The D-Cracking Phenomenon: A Case Study for Pavement Rehabilitation

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THE D-CRACKING PHENOMENON: A CASE STUDY FOR PAVEMENT REHABILITATION

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

SUBJECT: Research Report No. 445; "The D-Cracking Phenomenon: A Case for Pavement Rehabilitation: I75-7(15)164 et seq; Northern Kentucky;" April 1976

The report submitted herewith was first prepared in five, draft copies and forwarded under the date of April 29, 1976. The report relates the present condition of the several sections of pavement, their histories, the state-of-knowledge at the time the projects were designed and the state-of-knowledge at the present time. Documentation of the present condition, together with the state-of-knowledge treatises, provides a basis for the development of an action-plan for rehabilitation of the pavement -- that is, for design, implementation, and funding.

Respectfully submitted,

Jas. H. Havens
Director of Research

JHH:gd
Enc.
cc's: Research Committee
THE D-CRACKING PHENOMENON: A CASE STUDY FOR PAVEMENT REHABILITATION

I 75-7(15)164, et seq., Northern Kentucky

by

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

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INTRODUCTION

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. As deterioration progresses, a term such as dilapidation seems more appropriate.

From the mid-1920’s until the late 1940’s, both expansion and contraction joints were considered to be necessary to the 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.

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 $5.5 \times 10^{-6}$ per degree F, and the modulus of elasticity to be $5 \times 10^{6}$ psi, the stress rise, when fully constrained, would be: $C_5 \Delta T E_5$, 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 of the 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 (1, 2) 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.

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.

The D-cracking phenomenon had not been recognized in Kentucky until 1972 or 1973. In retrospect, perhaps the first indication occurred on the Kentucky Turnpike, a 40-mile (64-km) PCC four-lane pavement, built in 1955. This was a timely forerunner of the interstate system, and design or material defects were rather expected to occur there first. Spalls or slivering occurred at several joints and required patching. This trouble was not then associated with the D-cracking problem as now defined. In 1967, attention was directed to a 10-mile (16-km) section of I 65, south of Bowling Green. The pavement there developed surface cracking and underwent a succession of blowups. The cause was traced to expansive, dolomitic limestone aggregate (3). Meanwhile, the principal concern in regard to the PCC pavements on I 75, in northern Kentucky, was wear in the wheel tracks (presumably caused by studded tires) and slipperiness. Deterioration of the wheel tracks (presumably caused by studded tires) and slipperiness. Deterioration at joints developed rather suddenly in some sections. Discovery and awareness, and an apparent, compelling need for extensive repair and rehabilitation led to an in-depth exploration. Northward, in Ohio, extensive repairs were already in process. There, too, the problem had been under investigation and was attributed, in an implied way, to freezing and thawing. D-cracking of this type has been observed in Minnesota, Ohio, Arkansas, Tennessee, and elsewhere. Early symptoms are recognizable elsewhere in Kentucky; eventually, all pavements having jointing systems of the current type are likely to develop some degree of deterioration of this classical type.
The purpose of this report, therefore, is to document the state of knowledge concerning the problem, the bases for earlier designs, and the history and condition of I 75 in Northern Kentucky and to discuss the probable impact of the problem in maintaining the interstate system to the proper level of serviceability.

In 1967, "Additional Stage Construction" of interstate pavements became eligible for federal funding (4). Pavements constructed prior to October 24, 1963, were eligible to be overlaid or reinforced to extend their service life to 1983—which was then considered to be the 20-year design year. The existing mileage was surveyed, and qualifying sections were included in the 1968 Interstate Cost Estimate. Portions of the North-South Expressway in Louisville, the Watterson Expressway in Louisville, I 64 in Clark and Montgomery Counties, and I 75 from Clay's Ferry to Richmond have been upgraded under that program; other sections included then presumably remain eligible. The condition survey of I 75 in Northern Kentucky did not indicate a need for restructuring at that time. It appeared then that the pavement would endure indefinitely.

PROJECT DESCRIPTION

Figure 1 (see also APPENDIX I) shows the paving projects and dates from the interchange at Crittenden northward to the Ohio River Bridge.

DESIGN

Interstate pavements were designed at the beginning (5) to withstand between 160 and 320 million, equivalent 5,000-pound (2.22-kN) wheel loads (EWL's)—that is, without regard to lane use. All truck traffic was allocated to the outer lane. All lanes were designed equal to the outer lane. These pavements were designed before the AASHO Test Road and before the 18-kip (80-kN) axleload became a nationwide standard of reference. The conversion factor is 32; thus, the design range of EWL's converts to 5 to 10 million EAL's. Whereas 8 x 10^5 EAL's was now convertible to an equivalent load at 1 x 10^5 repetitions from the formula, EAL's = 1 x 10^5 (1.25)^P18, to a prevailing or controlling wheel load (1,000 P/2) of 10,000 pounds (44.5 kN), the original designs for PCC pavements was reasoned as follows:

1. Although the legal weight limit was 9,000 pounds (40.0 kN) on dual wheels, a 30-percent overload allowance for military and expected trends in civilian traffic was superimposed; an impact factor of 1.5 was selected on the basis of expected increasing roughness with time and use. The controlling dual-wheel load was 17,550 pounds (78.1 kN).

2. A k of 150 pci (4.1 Pa/mm) and a working stress of 300 psi (2.07 MPa) were assumed. The PCA's Concrete Pavement Design ... design charts (1951), based on rational theory, indicated a required thickness of concrete of 9.6 inches (244 mm); this was rounded to an even 10 inches (254 mm). The typical section becomes 10 inches (254 mm) on 6 inches (152 mm) of dense-graded, crushed aggregate base (DGA).

For comparison with more current criteria, 8 million EAL's, a working stress of 300 psi (2.07 MPa) (50 percent of modulus of rupture of concrete), and a k of 150 psi (4.1 Pa/mm) would require approximately 12 inches (305 mm) of concrete (cf. AASHO Interim Guide for Design of Pavement Structures, 1973, Figure III-1, P_t = 2.5). At 75 percent of the modulus of rupture (recommended there)—say 450 psi (3.10 MPa) —the required slab thickness would be approximately 9.9 inches (251 mm). To be more faithful to the 1972 Guide, the 8 million EAL's above (determined in terms of damage factors given for flexible pavements) converts to 11.5 million (approximately 40-percent increase) when applied to PCC pavements. Using 11.5 million and 450 psi (3.10 MPa) as the working stress, yields 10.6 inches (269 mm) of thickness.

SOILS

The soils in the study area ascend through the Kope Formation (formerly Eden) beginning southward from Crittenden, the Fairview Formation, and the Bull Fork Formation. These soils are shaly, very plastic, and are subject to creep. The CBR ranges between 1.5 and 4. Numerous fill-slips have occurred on this portion of the road. US 25 and US 27 both have had a longer but similar history of performance in the area. Although the soils are considered to be poor, their effect on the performance of the pavements may be merely acceleration of distress rather than the development of a unique mode of failure.

TRAFFIC HISTORIES

From the standpoint of evaluating the performance of the existing pavements—thus far and from the standpoint of forecasting loadings (EAL's) for the purpose of restructuring or renewing the pavements, only the equivalent loadings have to be known or estimated. Unless more favorable, future, alternate routes divert traffic away or unless other facilities somehow concentrate more traffic onto the various sections of I 75, the extensions of the EAL-lines (dashed) in APPENDIX II represent the best forecast of future loadings.
In synthesizing these loadings, traffic volumes were taken from traffic flow maps for 1963, 1965, 1967, 1971, and 1973. Lane-distribution factors were developed in Research Report No. 444. Classifications were determined by counts made in 1969, 1970, and 1973. Axle weights were taken from W-4 tables (1959 - 1973). To apply these values faithfully to the 1972 AASHO guides, they would have to be increased approximately 40 percent.

PERFORMANCE HISTORY AND PRESENT CONDITION

As mentioned previously, the deterioration at joints appeared rather suddenly (1972 - 1973). Some sealing and chipping of cracks began in that period.

ROUGHNESS

Roughness records from the time of construction through 1975 are given in Table 1. Values through 1969 are in terms of an Automobile-Passenger Roughness Index (average g's x 10^4). Values from 1969 through 1975 are GM Profilometer data (in inches per mile); the corresponding Automobile Roughness Indexes are shown in parentheses. Values in the order of 650 and greater are typical of pavements considered for resurfacing (6). I 75-7(13)73 appeared to be the most critical section in 1975.

BLOWUPS AND/OR D-CRACKS

From the Crittenden Interchange northward to near the junction with I 71 (MP 164 to 173), severe deterioration has occurred at intervals of 1/2 to 1 mile (0.8 to 1.6 km) (Figures 2 and 3). These are equal to, if not actually are, blowups. They exhibit the characteristic heave and bump. Lesser deterioration has occurred at intervening joints. It appears that these are natural points of relief of compression in the pavement. When expanding, concrete having a compressive strength of 3,500 psi (24.1 MPa) is capable of pushing 3,500 feet (1.07 km) of pavement in both directions if the remote ends are not restrained (friction of slab on base assumed to be unity). Partial restraint at the ends such as abutting pavement expanding to a lesser extent decreases the interval proportionately.

On northward, consecutive joints have deteriorated; deterioration tends to diminish on northward of MP 181.

Figures 4 and 5 illustrate two stages of deterioration on the two-lane portion, south of I 71. Figure 6 shows the most extreme condition found (near the Richwood Interchange).

EXPLORATIONS

I 75, NORTHERN KENTUCKY

On June 16, 1975, two joints (Figures 7 and 8) at MP 167, northbound, were sawed a few feet on each side and lifted out lane-width in order to see details of the failures. Two additional joints (Figure 9) were lifted out on July 1, 1975 (MP 177, approximately). The very first site was badly deteriorated (Figure 7). The broken pieces of concrete and rubble were merely excavated and an inverted-T concrete patch made.

OTHER SITES

Two joints (Figures 10 through 17) were excavated on I 64, near MP 50, eastbound, on July 23, 1975. While sawing, there was explosive closure of the saw cut and binding. After exposure, it was found that the slab had split horizontally.

Two joints (Figures 18 through 25), south of the Outer Loop, on I 65 (portion formerly the Kentucky Turnpike) were lifted out on September 25, 1975. At one site, the dowels worked freely. At the other site, one slab was found to have split horizontally a considerable distance; this crack appeared to have existed prior to sawing. Nearby slabs appeared to ramp upward toward the joint. This was interpreted as an indication of splitting and a tendency toward overthrust.

On October 2, 1975, two joints (Figures 26 and 27) on I 65, south of Bowling Green, near MP 18, were sawed out and examined. There, the contraction joints had not cracked through; and the joints were otherwise as new.

PHOTOGRAPHIC RECORDS

The several sites are shown in the series of photographs included herein (Figures 7 through 27). Explanatory notes are included in the respective captions.

WATER UNDER PAVEMENT

The presence or absence of water under the pavement could not be interpreted as being meaningful inasmuch as saw-water and mud were not prevented from flowing under the pavement. Both the water and cuttings appeared to remain perched on the DGA. There did not appear to be any significant inflow after the slab sections were removed. There was no water table in the DGA or in the upper soil of the foundation. Some drainage or seepage occurred between the DGA and the soil.
CAVITATION

There were indications of fresh slivering at the bottom of the saw cut. There were no indications of cavitation under the concrete nor evidences of non-uniform support.

CRACKING OF AC SHOULDERS

Cracking of the asphalitic concrete (AC) surface at the beginning of the shoulder, as shown in many of the photographs included in this report, is regarded as a distinctly separate problem and should not confuse the D-cracking problem. However, such an obvious defect should not be ignored. It is hypothesized that water entering through the joint or emerging from underneath the PC pavement tends to travel longitudinally downgrade in this zone; upon freezing, an ice wedge grows; upon melting, the asphalt surface becomes unsupported and breaks.

Sealing the edge joint would not necessarily prevent this type of damage.

NON-DESTRUCTIVE TESTING

The Road Rater deflections given in APPENDIX III indicate structural weaknesses as were already apparent to the eye.

ANALYSES

Two, seemingly conflicting ideas of the cause of joint deterioration are, here, merged. The freeze-thaw mechanism alone cannot account fully for near-horizontal splitting or diagonal cracking into the slabs. Whereas the compression-and-fatigue hypothesis could account for cracking and fragmentation, it seems somewhat improbable that this type of mechanism alone could account for the rubbly ridge of stripped aggregate and debris found directly under the joints. Indeed, it seems most likely that compression at the joints gradually (except in the case of true blowups) fatigued the concrete, caused cracking, and tended to grind the fragmented concrete to rubble. Fractured concrete is, indeed, more vulnerable to freezing and thawing. At the early stages, maintenance of joint seals would not necessarily have delayed damage; on the other hand, maintenance of seals and control of water under the pavement might have delayed the upward progression of damage. According to the compression hypothesis, the loss of bearing area at the bottom of the crack merely transfers and concentrates the bearing thrust onto the remaining section above.

FAILURE SIMULATIONS

Plain concrete, 6- by 6-inch (212- by 212-mm) prisms were cast, and a groove was sawed into one side to a depth of 1.5 inches (38 mm); they were loaded axially to failure. The failure modes are shown in Figures 28 through 32. The principal mode of failure was shear — as typified by concrete cylinder tests. However, the eccentricity of the axis of loading with respect to the reduced cross section affected the failure angle and induced bending at the simulated joint.

Figures 28 and 29 show a prism loaded through the base of the sawed groove. The opposite side (representing the bottom side of a pavement joint) shattered. The joint, in this case, had not been pre-cracked. Figures 30 and 31 show a test in which the axis of loading was centered through the remaining bearing area at the sawed section. Spalling opposite the groove, splitting, and eventual shear failure are evident.

Figure 32 shows the failure of a specimen which was pre-cracked through the remaining section after the groove was sawed.

PHOTOELASTIC MODELING

A small prism of plexiglass was notched to simulate the grooved joint in a pavement and was then compressed axially. Polarized light photographs are shown in Figures 33, 34, and 35. Lines of equal color are lines of equal shear strain. In Figure 33, the loading is nearly centered through the end areas. Bending tended to produce high shear at the base or bottom of the groove — and spalling or diagonal shear failures at the opposite edge.

In Figure 34, the side opposite the groove was notched to simulate the condition after diagonal shear failure has occurred opposite the groove; the axis of loading, there, is centered through the end areas, as before. In Figure 35, the axis of loading is nearly centered through the remaining cross section. In this case, there is a nearly symmetrical pressure bulb about the remaining bearing area. This condition is conducive to near-horizontal splitting of slabs (see Figure 36).

CONTINUOUSLY REINFORCED CONCRETE OVERLAYS (AND PAVEMENTS)

CRC pavements and overlays have one unwanted characteristic; they crack. The periodicity of cracking, or the interval between cracks, is explained as follows:

For a 100-degree-Fahrenheit rise in temperature, allowing free expansion, but respecting continuity of length:

\[ C_S \Delta T - E_S = C_c \Delta T + E_c \]

where

- \( C_S = \) coefficient of thermal expansion of steel = \( 6.5 \times 10^{-6}/\text{°F} \)
- \( C_c = \) coefficient of thermal expansion of concrete = \( 5.5 \times 10^{-6}/\text{°F} \)
- \( \Delta T = \) rise in temperature (degrees Fahrenheit)
- \( E_S = \) elastic strain in steel = \( \sigma_S/E_S \)
- \( E_c = \) elastic strain in concrete = \( \sigma_c/E_c \)
- \( E_S = \) modulus of elasticity of steel
- \( E_c = \) modulus of elasticity of concrete.
The rise in the expansive force in the steel must equal the rise in the resisting tensile force in the concrete: 

\[ F_s = \sigma_s A_s = F_c = \sigma_c A_c \]

Since \( A_s \) is small, \( A_c \) is approximately \( = 1 \) and \( \sigma_s = -\sigma_c / A_c \). Then

\[
\begin{align*}
6.5 \times 10^6 \times 100 \cdot \sigma_c/A_c F_s &= 5.5 \times 10^6 \times 100 \cdot \sigma_c/E_c, \\
1 \times 10^{-4} &= -\sigma_c / A_c E_s - \sigma_c/E_c, \\
1 \times 10^{-4} &= -\sigma_c / A_c E_s + 1/E_c.
\end{align*}
\]

For

\[
E_s = 30 \times 10^8 \text{ psi}, \quad E_c = 5 \times 10^6 \text{ psi}, \quad \text{and} \quad A_c = 0.00677 \text{in.}^2
\]

\[ \sigma_c = -19.6 \text{ psi}. \]

\( \sigma_c \) is the stress rise per inch of length; it may be attributed to skid friction or bond between the steel and the concrete. However, maintaining continuity of length assures no slip but not equality of elastic strains; therefore, bond is not essential; only a mutual anchorage at the ends, 1 inches apart, is essential. The forces to be balanced increase with the length considered. Therefore, if the critical tensile stress for concrete is equal to the modulus of rupture — say 600 psi;

\[ l_c \sigma_c = -600 \]

\[ \sigma_c = 19.6, \quad l_c = 30.6 \text{ inches.} \]

Similarly, the stress rise in the steel is 2,940 psi; and when \( \sigma_c \) (ultimate) is 90,000 psi, \( l_c = 30.6 \) inches. The crack intervals are approximately 21 ft.

Cracking of the concrete and yielding of the steel occur spontaneously during a warming cycle. Cracks do not necessarily widen unless some external force is involved. It has been observed that the first 50 to 100 feet (15 to 30 m) of CRC pavement from a free end cracks but that the cracks do not widen as they do elsewhere. This is readily explained by the drag theory, as follows:

The force of friction (per unit width) of the pavement laying on the earth is

\[ F_s = fWl, \]

where

\[ f = \text{coefficient of friction} = 1, \]

\[ W = \text{weight of pavement per square inch}, \]

\[ l = \text{length of pavement considered}. \]

Since \( F_s = \sigma_s A_s = 90,000 \text{ psi} \times 0.00677 \text{ in.}^2/\text{in.} \) and assuming concrete has a unit weight of 144 pounds per cubic foot, and a 10-inch thick pavement is being considered,

\[ l = 90,000 \text{ lb/in.}^2 \times 0.00677 \text{ in.}^2/\text{in.}/(10 \text{ lb/12 in.}^2) \]

\[ l = 1,462 \text{ inches or 122 feet.} \]

It is seen that 0.677 percent steel is capable of pushing or pulling approximately 120 feet (36 m) of pavement without yielding. Cracks more remote than 120 feet (36 m) from a free end are likely to widen (due to yielding of steel) during a cooling cycle.

**NON-REINFORCED CONCRETE**

Concrete having a compressive strength of 3,500 psi (24 MPa) is capable of pushing approximately 3,500 feet (1.1 km) of pavement in both directions from a given point; this is the normal distance between blowups — one to two per mile, more or less. This is a basis for spacing expansion joints at approximately one-half mile (0.8-km) intervals.

Plain concrete (not reinforced) having a tensile strength of 600 psi (4 MPa) would be capable of pulling 600 feet (0.18 km) of pavement in both directions from a given point during shrinkage or during cooling. Very fresh concrete, having a tensile strength of 30 psi (0.21 MPa) at the first onset of cooling or drying shrinkage would be capable of pulling only 30 feet (9.1 m) of pavement in both directions. This is a basis for spacing contraction joints at 50 feet (15.2 m) or less.

Contraction joints should not be sawed (should not have a reduced cross section); they should consist of a parting or separating panel extending full depth and width and dowel bars.

**REHABILITATION**

Various terms may be applicable to the decision process; only the term describing the action plan will be specific. Terms like "restructuring", "additional stage construction", "reconstruction", "rehabilitation", "overlaying", and others allude to alternatives to be considered. Cost-effectiveness should surely guide the decision. Decisions concerning sections of I 75 in Northern Kentucky are likely to become precedental — that is, in the sense of establishing a model plan which may be applied successively to other PCC, interstate pavements developing the same type of deterioration in the near or distant future.

**ZERO MAINTENANCE-COST CONCEPT**

Once upon a time, many engineers argued that a pavement which did not develop minor failures during its designed life was overdesigned and extravagant. However, the impracticality of performing maintenance or any operation which interferes with traffic on high-volume expressways has given greater credence to the idea of designing a maintenance-free facility. In analyzing cost-effectiveness, due weight must be given not only to the inflating costs of the maintenance operations but also the cost of the inconvenience to the travelling public. Until the problem on I 75 arose, maintenance costs of PCC pavements on the interstate system had been very low; and those costs charged as pavement maintenance were attributable largely to leveling approaches to bridges, dips, and fill-slips (7).
Had the D-cracking problem not emerged, the original designs might otherwise have been nominally free of maintenance during their design life. Design life, in this context, is differentiated from design term. For instance, sections of pavement on I 75 have accumulated their 20-year, estimated, design traffic in 15 years or less. **SALVAGE VALUE (Excerpted from Ref. 7)**

Residual or salvage value depends on the particular circumstances. If a pavement is to be abandoned and the land reclaimed, it is a liability; and the cost of disposal would be accounted in the project. The residual value of the pavement would be a technical loss. If an existing pavement is to be incorporated into a new but equivalent structure, the estimated cost of a totally new structure minus the estimated cost of reinforcing the existing structure is the estimated salvage value of the existing pavement. Existing pavement layer thicknesses multiplied by estimated, fractional structural worth factors yields an estimated, equivalent like-new thickness contribution to the new structural design. The structural worth factors treated as (100-percent deterioration)/100, multiplied by the current cost per square yard per inch of thickness of the material in place yields a salvage value more directly. Certain pretreatments or preconditioning may be necessary to render an existing pavement usable in a reinforced (overlayd) structure; the cost of preconditioning would add to the cost of utilizing the old pavement and has the effect of diminishing its residual value. A portion of the salvage value estimated at the first extension of service life extends successively into the second, third, etc.; and, therefore, the value compounds in some yet undefined way. Nevertheless, the value at any point in time is probably best and most conveniently estimated by the current cost of the material in place multiplied by the residual structural worth factor. Continual assessments of pavement conditions are, indeed, important factors in guiding decisions to defer or intensify maintenance, to overlay, or reconstruct — whether based on situational analysis or systems management theory. The residual values of interstate and parkway pavements and others designed to comparable standards are expected to be very high at the end of their designed service life. Additional thicknesses required in redesigning and extension of service life actually may not greatly exceed overlay thicknesses required to rehabilitate lesser pavement structures. If the renewal or extension of service life is deferred or delayed too long, structural deterioration accelerates. In reality because of competing priorities for funding, each pavement project generates its own unique history.

**REFLECTION CRACKING**

Preventing, or at least minimizing, reflection cracking (sometimes called sympathetic cracking) in pavement overlays — especially PCC pavements — has challenged the most ingenious minds for many years. Cracking and segmentation of a pavement into slabs and plates allows differential or discontinuous movement; unless the movement is minimized — by burying to sufficient depth to minimize temperature changes and to reduce live-load deflections — the high stress concentrations resulting must be borne in the overlay itself. In overlaying portions of the German Autobahn, the existing PCC was fragmented into plates about 2 feet (0.6 m) or less in width. Presumably, one avoids the reflection-cracking problem in this way and may design the overlay in terms of carrying the live load. On the other hand, thin asphaltic concrete overlays, because of their early blackness, tend to absorb more sun heat and increase the temperature ranging in PCC slabs underneath. It is estimated (Figure 37) that about 5 inches (127 mm) of AC overlay would be needed to reduce the cycling again to a level no more severe than when the PCC was exposed directly to sun heat. Even so, estimates of stressing in the overlay, as well as experience, indicates that reflection cracking will emerge within a few years. Some reflection cracking may be tolerated — as indeed it has been heretofore. US 25, from north of Georgetown and paralleling I 75, has been overlaid many times. Sections, there, were overlaid before World War II. The life cycle of overlays, in the past, has always seemed to be too short — mostly because of reflection cracking and raveling of the overlay at joints.

Wire mesh and expanded metal both have been used to reinforce the overlay at joints and cracks. Bond-breaking treatments in the vicinity of joints has been employed; rubber has been incorporated into the AC overlay mixture; asbestos fibers have been included also (8). Most of these innovations delay the appearance of cracks.

To think in terms of a 20-year, extended, maintenance-free life ahead means that one must consider thicknesses of AC significantly greater than 5 inches (127 mm). Precedences (9, 10) now exist for 9 or more inches (227 or more mm).

Problems besetting CRC overlays have been mentioned. Figure 38 shows a schematic attempt to control cracking. The type of cracking illustrated in Figure 38 is induced by the steel and should not be confused with reflection cracking. It appears that this cracking may supplant or otherwise obscure reflection cracking. Rigid foundations appear to improve the
performance of CRC. Of continuing concern is the eventual corrosion of the reinforcing steel and deterioration of the concrete at the cracks.

Prestressed concrete overlays have not been given widespread trials; the principal concern involves the eventual corrosion of the tendons and the loss of prestressing.

**OHIO-TYPE REHABILITATION**

Figures 39 and 40 show the method of repairing joints on I 75 north of the Ohio River. The problem there was similar to the problem in Kentucky. There, the joints were spaced at 40-foot (12-m) intervals; whereas in Kentucky, they were spaced at 25-foot (7-m) intervals. Gravel (glacial outwash) aggregates were employed in both cases. In the Cincinnati area, the excavated joints (in the photo shown) were backfilled with concrete. Elsewhere in Ohio, where isolated joints had deteriorated (similar to blowups), the pit was backfilled (sometimes) with bituminous concrete; usually a bump was noticeable. The eventual plan for I 75 through Cincinnati was, or is, to overlay it with bituminous concrete.

**ARKANSAS-TYPE OVERLAY**

In August (1975), the Arkansas Highway Department, the FHWA, the Asphalt Institute, and the Tennessee Highway Department held a seminar to announce and exhibit a so-called, crack-relief overlay system. The system is basically a very thick overlay (9), the first layer of which was originally made up of large aggregate (2 to 3 inches (50 to 75 mm)) and 1.2 to 1.5 percent asphalt. Because of instability, the gradation has been moderated to include some smaller sizes (10).

The overlay is shown schematically in Figure 41. A photograph of the fresh layer is shown in the rightward portion of Figure 42. The leftward portion of the photograph shows dense, asphaltic concrete over the coarse course.

The apparent principle or hypothesis is that the large stone bridge over the joint; slippage occurs; cracking tends to bifurcate or branch as it migrates upward and diffuses and disappears before reaching the surface.

It was observed that excess asphalt drained out of the mixture while enroute to the paver.

The pavement being overlaid was comparable to I 75 in Northern Kentucky. D-cracking, or joint deterioration, was the principal reason for overlaying.

The sponsors of the seminar implied that long-time service was expected from the system but none seemed to feel the usual burden of proof which customarily precedes such disclosures. At that time the longest service-history mentioned was about 6 years; the case cited was in Tennessee and involved a thinner but denser, coarse course.

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<td>I 75-7(10)/169</td>
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<td></td>
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<tr>
<td>I 75-7(15)/164</td>
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</table>

*Average of four outer lanes
Figure 1. PCC Paving Projects; I 75, Northern Kentucky.

- **I 75-8(16) 89**
  - 5th St to Bridge
  - Bates & Rogers Construction Co.
  - Completed: 11-8-1963

- **I 75-8(17) 85**
  - U.S. 25 to 5th St
  - DeSalvo Construction Co.
  - Completed: 7-6-1962

- **E 5**
  - I 75-7(17)
  - Third Inner Lanes Added
  - DeSalvo Construction Co.
  - Completed: 6-30-1966

- **F 1**
  - I 75-7(41)
  - U.S. 42 to Boone-Kenton Co. Line
  - Fischer Construction Co.
  - Completed: 4-12-1962

- **E 11**
  - I 75-7(15) 169
  - Walton Interchange to Walton Interchange
  - W.L. Harper Co.
  - Completed: 4-12-1962

- **E 12**
  - I 75-7(16) 164
  - Grant-Kenton Co. Line to Walton Interchange
  - W.L. Harper Co.
  - Completed: 4-12-1962

- **E 14**
  - I 75-7(15) 197
  - Williamstown Interchange to Grant-Kenton Co. Line
  - Shamrock & Schneider
  - Completed: 11-3-1961
Figure 2. Blowup; Two-Lane Section; I 75 South of I 71.
Figure 3. I 75 South of I 71; Shows Severe Deterioration at One Joint but Lesser Deterioration at Joints Nearby (February 1976).
Figure 4. Advanced Stage of D-Cracking; I 75, Northern Kentucky.
Figure 5. Fully Developed D-Cracking and Joint Deterioration; I 75, Northern Kentucky.
Figure 6. Most Extreme Condition; I 75, near Richwood Interchange.
Figure 7.  I 75, MP 167.7, Northbound; First Site Excavated (June 3, 1975).
Figure 8. I 75, MP 168 + 100 feet (30 m), Northbound; Second Site Excavated (June 3, 1975).
Figure 9. I 75, MP 176 + 300 feet (91 m); Third Site Excavated (June 3, 1975).
Figure 10. I 64, near MP 50, Eastbound; Note Closure of Rightward Saw Cuts; Explosive Report Was Heard (July 23, 1975).
Figure 11. I 64, MP 50; Removing Lateral Section.
Figure 12. I 64, MP 50; after Exposure.
Figure 13. I 64, MP 50; Outer Edge of Pavement at Joint; Cracks Faintly Visible.
Figure 14. I 64, MP 50, Underneath Side of Joint, Adjacent to Centerline (Note fresh slivering along sawed edge and old crack surface approaching center joint).
Figure 15. I 64, MP 50; Sawed Cross Section through Contraction Joint, Midway of Outer Lane (Note cracks).
Figure 16. I 64, MP 50; Exposed Centerline Joint at Contraction Joint.
Figure 17. I 64, MP 50; Joint near Sites Excavated, Showing Early Stage of D-Cracking (Concrete contains limestone aggregate).
Figure 18. I 65, South of Outer Loop; Excavated on September 25, 1975.
Figure 19. I 65, South of Outer Loop; Showing Debris under Joint and Horizontal Splitting.
Figure 20. I 65, South of Outer Loop; Showing Low-Angle Splitting (Note dowel bars).
Figure 21. I-65, South of Outer Loop; Showing Low-Angle Splitting Extending into Slab; Pavement 9 Inches (229 mm) Thick.
Figure 22. Second Site, I 65, South of Outer Loop; Saw Cuts Made a Day Previous.
Figure 23. Underneath Side of Joint, Second Site, I 65, South of Outer Loop.
Figure 24. I 65, South of Outer Loop, Second Site; Showing Socket at Centerline.
Figure 25. I 65, South of Outer Loop, Second Site; No Splitting; Dowels Slipped Freely.
Figure 26. I 65, Vicinity of MP 18; Pavement Containing Expansive Aggregate (October 2, 1975).
Figure 27. I 65, MP 18; Showing Depth of Cracking (October 2, 1975).
Figure 28. Compression Failure Simulation; Sawed Groove Is on Right Side of Prism; Axis of Load Is through Base of Groove.
Figure 29. Sequence to Figure 28; Note Splitting.
Figure 30. Simulation of Compression Failure; Axis of Loading Centered through Reduced Cross Section.
Figure 31. Sequel to Figure 30.
Figure 32. Compression Failure Simulation; Loading Centered on Total Section.
Figure 33. Photoelastic Model; Simulating Sawed Joint; Loading Centered.
Figure 34. Sequel to Figure 33; Side Opposite Groove Notched to Simulate Shear at Bottom of Joint.
Figure 35. Sequel to Figure 34: Load Centered through Remaining Bearing Area.
Figure 36. Schematic Diagram of Stages of Deterioration at Joints.
Figure 37. Estimate of the Depth of Asphalitic Concrete Overlay Needed to Avoid an Increase in the Temperature of the Existing Pavement.

![Graph showing the relationship between Depth of Overlay and Pavement Temperature.](image)
Figure 38. Continuously Reinforced Concrete Overlay with Elastic Joints (Controlled Cracks) (from Ref. 11; also see Ref. 12).
Figure 39. I 75, North of Ohio River (July 16, 1975).
Figure 40. I 75, North of Ohio River; Finished Patch at Left (July 16, 1975).
Dense-graded asphalt concrete surface course: 1.5 in. (4 cm)
Dense-graded asphalt concrete leveling course: 2 in. (5 cm)
Asphalt crack-relief layer: 3.5 in. (9 cm)
Existing Pavement

Cross-section
CRACK-RELIEF LAYER OVERLAY SYSTEM

Figure 41. Arkansas-Type, Thick Overlay.
Figure 42. Arkansas-Type Overlay; Coarse Course at Left (August 15, 1975).
REFERENCES

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APPENDIX I

Aerial Photographs
Taken
April 3 and 5, 1976
APPENDIX II

Graphs Showing Estimated 18-kip (80-kN), Equivalent Axleloads Accumulated and Projected
KY 14, WALTON TO 171

KY 491, CRITTENDEN TO KY 14, WALTON
KY 236, DONALDSON ROAD TO I-275

I-275 TO BUTTERMILK PIKE
APPENDIX III

Strip Charts Showing Road Rater Soundings about Joints on I 75 Showing Severe Deterioration Compared to a Joint on I 64 Showing no Deterioration
I 75, NORTHBOUND
MILEPOST 168 + 100 FT.

ROAD RATER TESTS

STATIC LOAD = 1670 LBS.
DYNAMIC LOAD
PEAK TO PEAK = 600 LBS.
READINGS = INCHES X 10^6
"35" = (0.00035)
FREQUENCY = 20 CPS

DISTANCE FROM SHOULDER, FEET

DISTANCE FROM STATION 1269 + 12.5
ROAD RATER TESTS

STATIC LOAD = 1670 LBS.

DYNAMIC LOAD
PEAK-TO-PEAK = 600 LBS.

READINGS = INCHES X 10^{-5}
"35" = 0.00035 INCHES

I 75, NORTHBOUND
MILEPOST 167 + 3830 FT.
I 75, NORTHBOUND MILEPOST 176+300 FT.

ROAD RATER TESTS

STATIC LOAD = 1670 LBS.
DYNAMIC LOAD
PEAK TO PEAK = 600 LBS.
READINGS = INCHES X 10^5
"35" = 0.00035 INCHES
FREQUENCY = 20 CPS
DISTANCE FROM SHOULDER, FEET

CONTRACTION JOINT

I-64, EASTBOUND
MILEPOST 85 + 300'

ROAD RATER TESTS
STATIC LOAD = 1670 LBS.
DYNAMIC LOAD
PEAK TO PEAK = 600 LBS.
READINGS = INCHES X 10^5
"35" = 0.00035 INCHES
FREQUENCY = 20 CPS