A Twenty-Year Report on the Illinois Continuously Reinforced Pavement

JOHN E. BURKE and JAGAT S. DHAMRAIT, Illinois Division of Highways

THE possibility that continuous reinforcement could serve in portland cement concrete pavements as a means for eliminating or greatly reducing the numbers of transverse joints and open cracks that too often have become points of weakness in otherwise sound pavements was suggested many years ago. Continuous reinforcement was used as early as 1921 in the Columbia Pike experimental pavement near Washington, D.C. The next recorded use was in an experimental pavement built on US 40 west of Indianapolis in 1938.

With this background of experience to draw upon, the Illinois Division of Highways, in cooperation with the U.S. Bureau of Public Roads, in 1946 undertook an experimental study to develop more definite information on the relationships between performance and slab dimensions and steel amounts of continuously reinforced pavements. The establishment of the project in Illinois was generated largely through the efforts of W. R. Wooley of the Bureau of Public Roads and J. D. Lindsay of the Illinois Division of Highways.

The Illinois study has centered on an experimental continuously reinforced pavement constructed in 1947 and 1948 on US 40 west of Vandalia. The study was first described by Russell and Lindsay in a 1947 report (1). A second and third report were subsequently published (2, 3). The present paper, which covers a 20-year span of service of the pavement, is intended to be the final report of the study. While most of the experimental pavement is structurally capable of continuing to serve the heavy traffic now being served, the construction of paralleling pavements on the Interstate system soon will divert all but local traffic from the experimental pavement.

Construction of the Illinois experimental pavement began on September 25, 1947, and was completed on May 20, 1948. It was opened to traffic immediately upon completion. The experimental section of continuously reinforced pavement is 5½ miles long and divided into eight test sections, six of which are about 3500 ft long and two of which are about 4200 ft long. The pavement is 22 ft wide, and placed directly on natural fine-grained soil. The longitudinal reinforcement is continuous through each test section but not between sections. It consists of round deformed bars (ASTM 305-47T) meeting the requirements of ASTM designation A 16 for rail-steel bars. Transverse reinforcement consists of round deformed bars meeting ASTM designation A 15 for intermediate grade billet steel, chosen to permit welding of supporting steel to the bars. The transverse bars extend through the entire width of the pavement and the customary center joint has been omitted. Various details of the test sections, including the size and spacing of the steel reinforcement, are given in Table 1. The results of mill tests of the reinforcing steel are given in Table 2.

Four of the test sections are uniformly 7 in. thick and four are 8 in. in thickness. Four percentages of longitudinal steel, 0.3, 0.5, 0.7, and 1.0 percent, based on the gross cross-sectional area of the pavement, were used with each thickness of pavement. Both bar size and spacing were varied to obtain the different percentages of steel. The reinforcement bars were assembled as a continuous mat on the subgrade by means of chairs extending upward to approximately 3 in. below the finished surface of the pavement. Splice laps were maintained at a length equivalent to 30 bar diameters. Air-entrained concrete was deposited to full pavement thickness in a single operation and finished by means of a conventional spreader and finishing machine followed by the usual hand operations.

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TABLE 1

TEST SECTION DETAILS

Section	Length (ft)	Thickness (in.)	Longitudinal Steel		
			Diameter (in.)	Spacing (in.)	Percent
1	3504	7	3/8	5 1/4	0.3
2	3504	7	1/2	5 9/16	0.5
3	3504	7	5/8	6 1/4	0.7
4	3504	8	3/8	4 9/16	0.3
5	3504	8	1/2	4 13/16	0.5
6	3508	8	5/8	5 7/16	0.7
7	4233	8	3/4	5 7/16	1.0
8	4233	7	3/4	6 1/4	1.0

Note: Transverse reinforcement 3/8-in. diameter round deformed bars at 12-in. centers in half of each section and at 18-in. centers in other half.

TABLE 2

RESULTS OF MILL TESTS OF REINFORCING STEEL

Bar No.*	No. of Tests	Yield Point (psi)	Tensile Strength (psi)	Elongation (%)
(a) Longitudinal Reinforcement				
3	6	78,701	119,469	14.3
4	14	66,223	107,355	14.5
5	17	70,778	127,759	12.5
6	21	70,157	124,888	11.7
(b) Transverse Reinforcement				
3	7	43,498	77,356	21.5

*Normal diameters of 3/8, 1/2, 5/8, 3/4 in., respectively.

A conventional concrete pavement constructed immediately west of the experimental pavement at about the same time and carrying approximately the same traffic has been used as a control section. This pavement was built to the Illinois standard design of the period and consists of a 10-in. slab reinforced with welded wire fabric at the rate of 78 lb per 100 sq ft, with doweled contraction joints spaced at 100-ft intervals and no expansion joints. Unlike the continuously reinforced pavement, which was placed directly on the natural fine-grained soil, the conventional pavement was placed on a 6-in. thickness of granular subbase.

The continuously reinforced pavement has been the subject of many observations and measurements from the time of its construction, and a relatively thorough record of its performance has been obtained. Certain pertinent observations and measurements have also been made on the adjoining conventional pavement.

TRANSVERSE CRACKING

Characteristic of continuously reinforced pavement are stress-relieving transverse cracks that develop at very close intervals. Crack development begins shortly after placement of the concrete as shrinkage takes place during setting and drying. It is an essential feature in the design of continuously reinforced pavement that the longitudinal steel reinforcement be properly distributed and of sufficient area to maintain the structural integrity of the slab by preventing the cracks from opening excessively during contraction of the concrete.

The crack frequencies near the free ends of long, continuously reinforced slabs would be expected to be similar to those in conventionally reinforced slabs. This proved to be

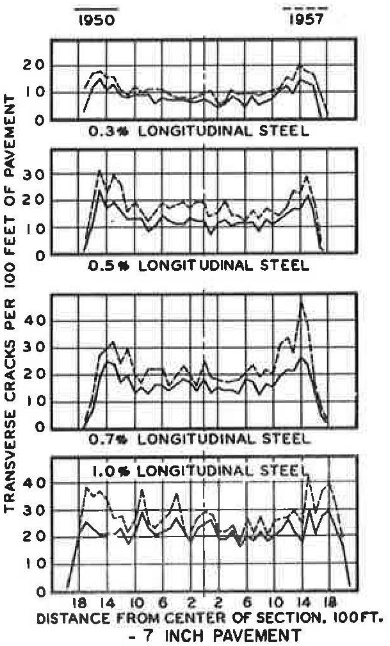

Figure 1. Crack frequency.

the case in the Illinois experiment and has been described in the earlier reports. The crack frequency was found to increase gradually away from the ends up to a distance of about 500 ft, to decrease over the next few hundred feet, then to become fairly stable through the longer center portion of the pavement. This is demonstrated in Figure 1.

Two relationships are of special significance in the consideration of transverse cracking of continuously reinforced pavements. One of these is that which exists between crack frequency and age, the other is that between crack frequency and steel percentage. Both are shown in Figures 2 and 3 for the 20-year period that the pavement has been under observation. With regard to the crack frequency and age relationship, it will be seen that the average crack interval decreased rather rapidly during the first 3 or 4 years following construction, after which the rate of decrease became and stayed very slow through the remainder of the 20 years of observation. It will be noted also that the rate of decrease has been slowest for the higher steel percentages.

A definite relationship between crack frequency and steel content is also obvious from Figures 2 and 3. At all ages at which observations of crack frequency have been made up to 20 years, the average crack frequency is consistently higher for each incremental increase in steel percentage. This relationship is in accord with theory regarding the behavior of long, continuously reinforced slabs subjected to temperature and moisture changes.

The magnitudes of the average crack intervals and of the changes in the average intervals that have taken place through the years are of interest. In Figures 2 and 3 it will be seen that, with the exception of the 0.3 percent steel sections, which have proven to be inadequate, average crack intervals ranged from about 5 to 9 ft at the end of 4 years and from about 5 to 8 ft at the end of 20 years.

Transverse Crack Width

It is essential that the steel reinforcement of continuously reinforced pavements hold the cracks to a narrow width if these pavements are to function properly. Open cracks can be expected to destroy the continuity afforded by aggregate interlock, a continuity

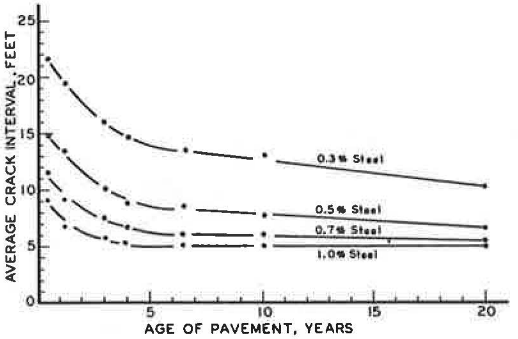

Figure 2. Relationship between crack interval and age, 7-in. pavements.

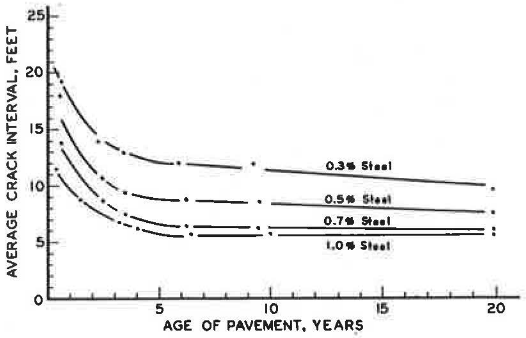

Figure 3. Relationship between crack interval and age, 8-in. pavements.

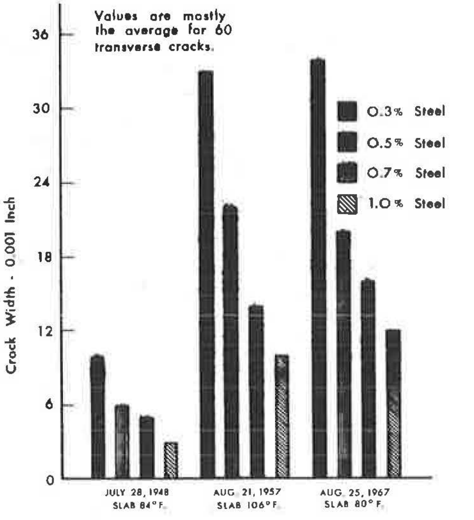

Figure 4. Crack width data, 7-in. pavements.

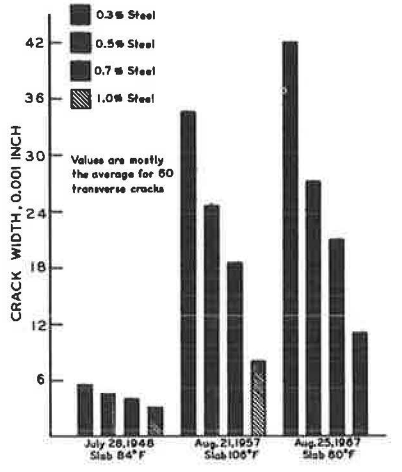

Figure 5. Crack width data, 8-in. pavements.

that is needed to prevent excessive flexing under traffic with resultant spalling of the concrete and rupture of the steel. Excessive openings can lead also to the infiltration of incompressible material, causing spalling and blowups; and to the entrance of water to reduce subgrade support and cause rusting of the steel. Failures that have taken place at some of the construction joints on the experimental project give ample evidence of the importance of aggregate interlock and the need for narrow crack openings.

Transverse crack openings in continuously reinforced pavements have been found to be not the same width for the entire thickness of the pavement and are difficult to measure. The apparent width observed by looking at the surface of the pavement is characteristically greater than the width through most of the depth of the crack.

Early in the study 60 transverse cracks were selected from each test section for periodic measurements of crack width with a measuring microscope capable of measuring to 0.001 in. Ten cracks were selected to represent the cracks at each end of each test section, another 10 to represent those at each quarter point, and 20 to represent those at the center of each section. The first crack width readings were made in July 1948.

In an attempt to obtain width readings most nearly representative of the true width of the cracks, the microscope was focused some distance down the crack. While the method is not considered to be extremely accurate, it is believed to have given fairly reliable results, although the widths may be somewhere in between those visible at the surface and the actual widths through the major portion of the pavement depth.

Average crack width data for the transverse cracks measured in each section are shown in Figure 4 for the 7-in. sections and in Figure 5 for the 8-in. sections for the years 1948, 1957, and 1967. Averages are for 60 cracks measured in each section, except for the 1967 measurements in which some patching and resurfacing removed a few cracks in the 0.3 percent steel sections from the observation. It will be noted that a correlation has existed between crack width and the percentage of longitudinal steel through 20 years of observation of the experimental pavement. Crack widths become progressively smaller as the percent of steel increases. It is also apparent that crack openings have become wider with time. The rate of increase, however, seems to have decreased with time.

The 7-in. sections, except for the 1.0 percent steel section, in 1948 showed somewhat wider crack openings than did the 8-in. sections. In 1967 this trend had become reversed. No explanation for these observed conditions is apparent.

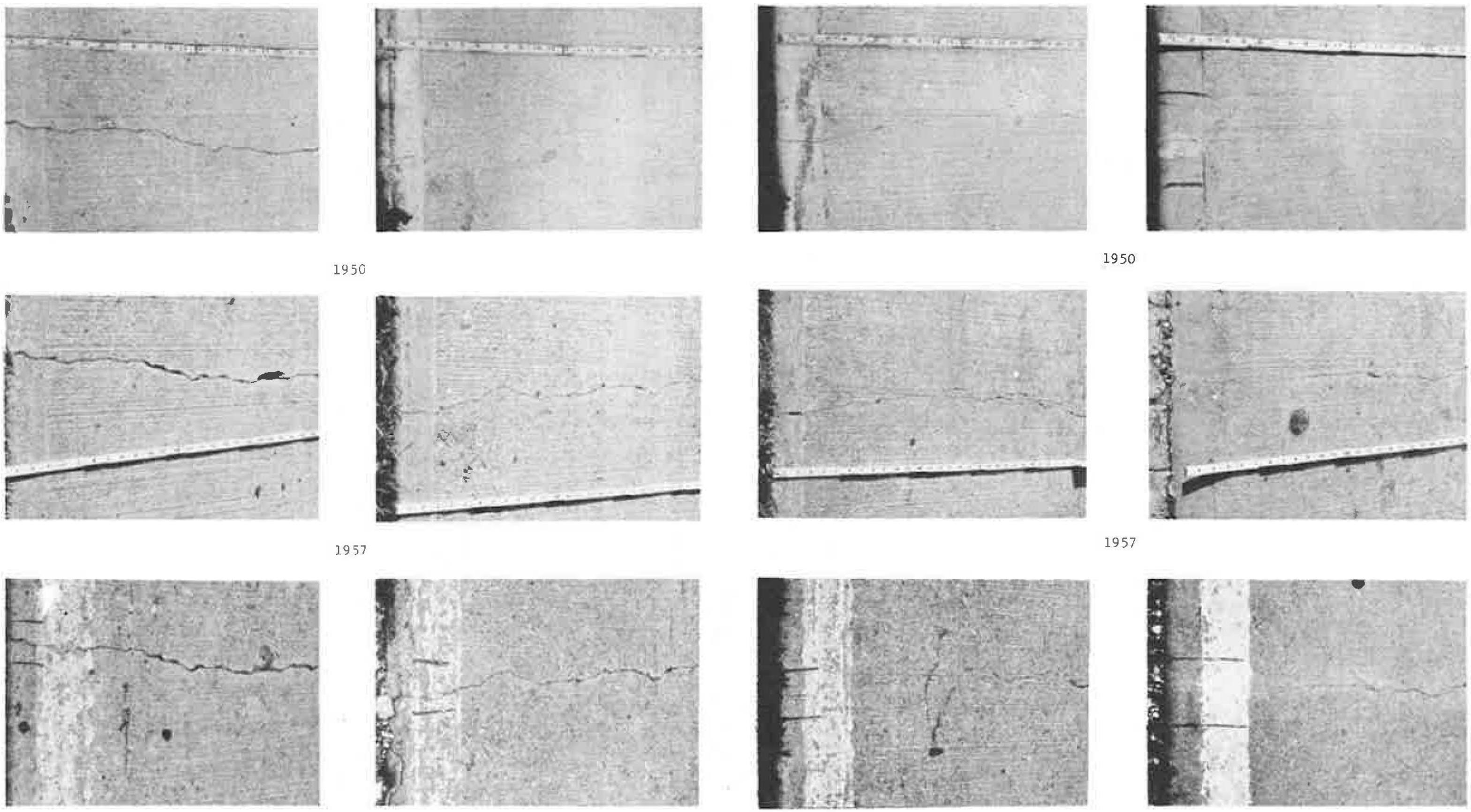

Figure 6. Typical transverse cracks in 7-in. sections.

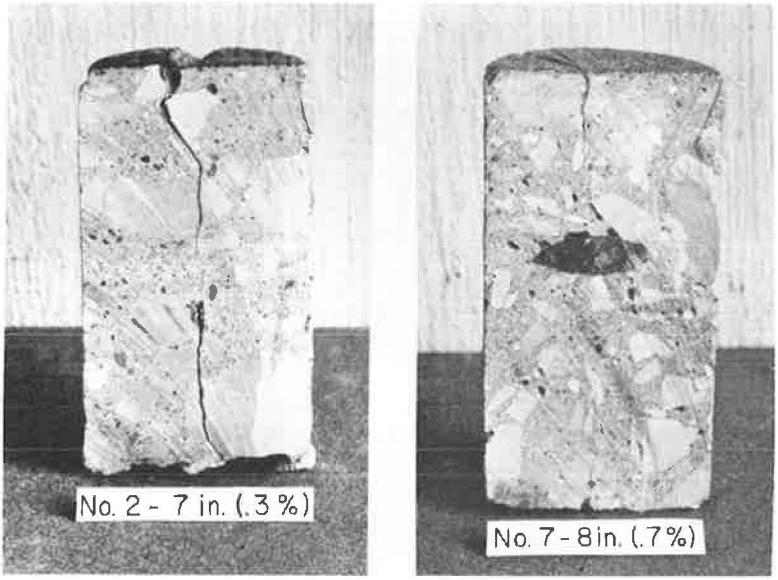

Figure 7. Condition of transverse cracks as seen in cores sawed to expose clean face of concrete.

The pavement surface at most of the cracks now shows occasional slight spalling. The amount and severity of the spall is related to the percentage of longitudinal reinforcement, and is most pronounced in the sections having 0.3 percent of steel. Little increase in spall has taken place after the first few years, and that which is visible after 20 years of service does not detract appreciably from the general appearance of the pavement, nor require any maintenance. The cracks are not visible to occupants of vehicles traveling the pavement at ordinary speeds. Photographs of a typical transverse crack in each of the sections of 7-in. pavement taken in 1950, 1957, and 1967 are shown in Figure 6.

The performance of the pavement at transverse cracks is considered to have been satisfactory except at a few locations in the 0.3 percent longitudinal steel sections, at two isolated locations in the 0.5 percent steel section of 7-in. thickness, and at cracks associated with the joints that have failed. Structural continuity at these locations obviously was lost; elsewhere, it has been maintained. While the crack openings are such that evidence of the infiltration of very fine material that might be speculated to have been moisture-deposited has been found, no signs of serious structural damage attributable to this source have been observed.

Photographs of cores taken through typical transverse cracks in 1967 are shown in Figure 7. It is apparent in the photographs that the crack openings become very fine a short distance below the surface, at about the level of the steel reinforcement, and remain fine to the bottom of the pavement. An examination of the cores showed a discoloration of the slab faces at the cracks, indicative of the infiltration of fine material. An examination of the condition of the steel reinforcement in the cores showed only a negligible amount of rusting.

The need to remove a portion of the experimental pavement in 1965 as part of a new construction contract provided an opportunity to observe closely the condition of the steel of a continuously reinforced pavement after 18 years of service. The new construction required the removal of 957 lineal feet of section 7, which is the 8-in. pavement with 1.0 percent of longitudinal steel.

The pavement of section 7 was considered to be in excellent condition at the time of removal. Transverse cracks appeared to be tight and showed only a minor amount of spalling at the edges. The longitudinal cracks that had formed through most of the section in the absence of a constructed center joint were somewhat wider than the transverse joints and showed somewhat more spalling.

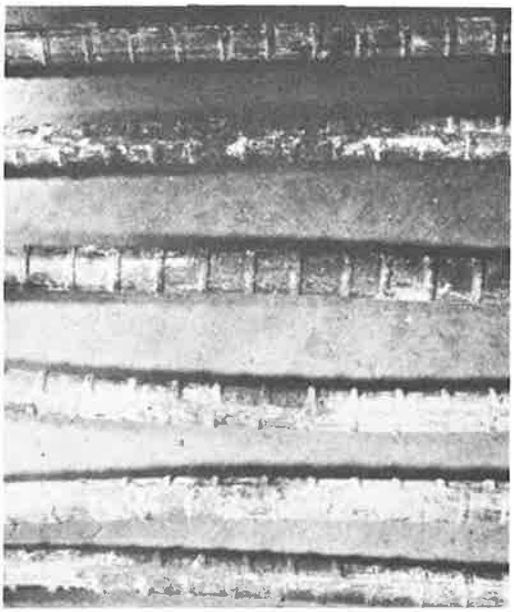

Figure 8. Condition of longitudinal reinforcement bars in the vicinity of transverse cracks.

Removal of the concrete confirmed previous observations that transverse cracks assume hair-like widths below the surface of the pavement. It was evident, too, that a thin layer of soil accumulates in most cracks.

Very slight rusting was found commonly to occur on the surface of the steel at transverse cracks. The rust stains imprinted on removed concrete appeared generally to progress no more than from 1 to 3 in. beyond the cracks. Beyond this point, no rust stain was evident. Observations of the condition of the steel deformation imprints in the concrete showed no evidence of slip between steel and concrete. No evidence of slippage was found in lap areas. The degree of rusting of the longitudinal steel at the transverse cracks was considered to be minor. Typical conditions of bars removed at transverse cracks are shown in Figure 8.

LONGITUDINAL CRACKS

As stated previously, the conventional center joint was omitted from the Illinois experimental continuously reinforced pavement. The No. 3 round deformed bars that were used as transverse reinforcement were extended across the entire width of the pavement spaced at 12-in. centers in half of each test section and at 18-in. centers in the other half.

Longitudinal cracks began to form in some of the test sections soon after construction and now involve most of the experimental pavement. Most of the longitudinal cracking does not coincide with the center of the pavement; however, it rarely meanders more than 3 ft away from the center. In a few instances two parallel longitudinal cracks are present; more rarely, there are three present.

Previous reports have described a relationship that appeared to exist between the time of year of construction and the extent of longitudinal cracking. The pavement constructed during the fall of 1947 showed a much earlier development of longitudinal cracking than did that constructed in the spring of 1948. As will be seen from Table 3, this relationship was still apparent at the end of 10 years. At the end of 20 years, longitudinal cracking has progressed to the extent that the relationship becomes less distinct. No explanation can be offered for the slower development of longitudinal cracks in the spring construction. No other consistent relationships between longitudinal cracking and the variables under consideration, including the two spacings of transverse steel, have been observable.

A considerable amount of surface spall is visible at some of the longitudinal cracks, and the cracks have the general appearance of being open wider than most of the transverse cracks.

In the removal of pavement from section 7 mentioned previously, the concrete was found to have been less protective of the steel, both transverse and longitudinal, at the longitudinal cracks. Transverse bars were often found to be quite severely rusted and sometimes broken at the longitudinal cracks. A "necking down" of the bars and an accompanying fracture in the form of a cup and cone noted in many instances indicate the possibility of tensile deformation before fracture. The stage of rusting of the fracture faces indicated in many instances that the steel had been broken for some time. Longitudinal bars exposed by the formation of longitudinal cracks also showed some rusting. An example appears in Figure 9.

TABLE 3

LONGITUDINAL CRACKING IN TEST SECTIONS

Pavement Thickness (in.)	Section No.	Dates Constructed (inclusive)	Longitudinal Steel (%)	Percent of Section Length Showing Longitudinal Cracking			
				March 1948	Dec. 1948	Sept. 1957	June 1967
7	1	9/25/47 9/30/47	0.3	0	22	57	77
	2	9/30/47 10/ 3/47	0.5	0	30	81	91
	3	10/ 3/47 10/ 7/47	0.7	0	39	63	76
	8	10/14/47 10/17/47	1.0	44	72	81	92
8	4	4/30/48 5/20/48	0.3	—	0	6	56
	5	4/26/48 4/30/48	0.5	—	0	6	84
	6	11/ 6/47 12/ 3/47 4/22/48 4/26/48	0.7	—	0	13	52
	7	10/14/47 10/17/47	1.0	48	68	76	87

The concrete that was removed was found to be of good quality and uniformly well consolidated around the steel. No durability defects were evident. Experience with the formation of longitudinal cracks and the subsequent behavior of the pavement at these cracks indicates that the center joint used in conventional pavement should be used also in continuously reinforced pavement.

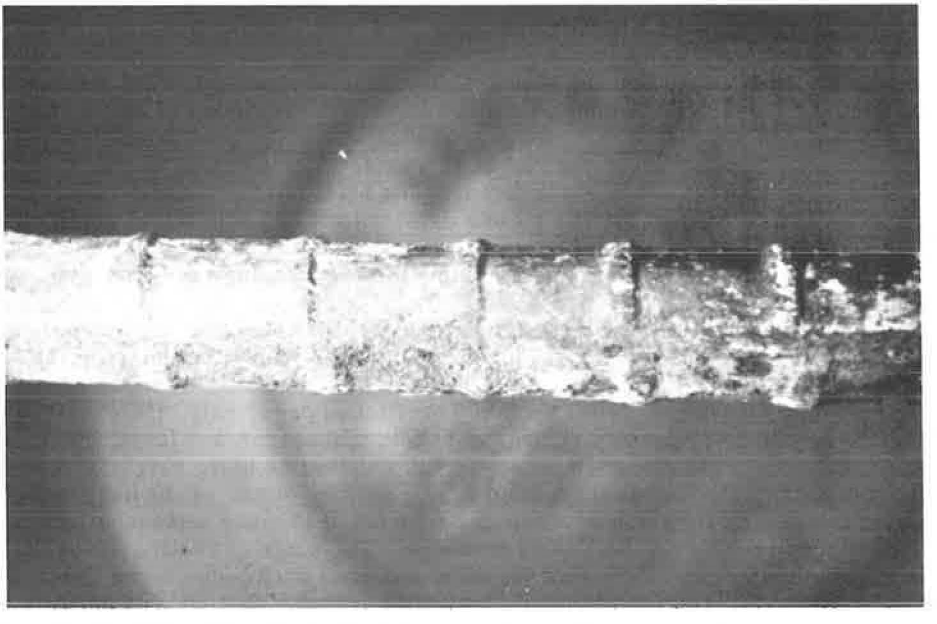

Figure 9. Condition of longitudinal reinforcement bar in the vicinity of a longitudinal crack.

PAVEMENT PUMPING

When the Illinois experimental continuously reinforced pavement was being planned over 20 years ago, it was believed that the absence of joints at closely spaced intervals, the tightness of the cracks, and an inherent flexibility, which would allow the pavement to adjust to minor subgrade movement, would be sufficient to control pumping of fine-grained soils and eliminate the need for a granular subbase. This was of special interest in reducing the total cost of construction. The pavement was therefore constructed on the natural fine-grained subgrade to test this belief. Most of the subgrade soils of the project are of the A-7-4, A-4, A-4-2, and A-6 groups, all known to be capable of pumping under the rainfall conditions (about 38 in. average annual precipitation) that exist at the location of the experimental project. The anticipated volume of heavy truck traffic is sufficient to produce pumping of conventional pavements.

At the time of preparation of the 10-year report, serious pumping had been noted at the expansion joints that were placed between the experimental sections, and at most of the construction joints placed at the end of each day's run in the sections with 0.3 and 0.5 percent steel. The construction joints in sections with 0.7 and 1.0 percent steel showed neither pumping nor faulting. The pumping, where found, was associated with considerable pavement distress that made patching necessary. Pumping also has been excessive at wide transverse cracks in the low-steel-content sections where the longitudinal reinforcement obviously had fractured.

A small amount of edge pumping not associated with joints or open transverse cracks has been observed on the 0.3 and 0.5 percent steel sections. No pavement failures have been observed that can be attributed to this pumping.

The expansion joints were installed in the pavement as part of the experimentation to assure that each test section would act independently. This usage would not be a part of normal construction of continuously reinforced pavement. However, the inherent weakness that is present at unprotected slab ends needs to be recognized when designing terminal systems at such locations as bridges and the ends of construction projects.

Construction work was terminated each day by the placing of a wood header. When concrete placement was resumed, the header was removed and the new concrete placed against the older concrete. No longitudinal reinforcement in addition to that of the normal design of the section was placed at these joints.

Pumping that was visible at construction joints in the 0.3 and 0.5 percent reinforcement sections at the time of the 10-year survey (at all construction joints in the 0.3 percent steel sections and two of seven in the 0.5 percent steel sections) has led subsequently to failure of the pavement at most of these locations, making patching necessary. The joints in the 0.7 and 1.0 percent reinforcement sections are still in good condition after 20 years of service.

The behavior of the pavement at the construction joints is interpreted as indicating a need for additional reinforcement at construction joints if performance equal to that away from the joints is to be achieved.

It has long been known that even slight pumping is likely to progress to a severe stage, and cannot be depended on to remain slight through the ordinary life of a pavement. Although pumping of serious consequence has been confined principally to joints and open cracks in the sections in which the steel content might be considered inadequate, it would appear prudent to include a protective subbase with continuously reinforced pavement where the use of a subbase would seem warranted with conventional pavement.

CHANGE IN LENGTH

The 4-in. expansion joints that were installed between test sections allowed each considerable freedom to change length. In the early years following construction, seasonal movements of 1 to 2 in. between summer and winter, with the sections being larger in the summer, were recorded. By the summer of 1957 all of the joints that were 4 in. wide when constructed were tightly closed. Large spalls indicative of excessive compression were observable at several of the joints and major repairs were required. Subsequently, patching has been required at almost all of these joints.

The cause of the growth of the pavement cannot be stated with certainty. Growth of the concrete itself cannot be excluded entirely as a possibility. However, some readings taken in 1957 between gage plugs which were not influenced by transverse cracks did not show much variation from initial readings, giving indication that growth of the concrete was not a major factor. The most logical explanation of the growth seems to be an incomplete closure of transverse cracks once they form. The finding of slight coatings of soil in crack openings supports this explanation.

The experience gained on this research project, where expansion joints were placed at 3,500 to 4,200-foot intervals, has not been sufficient to allow prediction of the behavior of larger runs, or of the requirements for expansion space. Nor can predictions be made of the amount of restraint that would be required to prevent growth of the pavement. Theoretical considerations suggest that considerable restraint is already present.

STRESSES IN LONGITUDINAL STEEL

For successful performance of continuously reinforced pavements, stresses in the steel must remain below the yield point; otherwise, the cracks would become excessively wide.

During the construction of the Illinois experimental pavement, SR-4 strain gages were mounted on selected longitudinal bars in the 7- and 8-in. sections containing 0.7 percent steel. Many stress readings were taken during the first month following construction, and periodically but less frequently thereafter.

In the fall 1947 construction, a maximum average stress of 62,400 psi was measured at a preformed transverse crack in January 1948. This is within a critical range of the 70, 000-psi yield point of the rail steel bars of the section. Stresses away from the crack never measured higher than 10,000 psi. In the spring 1948 construction, a maximum average stress of 42,000 psi was recorded the following January.

The strain gages became inoperable in a relatively short period of time and did not yield usable data beyond one season of contraction. During their period of operation, the gages showed that the tensile stress of the steel bears an inverse relationship to slab temperature, being high when the temperature is low and low when the temperature is high. The stress in the steel was seen to respond rapidly to daily changes in temperature, and to show considerable variation in a 24-hour period.

The strain gage study is reported in more detail in earlier reports, particularly in the 3-year performance report.

RIDING QUALITY

The Illinois experimental continuously reinforced pavement was judged from the beginning by Illinois Division of Highways engineers to offer better-than-average riding quality. As the years passed, it was agreed generally that the pavement was retaining this characteristic. Roughness measurements that were begun in 1957 when the Division of Highways acquired a Bureau of Public Roads-type roughometer have confirmed the earlier judgments.

The results of roughometer measurements made in 1957, and again in 1962, 1964, and 1967, are given in Table 4. Each value of the roughness index shown is the average for measurements made in each of the four wheelpaths of the two-lane pavement. It will be noted that, with the exception of the roughness index reading of 92 recorded for section 1 (7-in. thick, 0. 3 percent steel) in 1967, all readings, including those for the conventional pavement, indicate riding surfaces in the "smooth" and "very smooth" categories. It will be noted also that, with the exception of section 1, little change in riding quality has taken place during the 10 years that roughometer measurements have been made. Most of the experimental sections have been, and have remained, somewhat smoother than the conventional pavement.

PERFORMANCE

Carey and Irick (4) describe a system for estimating pavement ratings of serviceability and for determining performance by summarizing the serviceability record over

TABLE 4

ROUGHNESS INDEXES FOR EXPERIMENTAL PAVEMENT AND ADJOINING CONVENTIONAL PAVEMENT

Test Section	Steel Content (%)	Pavement Thickness (in.)	Roughness Index (in. per mi)			
			1957	1962	1964	1967
1	0.3	7	73	87	78	92
2	0.5	7	70	81	72	74
3	0.7	7	68	70	79	70
4	0.3	8	71	75	74	78
5	0.5	8	74	79	73	80
6	0.7	8	79	79	76	81
7	1.0	8	71	71	71	70
8	1.0	7	70	70	71	75
Conventional pavement			81	85	78	81

Note: Illinois Adjective Ratings for Roughness of Rigid Pavements:

Roughness Index Range	Adjective Rating
Less than 75	Very smooth
75-89	Smooth
90-124	Slightly rough
125-169	Rough
170-219	Very rough
220 or more	Unsatisfactory

a period of time. The system, for concrete pavements, involves measurements of riding quality, cracking, and patching.

Under the serviceability performance concept, the term "present serviceability" was chosen to represent how well a pavement is serving high-volume, high-speed, mixed truck and passenger vehicle traffic at a specific time. A mathematical index (present serviceability index) was developed for estimating subjective ratings of present serviceability through objective measurements taken on the pavement. The relationship, modified to make use of the roughness index as provided by the Illinois roughometer, can be expressed as follows:

$$P = 12.0 - 4.27 \log \overline{RI} - 0.09 \sqrt{C + P}$$

where

P = Present serviceability index;
$\overline{RI}$ = Roughness index in inches per mile as obtained by the Illinois roughometer;
C = Cracking in lineal feet per 1000 sq ft of pavement area, considering only those cracks that are open or spalled at the surface to a width of $\frac{1}{4}$ in. or more for at least half the crack length, and sealed cracks; and
P = Bituminous patching in square feet per 1000 sq ft of pavement area.

Under the present serviceability concept, the present serviceability rating is expressed on a scale between 0 and 5 as follows:

5
Very good
4
Good
3
Fair
2
Poor
1
Very poor
0

TABLE 5

PRESENT SERVICEABILITY PERFORMANCE OF EXPERIMENTAL PAVEMENT AND ADJOINING CONVENTIONAL PAVEMENT

Test Section	Steel Content (%)	Pavement Thickness (in.)	Year	Cracking (lin. ft/ 1000 sq ft)	Patching (sq ft/ (1000 sq ft)	Roughness Index (in./mi)	Present Serviceability Index
1	0.3	7	1962	0.3	143.6	87	2.6
			1964	14.3	172.5	78	2.7
			1967	45.4	153.0	92	2.4
2	0.5	7	1962	3.9	50.2	81	3.2
			1964	4.6	50.2	72	3.4
			1967	6.8	50.2	74	3.3
3	0.7	7	1962	0.0	0.0	70	4.0
			1964	0.0	0.0	69	4.2
			1967	3.7	0.0	70	4.0
4	0.3	8	1962	8.6	1.7	75	3.7
			1964	15.1	2.8	74	3.6
			1967	34.0	7.6	78	3.3
5	0.5	8	1962	0.5	0.0	79	3.8
			1964	0.3	0.5	73	4.0
			1967	6.3	0.2	80	3.6
6	0.7	8	1962	0.0	0.0	79	3.9
			1964	0.0	5.8	76	3.8
			1967	1.4	11.9	81	3.5
7	1.0	8	1962	0.0	0.0	71	4.1
			1964	0.0	0.3	71	4.1
			1967	0.4	1.9	70	4.0
8	1.0	7	1962	0.0	0.0	70	4.1
			1964	0.0	0.3	71	4.1
			1967	0.0	1.2	75	3.9
Conventional pavement			1962	0.3	0.0	85	3.7
			1964	3.2	0.1	78	3.8
			1967	4.0	0.2	81	3.7

Illinois highway engineers consider a present serviceability index (PSI) of 2.0 or lower to indicate a need for reconstruction or replacement of a two-lane pavement on the primary system.

The results of measurements made in 1962, 1964, and 1967 to compute the PSI's of the experimental pavements, and the computed PSI's, are given in Table 5. The computations and the PSI's for the conventional pavement are also presented.

As mentioned previously, failures in the continuously reinforced pavement sections other than a few that occurred in the 0.3 percent steel content sections and in the 0.5 percent steel section of 7-in. thickness (two isolated occurrences in the latter), have been confined to small areas associated with transverse joints. Evidences that total failure is imminent have been a widening of crack openings, spall at cracks and joints, excessive slab deflection, faulting, and pumping. A series of closely spaced transverse cracks usually becomes involved as failure progresses. If repairs are not made within a reasonable period of time, the individual slabs between cracks, or between cracks and joints when joints are involved, become broken into smaller sections and are pushed into the subgrade soil by passing traffic. The failures not associated with transverse joints that have occurred in the lower steel content sections have been similar except for the absence of joints.

Defects in the conventional pavement have been confined to a very limited number of transverse cracks that are of more than a simple stress-relieving nature and to some spalling at the edged full-depth metal plate contraction joints.

No pavement maintenance has been required in the sections of continuously reinforced pavement of higher steel content except in the few instances where joint weakness has been involved. Maintenance of the conventional control pavement has been limited almost entirely to a twice-yearly application of joint and crack seal. Maintenance cost records are not adequate for use in the study.

TABLE 6

VOLUME AND CHARACTER OF TRAFFIC ON EXPERIMENTAL PAVEMENT AND ADJOINING CONVENTIONAL PAVEMENT

Vehicle Type	Average Daily Traffic						
	1950	1953	1956	1959	1962	1965	1967
Passenger cars	2,060	3,100	2,885	3,000	3,400	4,500	4,810
Single-unit trucks	305	340	335	390	400	525	702
Multiple-unit trucks	300	760	1,030	1,210	1,100	1,475	1,768
Cumulative 18,000-lb single-axle equivalent loadings	207,000	737,000	1,400,000	2,178,000	2,889,000	3,524,000	4,273,000

It will be noted in Table 5 that during the past five years all of the experimental sections with the exception of section 1 (0.3 percent steel), and also the conventional pavement, have rated as "Good" or "Very Good" under the present serviceability concept. Section 1 has rated as "Fair." A fairly consistent relationship exists between steel content and PSI, with pavements of higher steel content showing higher PSI's. The relationship between pavement thickness and PSI is less consistent.

With the possible exception of the 7-in. section having 0.3 percent steel content, all experimental sections and the conventional pavement used as a control are continuing to offer satisfactory service 20 years after construction. Some of the experimental pavements may be providing slightly better service than the conventional pavement; some appear to be providing a slightly lesser degree of service. In general, the sections of greater steel content and greater thickness are providing the better service.

It will be recalled that the conventional pavement was placed on granular subbase while the experimental continuously reinforced pavements were placed on the natural fine-grained soil subgrade. It is also to be remembered that the principal failures in the continuously reinforced pavements took place at expansion joints installed only because of the experimental nature of the project, and at construction joints where the weakness undoubtedly can be overcome through the use of additional steel.

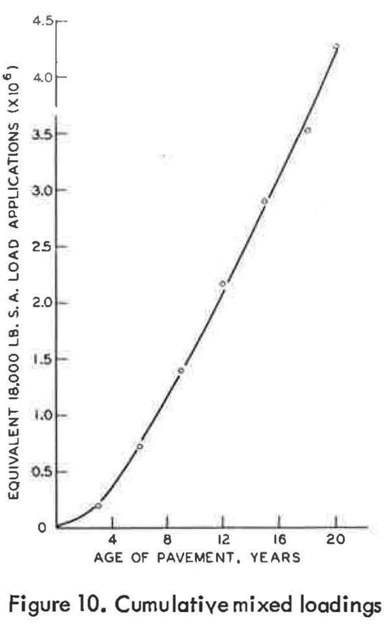

Figure 10. Cumulative mixed loadings.

TRAFFIC

The traffic that the experimental pavements have carried through the years is typical in volume and character of the traffic on many primary highways serving a high percentage of through truck traffic. Average daily traffic volumes for representative years, based on manual and machine counts, and shown separately for passenger cars, single-unit trucks, and multiple-unit trucks, are given in Table 6.

To evaluate the performance of a pavement in relation to the axle loadings to which it has been subjected, it is convenient to represent the varying axle loadings a pavement receives to a single loading having the same effect on the pavement structure. The AASHO performance equations are often used to accomplish this purpose.

Engineers of the Illinois Division of Highways and others have made use of loadometer data and traffic volume count and classification data in conjunction with the AASHO Road Test performance equations to estimate equivalent numbers of a standard axle loading to represent mixed loadings applied to pavements. Usually, the equivalency is established on the basis

of an 18, 000-lb single-axle loading. The system used in Illinois was reported by Chastain (5).

Equivalent 18, 000-lb single-axle loadings have been computed by Illinois procedures to represent cumulative mixed loadings applied to the experimental pavements beginning with the opening of the pavement to traffic. The results of these computations are shown in Figure 10.

SUMMARY OF PRINCIPAL FINDINGS

Observation of the behavior of the Illinois experimental continuously reinforced pavement and an adjacent conventional control pavement over a 20-year service period indicates the following:

1. Transverse cracks begin to develop at closely spaced intervals in continuously reinforced concrete pavements soon after construction. The number of cracks increases with age, but at a very slow rate after the first few years. Crack frequency increases with the amount of longitudinal steel.
2. For the sections under study, no strong differences occurred between the behavior of the 7-in. and 8-in. thicknesses of pavements.
3. Crack width is an important factor in the behavior of continuously reinforced pavements. Narrow cracks are essential to the maintenance of structural integrity of the pavement system. Crack width is a function of steel content, with lesser width being associated with higher steel content. Crack width increases slowly with age.
4. Slight spall begins to occur at transverse cracks in continuously reinforced pavement soon after construction. The spall increases with age and is related in amount inversely to the amount of longitudinal steel. Within the 20 years of service of the experimental pavements, spall at transverse cracks has not become a defect requiring maintenance.
5. Meandering longitudinal cracks will occur in a continuously reinforced pavement constructed wider than one lane without a center joint. Within the transverse steel contents investigated (No. 3 bars at 12- and 18-in. centers), the longitudinal cracks can become unsightly and a source of structural weakness within the expected life of a pavement.
6. Construction joints are potential sources of weakness at longitudinal steel contents of less than 0.7 percent.
7. When longitudinal movement is unrestrained, seasonal movements of continuously reinforced pavements can be expected, and also permanent increases in length with age. The cause of the growth was not apparent from examination of the existing experimental pavements.
8. Continuously reinforced pavements can be designed and constructed to serve at least as effectively as conventional pavements, and have the potential of overcoming the basic weakness that occurs at transverse joints of conventional pavements.
9. Continuously reinforced pavements can be built to, and can retain for long periods of time (at least 20 years), a high standard of surface smoothness.
10. While pumping has not been a major cause of failure of the experimental pavement during 20 years of service, it has nevertheless been sufficient in amount to indicate that continuous reinforcement is not a positive protection against pumping.

DESIGN RECOMMENDATIONS

Considering the traffic loadings to which the experimental continuously reinforced pavement has been subjected, it would appear that:

1. A 7-in. thickness of pavement will probably be adequate. However, unless future loadings can be predicted with a fair degree of certainty, conservatism would suggest an 8-in. thickness.
2. The selection of the amount of longitudinal steel requires considerable judgment. The general appearance of the 0.3 percent longitudinal steel sections after 20 years suggests that satisfactory performance through many more years cannot be expected.

The 0.5 percent steel sections are still performing well, and this steel content can be considered a minimum. The 0.7 percent steel sections have a generally better overall appearance (slightly less spall at transverse cracks), but to date can be said not to be performing significantly better. Again, conservatism would suggest a longitudinal steel content of perhaps 0.6 percent.

3. The extent and condition of the longitudinal cracks that developed in the absence of a center joint leave no doubt as to the need for a center joint.

4. The magnitude of the movement that has taken place at unrestrained section ends suggests that some further study of the control of slab ends is needed. Whether some form of restraint needs to be provided, or whether an improved expansion joint that will provide a positive means of load transfer or of slab support is needed, is not known.

5. Some form of subbase to control pumping is suggested.

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