CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

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INTRODUCTION

Historically the design of rigid pavements has involved, to a significant degree, attempts to inhibit the formation of cracks. Such cracks are planes of structural weakness and thus may 1) be points at which blow-ups may occur, 2) permit surface water to enter the pavement and subgrade and to contribute to the freeze-thaw deterioration of the concrete as well as a weakening of the subgrade, 3) contribute to pumping of the rigid pavement, and 4) be esthetically undesirable to the traveling public.

Most efforts to control cracking in rigid pavements has involved the use of reinforcement steel and (or) transverse joints. A wide variety of jointing practices are now in existence. Joints, however, possess many of the undesirable features of the natural cracks that they were intended to eliminate or minimize. The perfect joint would have good load transfer characteristics so that stress concentrations would not occur under any condition of loading. A good joint would be one which would not fail; one which could be sealed against water and incompressible materials which might cause pumping, spalling, and blow-ups; one in which the sealant has no tendency to "pile up" and produce surface irregularities; and one that would resist curling so that the riding quality of the pavement surface would not be impaired. Unfortunately, the perfect joint has not yet been found; thus the concept of eliminating transverse joints has been used more frequently in recent years.

Continuously reinforced concrete (CRC) pavement is a rigid pavement in which the longitudinal reinforcement steel has been placed in a continuous manner by overlapping the steel. Such construction permits long sections of pavement to be placed without the installation of the traditional transverse joints. The only transverse joints in a CRC pavement are the construction joints placed at the end of a day's run and those joints placed where the pavement terminates near a structure or abuts an existing pavement.

Cracks occur in CRC pavements at close intervals; however, those cracks are considered harmless since their interfaces are held closely together by the longitudinal reinforcement. The surface continuity of jointless pavement generally assures a smooth riding quality. Elimination of periodic cleaning and rescaling joints reduces maintenance costs as well as interferences with motorists. Cracks are generally sufficiently narrow to prohibit the ingress of water and incompressibles.

The first recorded continuously reinforced concrete pavement 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, in California in 1949 and in Texas in 1951. By 1959, there had been about 100 miles of equivalent two-lane pavement constructed using CRC concepts. By 1969, the mileage of CRC pavement had increased to over 5,000 miles and an increasing number of highway and airfield agencies were adopting the CRC pavement as a standard or alternative construction procedure (1).

DESIGN ASPECTS OF CRC PAVEMENT

Based on experience from pavements in service for over 30 years, it has been suggested that for equal structural capacity a CRC pavement need not be as thick as jointed concrete pavement. It is now generally recognized that, because of the extremely high exposure to wheel loads, the critical point for thickness design of rigid pavements is at the edge of a transverse joint or crack. Stress reductions in the loaded slab can be obtained by transferring a portion of the load to the adjacent slab by means of dowels and (or) the intimate contact of the interfaces of the cracks. Since a crack in a CRC pavement is held tightly closed by the longitudinal steel and has aggregate interlock for its full depth, it has been reasoned that load transfer at the crack is near perfect (2). Thus, it can be assumed that half the load is carried on each side of the crack because of this intimate contact and aggregate interlock between the interfaces. Allowing for this 50 percent decrease in shear stress within the pavement slab over that of a free edge, it has been reasoned that a CRC pavement which is about 75 percent as thick will be equivalent structurally to the thicker traditional jointed pavement (2). This fact seems to be confirmed by performance of experimental CRC pavements over 30 years of life and by load-deflection measurements on many CRC pavements in Texas (2).

The longitudinal reinforcement holds transverse cracks tightly closed (Figure 1), and good load transfer between adjacent slabs maintains structural integrity of the pavement. The percentage of steel required to control attendant volume changes is primarily dependent on thickness of the slab, tensile strength of the concrete and yield strength of the steel. Other factors which may influence the amount of steel needed are expected temperature changes, shrinkage due to hardening, and moduli of elasticity of the concrete and steel.

![Figure 1. Core from 171 Continuously Reinforced Concrete Pavement.](image-url)

A controlling factor is the crack width to be tolerated. If cracks become too wide, the pavement will not perform properly. However, if an unrealistically low maximum crack width is specified, extremely large amounts of steel would be required to maintain the necessary closure. The optimum width of a crack should be small enough to prevent infiltration of water and at the same time provide adequate load transfer across the crack through the aggregate interlock between the interfaces. A number of investigators (3, 4, 5, 6, 8, 9 and 10) have proposed equations for the determination of the minimum amount of longitudinal steel and theoretical crack interval. However, the amount of steel is generally based on empirical data obtained from experimental pavements. It is the practice in most areas to specify steel having a minimum yield strength of 60,000 psi and to require longitudinal steel of at least 0.6 percent of the gross cross-sectional area of the pavement. In severe climates, where freezing and thawing are extreme, or where unusually heavy traffic loads prevail, it might be desirable to consider use of somewhat higher percentages of longitudinal steel.

The minimum size of longitudinal steel should be such that the spacing between the members will be large enough to permit easy and proper placement of the concrete. A clear space between members of at least twice the top size of the aggregate being used, but in no case less than four inches, is recommended. The maximum size of longitudinal steel to be used is governed by the percentage of steel and the maximum spacing permitted. It is also influenced by bond strength and load transfer considerations. The present consensus is that the maximum size should be a No. 6 bar or its equivalent in deformed wire. For good load transfer and bond strength, it is believed that the spacing should not exceed nine inches. Longitudinal steel should not be placed directly under longitudinal joint.

Pavements have been built with the center of the longitudinal steel...
ranging from 2 1/2 inches below the surface to mid-depth of the slab. Those agencies placing steel at mid-depth feel this results in less total vertical movement under wheel loads and less steel stress at cracks due to temperature differential and wheel loads than at any other placement position. Agencies that place steel near the surface contend it reduces surface width of the crack and provides more protection against the infiltration of surface waters. It is of prime importance that the steel be placed at a sufficient depth to be protected from corrosive attack by salt and water.

**SLIP-FORM PAVING**

Proper placement, consolidation, and finishing of low-slump concrete without pre-erected side forms has been acclaimed as an economical and effective procedure of constructing Portland cement concrete pavements. This construction procedure, commonly referred to as slip-form paving, has often been used in conjunction with the placement of continuously reinforced concrete pavements.

Slip-form pavers differ from conventional pavers in that no fixed side forms are required inasmuch as the slip-form paver has side forms that advance with the machine. Concrete is deposited on the prepared subgrade in front of the machine (see Figure 2), which strikes off and consolidates the concrete (see Figures 3 and 4). The slab is then shaped by an extrusion plate and given an initial floating. The slab is given another floating and then dragged with a burlap bag. All that remains to be done is to cure the concrete and saw the joints, if required. A slip-form paver replaces several pieces of equipment that are normally required for conventional paving, resulting in a reduction in capital investment. A savings is also realized in the accompanying reduction of labor costs.

To investigate the feasibility of slip-form paving, a small pilot unit capable of placing a continuous layer of concrete 18 inches wide and 3 inches thick was built and tested in Iowa in 1947. By 1949, a full scale model was built and used to pave an experimental road ten feet wide and six inches thick. The first commercial slip-form paver was used in 1955; continued improvements have led to the successful use of slip-form pavers in many states.

**KENTUCKY'S EXPERIENCE**

The slip-form process was introduced into Kentucky in 1965 when two highways were widened with a crude slip-form paver. A full-width pavement was first constructed in 1967. To date, more than 100 miles of roadway on 17 projects have been paved by this method in the state.

Through reduction in labor and form costs, contractors are able to economize on total paving costs. Unit bids have generally been lower for jobs on which slip-form pavers were used than on conventional paving jobs (11). Tables 1 and 2 show the average bid on nine-inch reinforced pavement was reduced from $4.87 to $4.83 per square yard, a savings of $0.04 (one percent). A reduction from $5.81 to $5.46 per square yard, or $0.35 (six percent), was shown for ten-inch paving projects of comparable lengths. These average values were calculated from paving projects longer than three miles which had been let since the first slip-form paving project in Kentucky.

Roughness data indicate a tendency of slip-form pavements to have very good ratings (see Figure 5). Two slip-form pavements were rated poor; but they were the first two slip-form paving projects in the state. Riding quality has improved as experience with this type of construction has increased. Strengths of cores taken from slip-formed pavements have exceeded average strengths of cores from conventional pavements, and concrete densities of the two types of construction are similar.

Approximately seven miles of continuously reinforced concrete pavement was placed, using slip-form techniques, on I 71 in Henry, Trinnie, and Carroll Counties (I 71-2(15)37). Paving was completed in late 1968. That experimental slab was eight inches thick and contained 0.677 percent
## TABLE 1. SLIP-FORM PAVING PROJECTS

<table>
<thead>
<tr>
<th>Project Number and County</th>
<th>Letting Date</th>
<th>Project Length (miles)</th>
<th>Pavement Thickness (Inches)</th>
<th>Square Yard Bid</th>
<th>Average Initial Roughness Index</th>
<th>Roughness Rating</th>
<th>Core Density (lb/sq ft)</th>
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## TABLE 2. CONVENTIONAL PAVING PROJECTS

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<th>Project Number and County</th>
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<th>Project Length (miles)</th>
<th>Pavement Thickness (Inches)</th>
<th>Square Yard Bid</th>
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<th>Core Density (lb/sq ft)</th>
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35
longitudinal steel placed at a nominal depth of 3 ± 1/2 inches below the surface. No. 5 deformed bars spaced transversely at 5 1/2 inches were used (Figures 6 through 10).

A sampling-type crack survey was made in July 1969; 100 feet in each 1,000 feet were sampled. The average interval between cracks at that time was 5.8 feet. In July 1970, another sampling crack survey (Figure 11) was conducted and the average crack interval was found to be 4.1 feet. It appears that cracking approached maturity or equilibrium during the first year. Apparently, most of the cracking occurred during the first few weeks after construction – suggesting curing shrinkage as a contributory cause. Although no discrete mode is apparent, there is a strong tendency for cracks to occur at intervals ranging between one and six feet. Typical cracks are illustrated in Figures 12 through 15.

In the July 1970 survey, there was no apparent cracking in the first 85 feet from the beginning station (1742+50) and no obvious cracking in the last 25 feet approaching Station 1866. Several other end situations exist within this project and all conform more or less to this pattern of no cracking. End situations are free to expand and contract to some extent.

Cores removed from the I 71 project approximately 3 1/2 years after placement of the pavement did show that the cracks extend full depth of the slab (see Figures 1a and 16). A close examination of the steel showed slight traces of corrosion (Figure 17), indicating that surface water and salt can penetrate the cracks, at least to the depth of the steel.

Closely-spaced cracking of continuously reinforced pavement has not yet been fully explained. A logic employed in Kentucky (12) attempts to explain the general order of magnitude of observed crack intervals follows.

For a temperature change of $\Delta T$, 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. Because of the bond between the steel and concrete, resisting stress changes per unit length of pavement of $\Delta \sigma_c$ and $\Delta \sigma_s$ are induced in the concrete and steel, respectively.

Assuming "continuity of strains",

$$C_s \Delta T + \frac{\Delta \sigma_s}{E_s} = C_c \Delta T + \frac{\Delta \sigma_c}{E_c}$$  \hspace{1cm} (1)

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

$$\Delta \sigma_s = -\frac{\Delta \sigma_c}{A_s}$$  \hspace{1cm} (2)

where $A_s$ is the area of steel per unit of cross-sectional area of pavement. Substituting Equation 2 into Equation 1 and integrating with respect to $T$, the stress rise per unit length in the concrete is found to be

$$\Delta \sigma_c = \left( \frac{1}{E_s} + \frac{1}{E_c} \right) \left( C_s - C_c \right) \left( T_2 - T_1 \right)$$  \hspace{1cm} (3)

Using Equation 2, the stress rise in the steel is

$$\Delta \sigma_s = \left( \frac{A_s}{E_s} + \frac{1}{E_c} \right) \left( C_s - C_c \right) \left( T_2 - T_1 \right)$$  \hspace{1cm} (4)

The total stress rise, $\sigma$, can be found from

$$\sigma = \Delta \sigma \Delta L$$  \hspace{1cm} (5)

where $\Delta L$ is the length of pavement under consideration. Substituting Equation 5 into Equations 3 and 4 and integrating with respect to $L$, it is found that

$$L_c = \left( \frac{1}{C_s} \cdot \frac{1}{F_c} \right) \left( \frac{1}{C_c} \cdot \frac{1}{F_c} \right) \left( \frac{1}{E_s} + \frac{1}{E_c} \right) \sigma_c$$  \hspace{1cm} (6)

and

$$L_s = \left( \frac{1}{C_s} \cdot \frac{1}{F_s} \right) \left( \frac{1}{C_c} \cdot \frac{1}{F_c} \right) \left( A_s \cdot \frac{1}{E_s} + \frac{1}{E_c} \right) \sigma_s$$  \hspace{1cm} (7)
Figure 6. Standard Drawing RPC-001.

Figure 7. Standard Drawing RPC-002.
For illustrative purposes, Equations 6 and 7 can be used to estimate \( L \) for the following typical values:

- \( C_c = 5.5 \times 10^{5/7} \text{ psi} \)
- \( C_s = 5.5 \times 10^{6/7} \text{ psi} \)
- \( E_c = 5 \times 10^6 \text{ psi} \)
- \( E_s = 30 \times 10^6 \text{ psi} \)
- \( A_s = 0.00677 \)
- \( T_2 - T_1 = 100^\circ \text{ F} \)

Recognizing that the distance between cracks may approach \( 2L \), it is found for concrete with a tensile strength of 600 psi that

\( L_c = 30.7 \text{ inches} \)

For steel with a compressive strength of 90,000 psi,

\( L_s = 31.2 \text{ inches} \)

Such an analysis suggests that crack intervals should typically be 2 1/2 to 5 feet, comparing favorably with crack intervals observed on I 71.

Initial overall roughness index of the I 71 continuously reinforced concrete paving project was 220 (see Figure 5). An adjacent project, constructed by the same contractor using conventional techniques, had an initial roughness of 290.

Further verification of the cracking phenomena may be forthcoming from the several miles of continuously reinforced pavement being constructed on I 275 in Boone and Kenton Counties [I 275-9(23)8]. Eight-inch slabs in the four-lane sections will contain No. 5 bars at a 5 1/2-inch transverse spacing and placed at the mid-depth of the slabs. No. 5 bars will be placed at intervals of 4 3/4 inches in the nine-inch slabs in the six-lane sections. This gives 0.677 percent longitudinal steel for both slab thicknesses.

A short portion of the pavement on I 275 was placed during the 1971 season. A crack survey made two months after placement indicated the...
average crack interval of the two inside lanes to be 9.0 feet. In the third
outside lane, about six weeks after construction, the average interval was
12.0 feet. Four months after placement, the crack interval of the two inside
lanes had decreased to 6.6 feet.

It is anticipated the remaining pavement will be constructed during the
1972 season. Pavement performance will be monitored closely in order to
determine the time and rate of crack development during the very early
life of a continuously reinforced concrete pavement.

CONJECTURE

As noted above, 25 to 80 feet of the ends of CRC slabs do not exhibit
the normal close cracking interval expected in continuously reinforced
pavements. By placing the reinforcement steel at an angle to the centerline
of the slab and in strand lengths of 50 to 100 feet, possibly up to 150
feet, advantages of the CRC pavement (i.e. cracks held tightly closed and
jointless slabs) might result without the development of closely spaced cracks.
Such a placement of steel would result in “end” conditions throughout the
pavement length. Locating the steel in such a manner would no doubt require
modification in placement techniques and (or) manufacture of “diagonal”
mats.

REFERENCES

1. Continuously Reinforced Concrete Pavement, Continuously Reinforced
Pavement Group, 1968.


