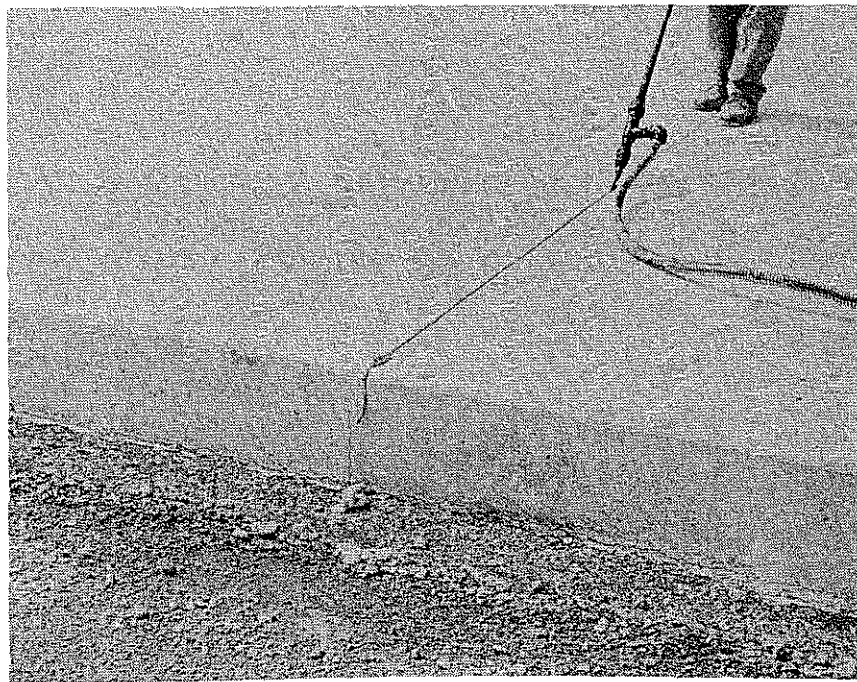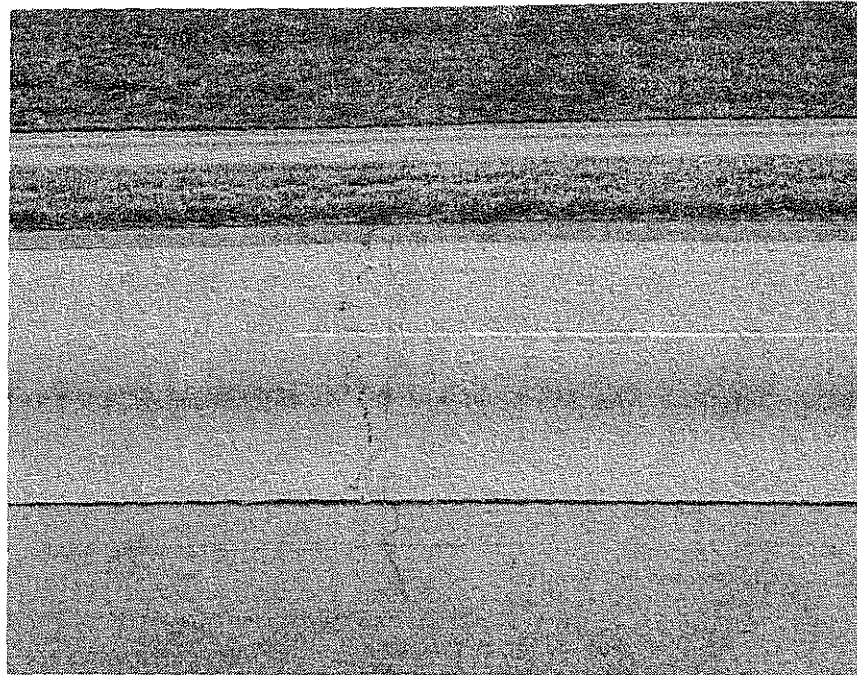

Figure 6: Normal Shrinkage Cracking, Well-controlled by Sawed Joint;
I 64, Louisville-Frankfort; Nov. 1965.

Figure 7. Shrinkage Crack. Either the joint was Sawed too Late or the Dowel Bars Locked or Froze (Misaligned).

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 mistakenly attributed, by others, to unsound aggregate and to freeze-thaw actions *(18, 19)*.

Whereas weakened-plane-type joints effectively discipline the close-interval cracking due to contraction, the pushing stresses accompanying temperature exapnsion 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.

Figure 8 shows a crack midway between contraction joints which are "frozen" or "locked". The direction of traffic is from left to right. Faulting is typically a drop-off. Figure 9 shows a sawed, doweled, contraction joint which is also faulted slightly. Each of these two cases illustrates a tendency to upthrust in the forward direction. Progressive deterioration, such as D-cracking, shattering, etc. is more likely to originate at the sawed joint because the force bears on a reduced cross section. The dowel holes in the concrete must have enlarged somehow under the forces causing faulting.

Figure 10 shows upthrusting at a full-depth concrete patch. The patch, there, is shaped like an inverted T -- that is, the granular base material was removed from under the ends of the slabs, and the whole cavity was refilled with concrete. Upthrusting is evident at the joint between the edge of the slab and the bituminous shoulder.

A cracking phenomenon not yet fully explained is shown in Figure 11. The separation at the centerline (groove sawed to one-fourth of slab thickness + 1/2 inch (13 mm)) could not have happened unless accompanied by failure of the tie bars.

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Figure 8.   Crack Midway between Contraction Joints which Are Frozen or Locked.

Figure 9.   Contraction Joint which Is also Faulted Slightly in One Lane.

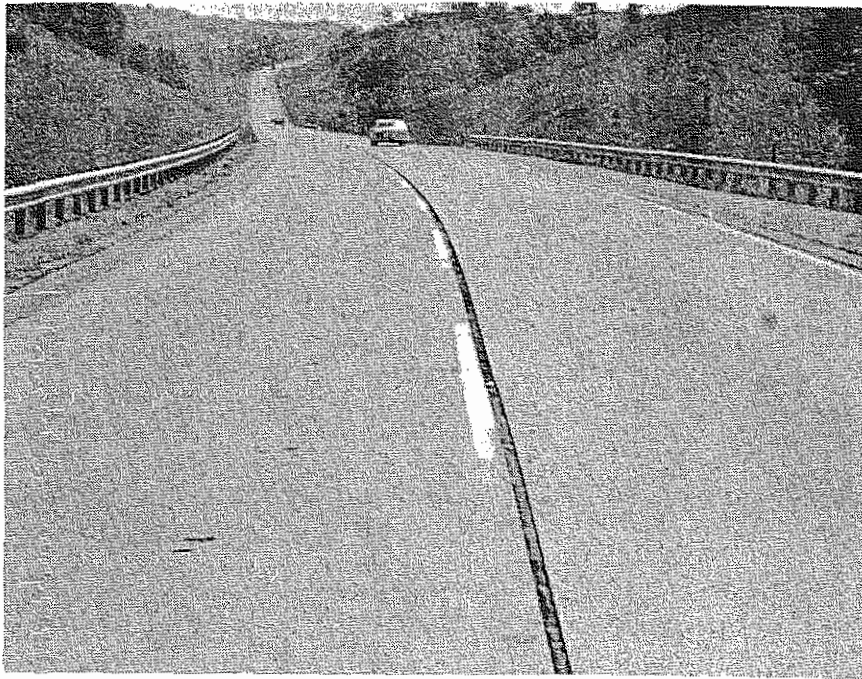Figure 10.   Upthrusting at a Full-Depth, Inverted-T Patch.

Figure 11.   Separation at a Sawed Centerline Joint; Tie-Bars Were Found Broken.

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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 of ultimate strength. 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 has 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 or under slightly different conditions. Nevertheless, fatigue tests in the compression mode have yielded typical S-N diagrams. Antrim and McLaughlin [20] found good correlation between the stress S, expressed as a percentage of compressive strength, and log N, the number of load applications; their equation for air-entrained concrete was as follows:

$$\log N = 20.501 - 0.214\ S. \qquad 4$$

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 is found (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. As a matter of historical interest to Kentucky, see Reference 21.

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. Prior history indicated that expansion joints did not prevent blowups. The joints filled with incompressible material and ceased to function, anyhow. Wooley, 1945 [22], credited this explanation to Griffin, 1943 [23].

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 1.38 times.

Assuming the coefficient of thermal expansion of concrete to be 5.5×10^{-6} per degree F (9.9×10^{-6} per degree C), and the modulus of elasticity to be 5×10^{6} psi (34.5 GPa), the stress rise, when fully constrained, would be: $C_s\ \Delta T\ E_s$, or 2,750 psi (19.0 MPa). The stress concentrated on the bearing area would be 2,750 x 1.38, 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 [18, 19] 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 [24]. Severe D-cracking on an airport apron is shown in Figure 12. Elsewhere, cracking and shear occurred at all joints, regardless of direction, and enlarged at corners; this is illustrated in Figure 13.

11

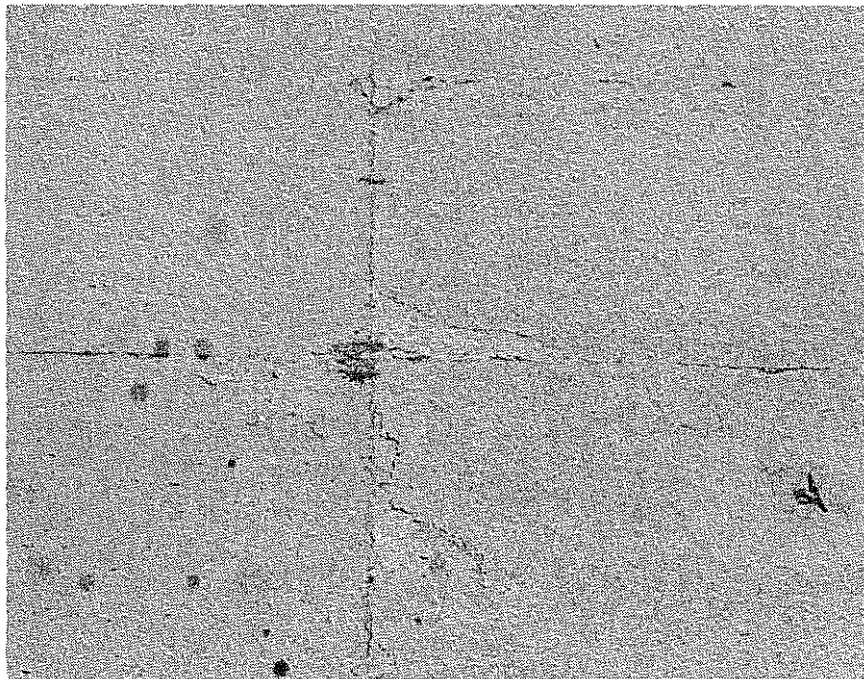Figure 12.   Severe D-Cracking; Airport Apron.

Figure 13.   Deterioration at Interior Corners; Airport Apron.

12

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 compression material in the joint should provide enough resistance to closure to push back and restore several slabs in each direction, from a given joints, 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.

THEORIES OF CRACKING: 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 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,

$$C_s \Delta T + \Delta\sigma_s/E_s = C_c \Delta T + \Delta\sigma_c/E_c \qquad 5$$

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

$$\Delta\sigma_s = -\Delta\sigma_c/A_s \qquad 6$$

where A_s is the area of steel per unit of cross-sectional area of pavement. Substituting Equation 6 into Equation 5 and integrating with respect to T, the stress rise per unit length is found to be

$$\Delta\sigma_c = 1/[(1/E_sA_s) + (1/E_c)] \; (C_s - C_c) \atop (T_2 - T_1). \qquad 7$$

Using Equation 6, the stress rise in the steel is

$$\Delta\sigma_s = 1/[(A_s/E_c) + (1/E_s)] \; (C_s - C_c) \atop (T_2 - T_1). \qquad 8$$

The total stress rise, σ, can be found from

$$\sigma = \Delta\sigma \; \Delta L, \qquad 9$$

where ΔL is the length of pavement under consideration. Substituting Equation 9 into Equations 7 and 8 and integrating with respect to L, it is found that

$$L_c = [1/(C_s - C_c)] \; [1/(T_2 - T_1)] \atop [(1/E_c) + (1/E_sA_s)]\sigma_c \qquad 10$$

and

$$L_s = [1/C_s - C_c)] \; [1/T_2 - T_1)] \; [(A_s/E_c) + (1/E_s)]\sigma_s. \qquad 11$$

Equations 10 and 11 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 $T_2 - T_1$ is used as an approximation of the effective, seasonal change in temperature. Estimates of parameters are as follows:

$$
\begin{aligned}
C_c &= 5.5 \times 10^{-6}/°F \ (9.9 \times 10^{-6}/°C), \\
C_s &= 6.5 \times 10^{-6}/°F \ (11.7 \times 10^{-6}/°C), \\
E_c &= 5 \times 10^6 \text{ psi } (34.5 \text{ GPa}), \\
E_s &= 30 \times 10^6 \text{ psi } (206.8 \text{ GPa}), \\
A_s &= 0.00677, \text{ and} \\
T_2 - T_1 &= 100°F \ (56°C).
\end{aligned}
$$

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 $\Delta T = 100°F$ (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

$$
\sigma_{sy}A_s = \mu \times \Sigma \times l_s \qquad \qquad 12
$$

where
$$
\begin{aligned}
\mu &= \text{bond strength needed (psi)} \\
\mu &= \sigma_{sy} \times D_s/4l_s, \\
\Sigma &= \text{perimeter of bar (in.)}, \\
\sigma_{sy} &\times \pi D^2_s/r = \mu \times \pi D_s, \times l_s, \text{ and} \\
D_s &= \text{diameter of bar (in.)}.
\end{aligned}
$$

When x_{sy} is 60,000 psi (410 MPa), D_s is 0.625 inch (16 mm), and l_s is 40 diameters, $\mu = 375$ psi (2.6 MPa). When $\sigma_{sy} = 90,000$ psi (620 MPa), D_s is 0.625 inch (1.6 mm), and l_s is 40 diameters, $\mu = 625$ psi (4.3 MPa). Forty diameters is a rule-of-thumb figure. Contrariwise, 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. Bond strength, or resistance to slip, was called the "slip modulus" by Shrader (25).

APPLICATION OF DRAG THEORY TO CONTINUOUSLY REINFORCED PAVEMENTS

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.

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

$$l_{(ft)} = \sigma_{sy} \times \text{percent steel}/100. \quad 13$$

When $\sigma_{sy} = 60,000$ 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.

Two mechanisms superimpose at the end. 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.

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. This is illustrated in Figure 14. 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. Figures 14 and 15, reprinted from the Madison Courier, May 21, 1977, shows a case which evolved from a previous partial-width patch in the outer edge of the pavement.
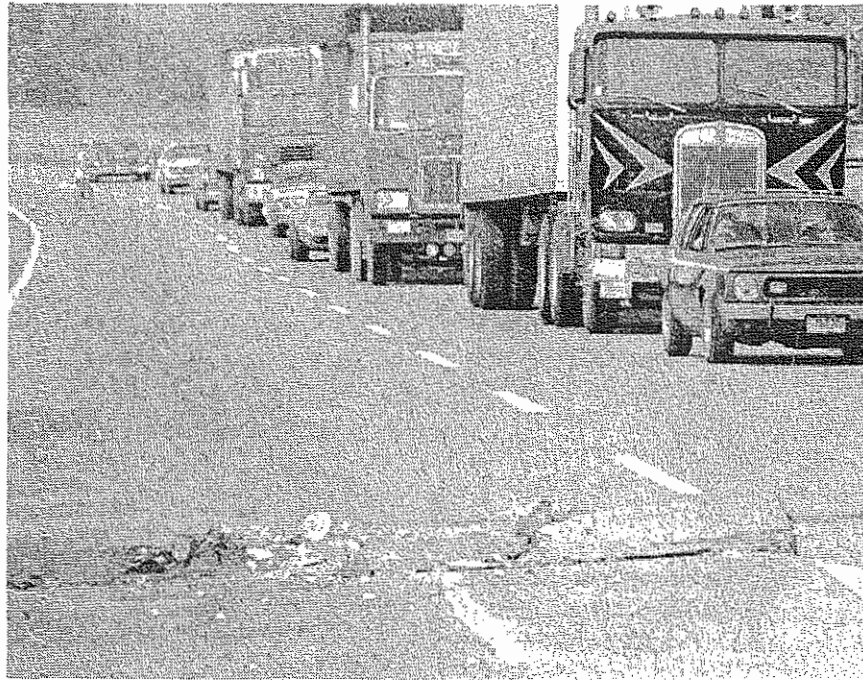
Figure 14.   Blowups or Buckling at Partial-Width Patch; I 71, near Carrollton (Madison Courier; May 21, 1977).

Figure 15.   Different View of Failure Shown in Figure 14 (Madison Courier; May 21, 1977).

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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 (26), 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. 3 re-bar (Grade 40, f_y = 50,900 psi (351 MPa)) (Figure 16) 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, cracking of concrete, yielding of steel, or mere slip, thermocouples were installed in the concrete at various depths; and 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 of the re-bar. 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 black in Figure 17, Beam 1.

A second beam was made using Grade 60 steel (f_y = 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 17, 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 f_y 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 17, 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 the 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 800 to 1,000 feet (240 to 300 m).

NO. 5 DEFORMED STEEL BAR

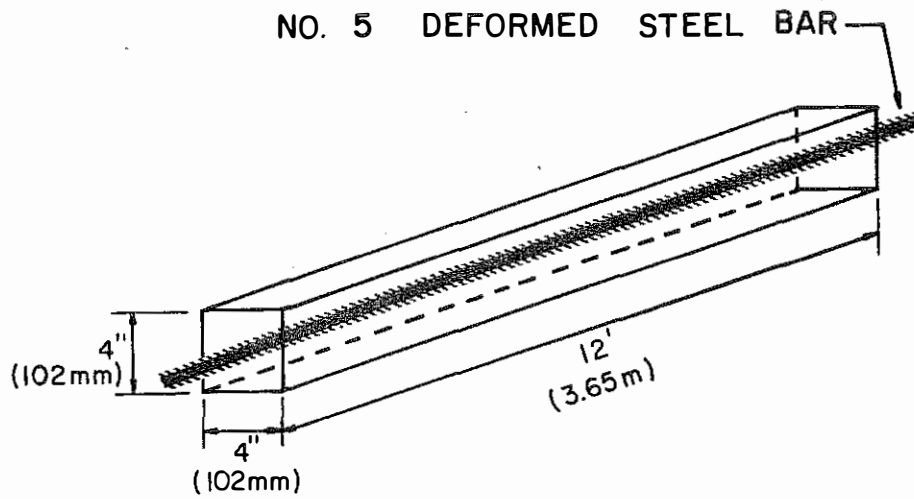4" (102mm)

4" (102mm)

12' (3.65m)

Figure 16.   Test Specimen; Beam Used to Simulate Crackling Conditions in Reinforced Concrete.
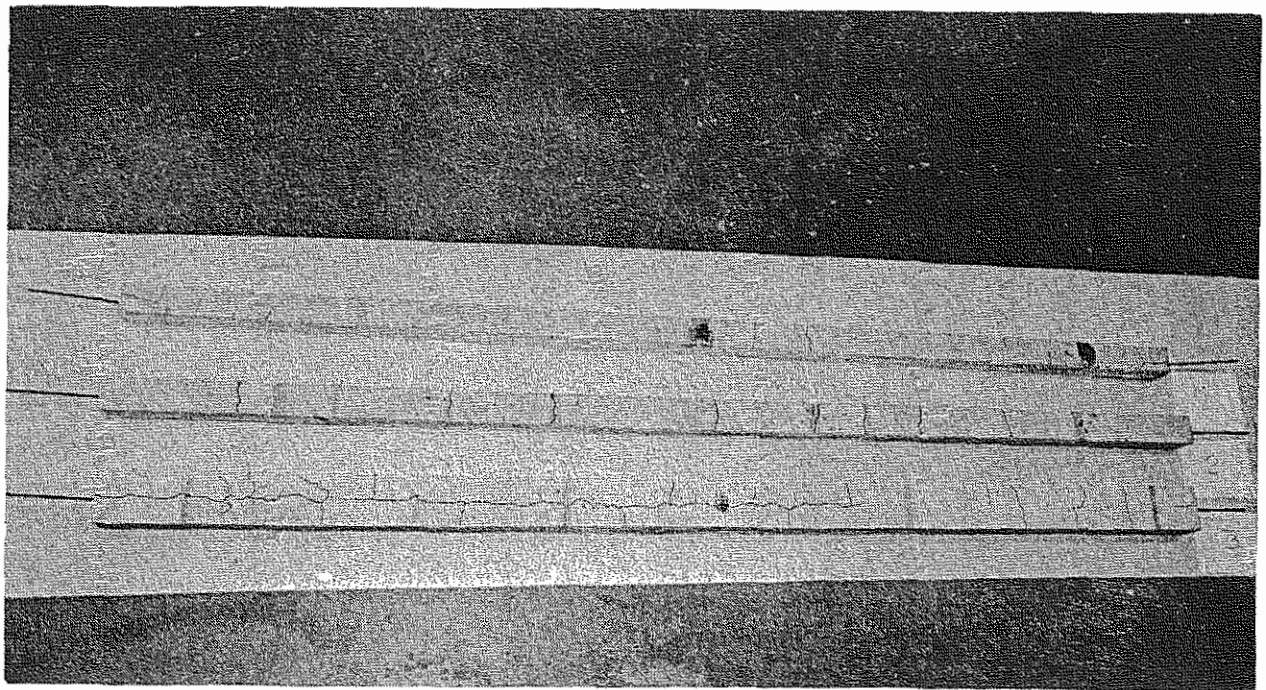

Figure 17.   Test Beams Showing Cracks (Lined) Induced by Warming Reinforcing Steel (by Resistance Heating).

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KENTUCKY DEPARTMENT OF HIGHWAYS
SPECIAL PROVISION NO. 63-B
CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

This Special Provision shall be applicable only when indicated in the plans or proposal; and shall supersede all conflicting requirements of the Department's Standard Specifications for Road and Bridge Construction, other Special Provisions, the plans, and the Standard Drawings. Article and Section references herein are to the Department's 1965 Standard Specifications.

I. DESCRIPTION

This work shall consist of a continuously reinforced portland cement concrete pavement, constructed on a prepared base of dense-graded aggregate in conformity with the established line, grade, thickness, and typical cross-sections shown on the plans and in accordance with Section 307, and applicable current Special Provisions, and Standard Drawings, except as modified herein.

II. MATERIALS

All materials incorporated into the work shall be sampled and tested in accordance with the Department's Manual of Field Sampling and Testing Practices, except that the reinforcing steel may be sampled and tested at any time or place, either prior to or after delivery to the project, at the option of the Department.

In addition to the requirements of Article 307.2.0 and current applicable Special Provisions, various materials shall comply with the following requirements.

A. Requirements.
1. Coarse Aggregate. The coarse aggregate shall be Size No. 57.
2. Reinforcing Steel. The reinforcing steel shall consist of deformed steel bars or bar mats, or welded deformed wire fabric. The type selected shall be used throughout the entire length of the project. The length of the pieces of longitudinal reinforcing steel shall be 16 feet or more. The size, spacing, and laps shall be as indicated in the plans and herein.

If bar mats or sheets of wire fabric are used as the reinforcing steel, the Contractor may install deformed bars as the reinforcement in tapered portions of ramps and other irregularly shaped areas in order to avoid cutting the mats or sheets. If the deformed bars are used in portions of the irregularly shaped areas, they shall be lapped a minimum of 24" with the mats or sheets and securely tied as directed by the Engineer.

a. Deformed Steel Reinforcing Bars. Deformed bars for longitudinal reinforcement shall be Grade 60 conforming to one of the following specifications.
(1) Billet Steel Bars   ASTM A615
(2) Rail Steel Bars   ASTM A616
(3) Axle Steel Bars   ASTM A617
(Transverse bars may be either Grade 40 or Grade 60 conforming to the above specifications.)

Where the tie bars are bent back and subsequently straightened in the construction of partial-width pavement and field bending is required, the tie bars shall be Grade 40 deformed billet steel or axle steel.

b. Bar Mats. Bar mats shall conform to ASTM A184 with the following restriction : The mats shall be fabricated by welding suitable Grade 60 deformed steel bars. (Transverse bars may be either Grade 40 or Grade 60.)

c. Welded Deformed Steel Wire Fabric. Welded deformed steel wire fabric for concrete reinforcement shall conform to the requirements of ASTM A497; except that all wire used in the fabric shall conform to ASTM A496.

3. Terminal Joint Steel. The wide flange beam and the angle used in the terminal joint shall conform to ASTM A36 and shall be galvanized after fabrication in accordance with ASTM A123.

III. CONSTRUCTION METHODS

The construction of the continuously reinforced pavement shall conform to the requirements of Article 307.3.0 and of current applicable Special Provisions, except as provided herein.

A. Placement of Reinforcement. At the time concrete is placed, the reinforcement shall be free from loose-thick rust, dirt, oil, grease, paint, or other coatings that would materially reduce the bond and shall be handled with such care that bar mats or fabric sheets, if used, will remain reasonably flat and free from distortions. Loose bars shall be free from all kinks or bends that prevent them from being properly assembled or installed. Forms shall be oiled prior to placement of reinforcement. The transverse steel bars may be positioned either under or over the longitudinal bars, provided that the position selected is not deviated from and that the specified clearance from the top of the pavement to the steel is maintained.

Regardless of the method of placement used, the horizontal spacing tolerances shall be 1/2 inch for the longitudinal steel and 2 inches for the transverse steel. These tolerances notwithstanding, the number of longitudinal members for a given width of pavement and the number of transverse members per station shall be as shown on the plans.

Reinforcement may be installed by one of the following methods: (1) pre-set on high chairs or continuous high-chair bars or (2) placed by mechanical means, except when slip-form paving is used.

Method 1. Reinforcement Pre-Set on High Chairs or High-Chair Bars. The high chairs or high-chair bars must be approved by the Engineer prior to being used and shall be manufactured to a constant height, with a maximum variation of plus or minus 1/8 inch. The chairs or bars shall have sufficient bearing area at their bases to prevent overturning or penetration into the base. They shall be designed so as not to impede the placing and consolidating of the concrete.

The high chairs or high-chair bars shall be arranged and spaced so that the bottom bars of the reinforcement will be supported in the proper positions to prevent permanent deflection or displacement from occurring during the placing and consolidating of the concrete. The longitudinal spacing of either shall not exceed 4 feet, and the transverse spacing of chairs shall not exceed 3 feet. High-chair bars shall not be set so close to any transverse bar as to make placing concrete through the space difficult. This is particularly important in lapped areas where there is a concentration of reinforcement. Welding of high chairs to the longitudinal bars of the reinforcement will not be permitted.

The Contractor shall submit a sample of the high chair or high-chair bar he proposes to furnish. The chairs or bars and the layout shall be subject to the approval of the Engineer. If the support system does not maintain the reinforcement in the position required by these specifications during placing and finishing of the concrete, the Contractor will be required to increase the number of supports or take such other steps as may be required to assure proper position of the steel. When the reinforcement consists of loose bars assembled on the project, the longitudinal bars shall be secured to the transverse bars by wire ties or clips at the locations shown on the plans. The spacing of the bars shall be maintained within the horizontal and vertical tolerances specified herein and in the plans.

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Method 2. Reinforcement Placed by Mechanical
Means. The reinforcement shall be tied as shown
on the plans and as required by this Special Provision
prior to being deposited on the surface of the concrete.
The methods and equipment used to depress the reinforce-
ment into the concrete shall not weaken or otherwise alter
the concrete, or the laps, joints, or the tied intersections
of the longitudinal and the transverse bars.

The reinforcement shall be embedded to the proper
depth in the concrete by means of a machine approved by
the Engineer. Embedment of the reinforcing steel shall
be performed in such a manner that the concrete shall
show no segregation attributable to the embedment opera-
tion. The machine shall be operated in such a manner
that it does not drag the reinforcement.

If the machine shows any tendency to displace the
reinforcement longitudinally, the Contractor, with the
approval of the Engineer and at no additional cost to the
Department, shall (1) tie the laps more securely if it can
be demonstrated that this will prevent the displacement
and assure the proper lap, or (2) halt paving operations
until the machine is properly adjusted or another machine
is furnished which will operate in a satisfactory manner,
or (3) install the reinforcement by Method 1.

The machine shall be operated in such a manner that
the concrete will not be excessively vibrated or otherwise
manipulated. If the machine employs a grid-type depressor
which operates intermittently to depress and vibrate the
reinforcement, vibration shall be of the shortest duration
that will assure that the reinforcement reaches its
specified depth. When the machine embeds the reinforce-
ment progressively through the action of a series of
oscillating bars as the machine moves forward, oscilla-
tions shall be discontinued whenever the forward movement
of the machine is stopped.

B. Lap Splices in Reinforcing Steel. Reinforcing
bars, bar mats, or welded wire fabric used as continuous
reinforcement shall be lapped in a staggered pattern and
securely tied or otherwise fastened as shown on the plans,
unless the Engineer authorizes an increase or decrease in
the number of ties or fasteners.

C. Placement of Concrete. Concrete shall be
placed in one lift on the base and struck off to full depth by
the method prescribed by the Standard Specifications.

In the event the Contractor elects to construct the
continuously reinforced cement concrete pavement with a
slip-form paver as set out in the current Special Pro-
vision No. 39 for Slip-Form Paving, the pavement rein-
forcing steel shall be furnished and placed as described
herein, except the use of vibratory equipment for the
placing of such reinforcement will not be permitted.

The equipment used for consolidation shall include
approved vibratory equipment having the frequency and
intensity sufficient to satisfactorily vibrate and consolidate
the concrete for its full width. Additional consolidation
and vibration may be required at laps and shall be required
at terminal joints as directed by the Engineer.

D. Transverse Construction Joints. A trans-
verse construction joint shall be installed at the end of
each day's work or whenever paving operations are inter-
rupted for more than 30 minutes. The joint shall be formed
by placing the concrete against a split header approved by
the Engineer. The split header shall be set perpendicu-
larly to the centerline of the pavement. The longitudinal
reinforcing steel shall extend through the split header and
shall be properly supported from the subgrade beyond the
split header to prevent undue deflections.

The steel reinforcement which extends beyond the
split header shall be covered with sheets of plywood or
other material to permit workmen to walk on the steel
without displacing it and to prevent any concrete from
spilling on the base during the completion of the joint.

The construction joint shall be strengthened by the

addition of supplementary deformed bars placed symmetri-
cally with the joint and at a uniform spacing along the joint
as shown on the plans.

No lap splices will be permitted within 8 feet beyond the
construction joint in the direction of paving, or within 3
feet back of the construction joint. Lap joints shall be
made at the locations and in the manner shown on the plans.

E. Terminal Joints. The terminal joints shall be con-
structed at the locations and to the details shown on the
plans. The terminal joint shall consist of the necessary
excavation, the sleeper slab, the wide flange beam and
angle, and the bond breaker, etc. as detailed. The
portion of the sleeper slab on which the bond breaker is to
be placed shall be finished with a metal finishing tool to a
smooth surface.

F. Final Finishing. The surface of the pavement
shall be finished in accordance with Article 307.3.10 of
the Standard Specifications. Extreme care shall be exer-
cised to prevent honeycomb. If necessary, internal vibra-
tion shall be employed adjacent to lapped bars to assure
proper consolidation.

IV. METHOD OF MEASUREMENT

Continuously Reinforced Concrete Pavement will
be measured in the manner specified in Article 307.4.0
of the Standard Specifications.
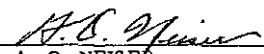
Terminal joints for Continuously Reinforced Con-
crete Pavement will be measured in linear feet to the
nearest one foot of joint measured perpendicularly to the
centerline of the pavement as shown on the plans.

V. BASIS OF PAYMENT

Accepted continuously reinforced concrete pave-
ment will be paid for in accordance with Article 307.5.0
of the Standard Specifications at the contract unit price
per square yard for "Continuously Reinforced Concrete
Pavement."

Accepted terminal joints will be paid for at the
contract unit price per linear foot for "Terminal Joint,"
which payment shall be full compensation for all excava-
tion, materials, labor, and equipment necessary to com-
plete the joint.

APPROVED June 16, 1970

A. O. NEISER
STATE HIGHWAY ENGINEER

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SPECIAL PROVISION NO. 39-C
SLIP-FORM PAVING

This Special Provision shall be applicable when indicated on the plans or in the proposal; and supersedes any conflicting requirements of the Department's Standard Specifications for Road and Bridge Construction. Section and Article references herein are to the Standard Specifications.

I. GENERAL

Except as otherwise provided herein, cement concrete pavements or bases shall be constructed in accordance with the Standard Specifications, plans, and Standard Drawings. The pavements or bases may be constructed with the use of approved slip-form pavers and approved complementary spreading and finishing equipment in lieu of the spreading, consolidating and finishing equipment specified in Section 307, except that "Belting" and "Final Finish with Burlap Drag" shall be required.

II. MATERIALS

The load-transfer assemblies shall be modified at the ends to allow for approximately 4 inches of clearance between the assemblies and the slip-forms by welding the outer leg of the chair at an angle of approximately 90 degrees with the upper and lower spacer bars.

The maximum slump of the concrete shall be 1½ inches, with a plus ½ inch tolerance being allowed.

III. CONSTRUCTION METHODS

A. Subgrade and Aggregate Base Course. The subgrade and aggregate base course shall be constructed in accordance with all applicable requirements of Article 307.3.5, and a fine grading machine shall be required to prepare the proper grade for the paving equipment to the accuracy necessary to ensure that the finished concrete pavement or base will be in conformity with the tolerances set forth in the Standard Specifications.

If any traffic is allowed to use the prepared aggregate base course, the course shall be rechecked and corrected prior to placing the concrete.

B. Paver. The slip-form paver shall be an approved self-propelled type designed to spread, consolidate, and finish the concrete, in one complete pass of the paver, in such a manner that a minimum of hand finishing will be necessary. The paver shall consolidate the concrete in a manner so as to not damage or displace the load-transfer devices and the steel reinforcement, and shall finish the concrete so as to produce a smooth, uniformly textured surface with the specified crown and slope ready for the final finishing.

C. Paving. The Contractor shall not operate a mixing plant inside the area to be traversed by the slip-form paver. The concrete shall be held to a uniform consistency so that unacceptable slumping will not occur at the edge of the pavement. The paver shall be operated with as nearly a continuous forward movement as possible, and, when forward movement is stopped, the consolidating devices of the paver shall be stopped immediately.

If the concrete is placed in two operations, the width of the first placement may be reduced a maximum of 3 inches on each side.

Methods approved by the Engineer shall be used to construct longitudinal construction joints in accordance with the Standard Specifications, plans, Standard Drawings, and as specified herein. Fixed forms shall be used to prevent any slump from occurring where the edge is to be abutted by subsequent pavement. The fixed forms shall be of adequate length, depth, and thickness to protect and maintain the pavement edge in correct alignment and depth, and shall be staked firmly in place with metal stakes. Where the edge is not to be abutted by subsequent pavement, edge slump not in excess of ¼ inch will be permitted. Straightedge tolerances as required in Article 307.3.12 shall apply to that portion of the pavement bounded by lines 6 inches from the edges of the pavement. Edge slump will be measured with a 10-foot straightedge laid on the pavement perpendicular to the edge.

The Contractor shall utilize proper and adequate methods to prevent unacceptable slumping and to correct unavoidable slumping of the pavement edges due to all causes, in a manner approved by the Engineer, and shall have available at all times, the necessary materials, equipment, and labor to provide immediate protection of the edges and surface of the unhardened concrete against adverse weather conditions.

When concrete is being placed adjacent to an existing pavement, that part of the equipment which is supported on the existing pavement shall be equipped, or supplemented with approved protective materials, so as to not damage the surface of the pavement and shall be positioned at a sufficient distance from the edge of the existing pavement to avoid damaging the edge.

APPROVED *September 17, 1968*

A. O. NEISER
STATE HIGHWAY ENGINEER

WELDED DEFORMED STEEL WIRE FABRIC

FABRICATED STEEL BAR MAT

LOOSE BAR CONSTRUCTION

Steel requirements per lane width: 2.6 No.5 bars = 8.06 Sq. in. = approx. 0.70% of the end area of a 8" x 12'-0" Slab.

DETAIL FOR ONE MAT PER LANE WIDTH
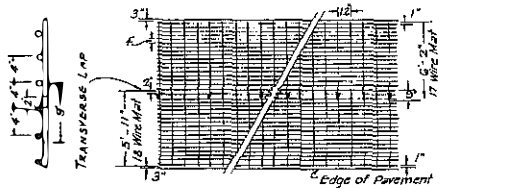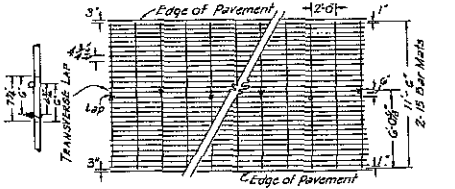
DETAIL FOR TWO MATS PER LANE WIDTH
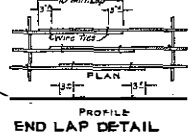
END LAP DETAIL

TRANSVERSE CONSTRUCTION JOINT USING DEFORMED BARS

NOTE:- Where the project terminates with a free end, a transverse construction joint shall be constructed as shown above for future additional construction.

~NOTES~

① The Contractor shall use the double lap requirement of the longitudinal steel at the first lap past the construction joint as shown on the plans, or he may add at least one, as long at two times the spacing of the longitudinal steel, at the first lap past this construction joint. Each additional bar shall be securely tied to the longitudinal bars on each side of the joint.

② When the normal pavement width exceeds 24', except at ramps, a longitudinal sawed construction joint shall be constructed in accordance with Standard Drawing RP5-010 current edition.
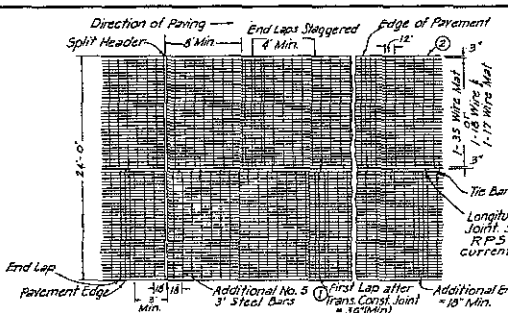
TYPICAL SAWED JOINT SECTION

KENTUCKY BUREAU OF HIGHWAYS
STEEL REINFORCEMENT FOR 8" THICK CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

STANDARD DRAWING No. RPC-001

WELDED DEFORMED STEEL WIRE FABRIC

Direction of Paving — End Laps Staggered — Edge of Pavement
Split Header — 8' Min. — 4' Min.

Steel requirements per lane width: 35 (D-260)
Wires = 9.10 Sq. In. = Approx. 0.70% of the end area of a 9" x 12'-0" Slab.

Tie Bars
Longitudinal Sawed Joint. See Std. Dwg. RPS-010 current edition.

End Lap
Pavement Edge — Additional No. 5 3' Steel Bars — First Lap after Trans. Const. Joint = 36"(Min) — Additional End Laps = 18" Min.

DETAIL FOR ONE MAT PER LANE WIDTH
D-5.2 Wires on 12" Ctrs.
Edge of Pavement

DETAIL FOR TWO MATS PER LANE WIDTH
Edge of Pavement

PLAN   PROFILE
A = 5" ± ½" = Normal Steel Position
B = 4½" Max @ End Lap Position

END LAP DETAIL

FABRICATED STEEL BAR MAT

Direction of Paving — End Laps Staggered — Additional End Laps = 18" Min.
Split Header — 8' Min. — 4' Min. — Edge of Pavement

Steel requirements per lane width: 90 No. 5 Bars = 9.21 Sq. In. = Approx. 0.70% of the end area of a 9" x 12'-0" Slab.

Note: Bars shall be placed in same plane. Staggering in profile view is for illustration only.

Tie Bars
Longitudinal Sawed Joint See Std. Dwg RPS-010 current edition.
Pavement Edge

End Lap — Additional #5 3' Steel Bars — First Lap after Transverse Construction Joint = 36"(Min)

DETAIL FOR ONE MAT PER LANE WIDTH
No. 3 Bars on 24" Ctrs.
Edge of Pavement

DETAIL FOR TWO MATS PER LANE WIDTH
Edge of Pavement

PLAN   PROFILE
Wire Ties

END LAP DETAIL

LOOSE BAR CONSTRUCTION

Direction of Paving —
Split Header — 8' Min. — 12'-0" — First Lap 36"(Min) — Additional Laps = 18"(Min) — Pavement Edge

Steel requirements per lane width: 30 No. 5 Bars = 9.21 Sq. In. = Approx. 0.70% of the end area of a 9" x 12'-0" Slab.

Tie Bars
Longitudinal Sawed Joint See Std. Dwg. RPS-010 current edition.

End Lap — Additional #5 3' Steel Bars — Lap Slope shown -1½:1 No flatter than 3:1

Additional #5 Bars at alternate longitudinal bars to be placed in plane of longitudinal bars.

END LAP DETAIL
Wire Ties
PLAN   PROFILE

TRANSVERSE CONSTRUCTION JOINT USING DEFORMED BARS
End of Days Pour
Bar Support   Split Header   First Lap 36"

NOTE: Where the project terminates with a free end, a transverse construction joint shall be constructed as shown above for future additional construction.

~NOTES~

① The Contractor shall use the double lap requirement of the longitudinal steel at the first lap past the construction joint as shown on the plans, or he may add #5 Bars, 6' long, at two times the spacing of the longitudinal steel at the first 18" lap past the construction joint. Each additional bar shall be securely tied to the longitudinal bars on each side of the joint.

② When the normal pavement width exceeds 24', except at ramps, a longitudinal sawed construction joint shall be constructed in accordance with Standard Drwg. RPS-010 current edition.

TYPICAL SAWED JOINT SECTION
3" with a tolerance of ± ½" to the longitudinal or transverse steel, whichever is on top.
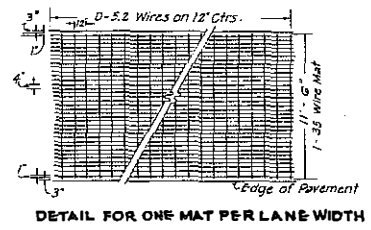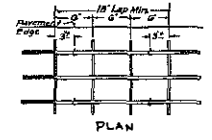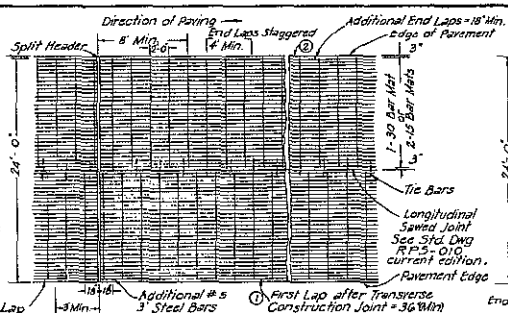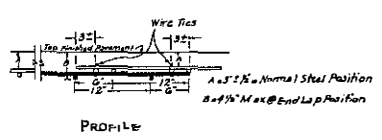Longitudinal Sawed Joint
Tie Bar

KENTUCKY BUREAU OF HIGHWAYS

STEEL REINFORCEMENT FOR 9" THICK CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

STANDARD DRAWING No. RPC-002

SUBMITTED _____ DIRECTOR DIVISION OF DESIGN   8-20-71 DATE

APPROVED _____ STATE HIGHWAY ENGINEER   8-30-71 DATE

ENTRANCE RAMP

EXIT RAMP

DETAIL A

Cont. Reinf. Conc. Pvmt ———

125' Min. Dist. to Structure
Conventional Jointed
Reinf. Conc. Pvmt.

Pavement Edge

⑦ ₵ Station
Terminal Jt.

End Cover (See Detail)

A

1" Groove

2½" x 2½" x ½" ∠

12 #4 Longitudinal
Bars Equally Spaced
Per Lane Width across
Sleeper Slab (1 Piece
Bent Bars)

Transverse Bars
not shown.

2"

End Cover

10'-0"   5'-0"

HALF PLAN VIEW OF TERMINAL JOINT
(Not To Scale)

Pavement Width

C.R.C. Pvmt. Thickness

2½" x 2½" x ½" Steel Angle
1" Groove, Hot Poured Joint Sealer

₵ Station Terminal Joint ⑦

Welds

20°   20°

Styrofoam Filler Material

Longitudinal Steel

1½"

Continuously Reinforced
Concrete Pavement

Conventional Jointed
Reinf. Conc. Pvmt.

Steel Trowel top of
Sleeper Slab

Mylar Bond Breaker or
Polyethylene (6 mills Thickness)

Wood Float Finish

Edge
Sleeper
Slab

① 12" Wide Flange Beam
@ 58 lbs./lin. ft.

1"   2"   2"

12 #4 Longitudinal Bars, Equally Spaced (Per Lane Width)
29 #5 Transverse Bars @ 8" C.-C.

DETAIL "A"

36'-0"
12'-0"   12'-0"   12'-0"
³⁄₁₆":1'   ³⁄₁₆":1'

TANGENT

(See Plans For
Pavement Width)

CURVE

TERMINAL JOINT CROSS SLOPES ⑧

Pay Limits of Continuously Reinforced Concrete Pavement

125'-0" Minimum Distance to Structure - Conventional Jointed Reinforced Concrete Pavement
Pay Limits of Conventional Jointed Reinforced Concrete Pavement

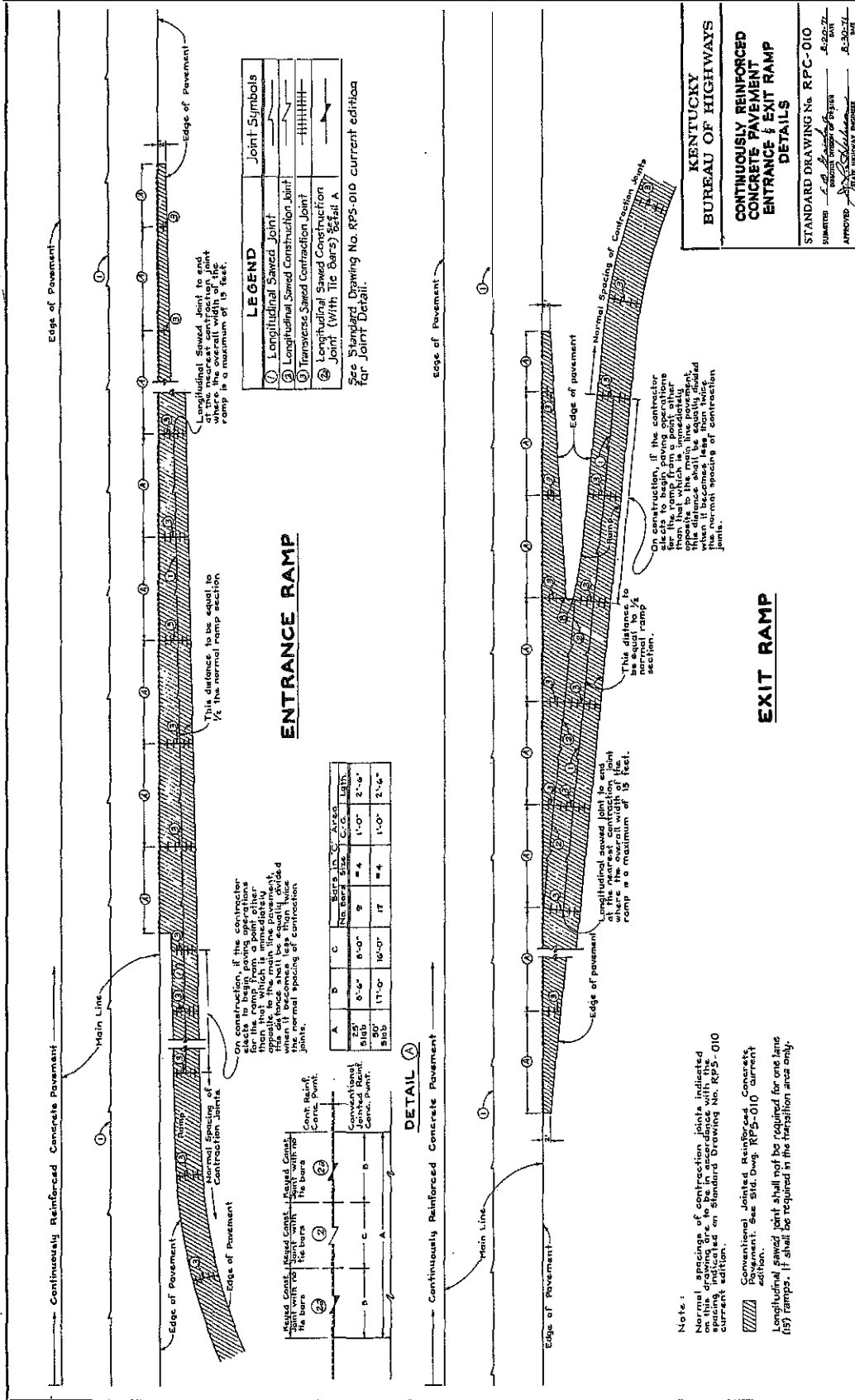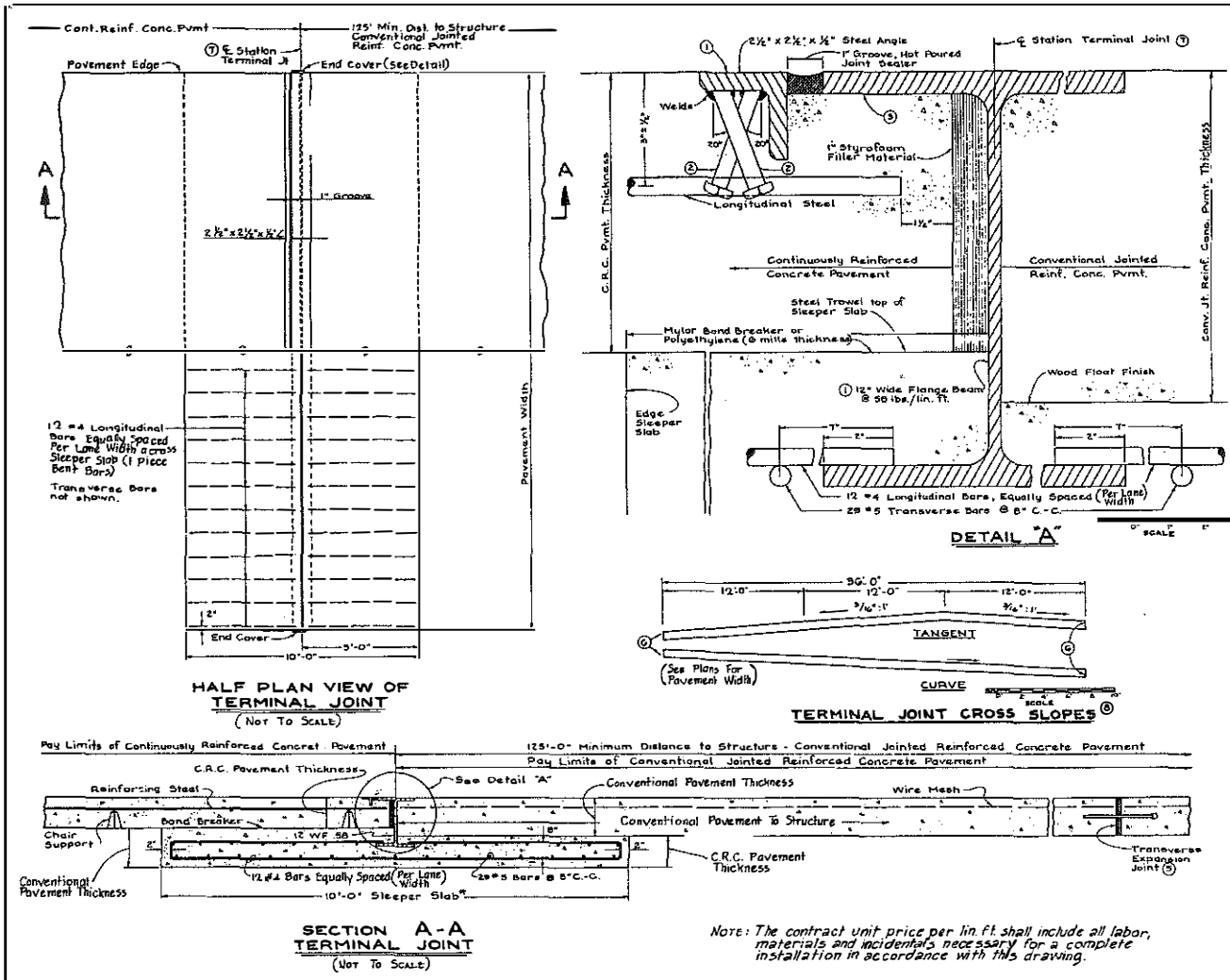C.R.C. Pavement Thickness

Reinforcing Steel

Conventional Pavement Thickness

Wire Mesh

See Detail "A"
Conventional Pavement Thickness

Chair
Support
Conventional
Pavement Thickness

Bond Breaker

12 WF 58

2"

12 #4 Bars Equally Spaced (Per Lane Width)   29 #5 Bars @ 8" C.-C.

10'-0" Sleeper Slab

8"

Conventional Pavement To Structure

2"

C.R.C. Pavement
Thickness

Transverse
Expansion
Joint ⑤

**SECTION A-A
TERMINAL JOINT**
(Not To Scale)

Note: The contract unit price per lin. ft. shall include all labor,
materials and incidentals necessary for a complete
installation in accordance with this drawing.

GENERAL NOTES

① Galvanize after fabrication.

② ½" x 5" bolts, sq. or hex. head, or stud with
forged head. Spacing shall be 12" C.-C. and
staggered on horizontal leg of channel.

③ Bottom of top flange on Cont. Reinf. Conc. Pvmt.
side of joint shall be painted with Culbeck
Asphalt MC-250

④ Drill and tap beam for ⅜" x ½" Stainless Steel
hex. head cap screws. (2 holes each end of
beam.)

⑤ See Std. Dwg. RP5-010, current edition, for
Conventional Reinf. Conc. Pvmt. joint details.

⑥ Ends of W-Beam and Steel Angle shall be
Miter Cut to conform with the vertical face
of the pavement edge.

⑦ Plans will show specified ₵ station or stations
of Terminal Joint location.

⑧ Prior to galvanizing, the W-Beam and Steel
Angle shall be fabricated to reflect the
correct pavement cross slopes as shown
herein, or as shown on the plans if the
specified terminal joint station falls within the
limits of the tangent run-off or the tangent
run-out of a curve.
Two lengths of W-Beams and Steel Angle may be
used with a welded joint at the Apex. Shop
Drawings shall be submitted for approval of
the splice if two lengths are used.
For crown at apex see Std. Dwg. RP5-010 current edition.

1½"   3½"

6½" x 12" x ½" - 10 ga.
Steel End Cover
(2 per joint)

Inside face of End Cover
to be painted with
Culbeck Asphalt MC-250

4¼"

Drill 1" dia. End Cover
Drain Hole

12 WF 58

B = 4½" - 6" C.R.C. Pavement
B = 3½" - 9" C.R.C. Pavement

Scale: Inches

END COVER

**KENTUCKY
BUREAU OF HIGHWAYS**

**CONTINUOUSLY REINFORCED
CONCRETE PAVEMENT
TERMINAL JOINT DETAILS**

STANDARD DRAWING No. RPC-020

SUBMITTED _____

APPROVED _____

A-7