A Ten-Year Report on the Illinois Continuously-Reinforced Pavement

J. D. LINDSAY, Engineer of Materials,
Illinois Division of Highways, Springfield, Illinois

In the fall of 1947 and the spring of 1948 the Illinois Division of Highways constructed on US 40 an experimental continuously-reinforced pavement consisting of eight test sections ranging in length from 3,500 to 4,230 ft. Four sections were uniform 7-in. pavement and four were uniform 8-in. pavement. The pavement was constructed directly on natural subgrade, 90 percent of which was composed of soils classified as potentially pumping types.

Longitudinal steel amounting to 0.3, 0.5, 0.7, and 1.0 percent of the gross cross-sectional area of the pavement was used with each pavement thickness. The longitudinal reinforcement consists of round deformed rail-steel bars. Number 3 round deformed intermediate grade, billet-steel bars at 12-in. centers in one-half of each section and at 18-in. centers in the other half were used as transverse reinforcement.

The pavement has been carefully observed and detailed surveys to determine its behavior and condition have been made periodically.

This report describes the behavior and performance of the pavement during the 10-year period. The behavior of the pavement, particularly of the more lightly reinforced sections, has been beyond expectations. All of the test sections have given good performance. There have been a few structural failures in the lighter reinforced sections; in those cases local conditions, such as poor subgrade, have been the primary factors. Pumping has been restricted mainly to the 4-in. expansion joints and the construction joints, which are notably points of weakness.

All test sections developed large numbers of transverse cracks, the frequency being proportional to the amount of longitudinal steel. Crack development has been progressive with time, but at a decreasing rate. Crack widths are inversely proportional to the amount of longitudinal steel, but in most cases the cracks have remained narrow, indicating that the steel is effective in holding adjacent slabs together. Only moderate ravelling has occurred at transverse cracks and there has been almost no faulting across cracks. The pavement is noticeably smooth riding, which is further indicated by a recent roughometer test that gave an average reading of less than 72 in. per mile.

There has been a progressive increase in the length of all test sections, resulting in full closure of the 4-in. expansion joints. At several of these joints high localized compressive stresses, probably caused by irregular interfaces, have resulted in rather large surface spalls that have required some maintenance. These repairs, limited amounts of undersealing to reduce pumping at expansion joints and construction joints, and repairs of a few local structural failures, account for the only slab maintenance performed during the 10-year period.

●IN 1947-48 the Illinois Division of Highways, in cooperation with the U. S. Bureau of Public Roads, constructed an experimental pavement to study the effect of continuous reinforcement in long unjointed pavement slabs and to determine, if possible the proper amount of steel necessary to provide a pavement of outstanding performance and service life. Construction began September 25, 1947, and the pavement was completed May 20, 1948.

The design and construction details have been covered in previous reports (1, 2,). It seems desirable, however, to summarize briefly the important facts about its design and construction.

The pavement, which is 22 ft wide and approximately 5½ mi long, is divided into eight test sections, six of which are approximately 3,500 ft and two about 4,230 ft long. The longitudinal reinforcement is continuous from end to end in each test section; consisting of round deformed bars (ASTM 305-47T) meeting the requirements of ASTM Designation A 16 for rail-steel bars. The pavement is reinforced transversely with No. 3 round deformed bars meeting the requirements of ASTM Designation A 15 for intermediate grade billet-steel bars. In one-half of each test section the transverse bars are spaced on 12-in. centers; in the other half, on 18-in. centers. The transverse bars extend the full width of the pavement and the customary center joint was omitted. The physical properties of the reinforcing bars, determined from mill tests, are given in Table 1.

Four of the test sections are uniformly 7 in. thick and four are 8 in. thick. Four percentages of longitudinal steel, based on the gross cross-sectional area of the pavement, were used with each thickness of pavement; namely, 0.3, 0.5, 0.7, and 1.0 percent. These percentages were obtained by using different bar sizes and varying their spacing. The reinforcement bars were assembled as a continuous mat on the subgrade and supported by means of chairs approximately 3 in. below the finished surface of the pavement. Air-entrained concrete was poured in one lift, being deposited through the reinforcement and consolidated and finished by means of a conventional spreader and finishing machine and regular hand operations.

The pavement has been the subject of frequent periodic observations and measurements from the time of its construction in order that a thorough record of its behavior and performance would be obtained. Extensive surveys have been made, usually twice a year (one in extremely warm weather and another in cold). At times, however, cold weather observations had to be omitted because of snow and ice. Observations have also been made immediately following periods of heavy rainfall.

It is the purpose of this report to discuss the data obtained, to describe the performance of the pavement during the 10-year period following its construction, to describe its present condition, and to offer some suggestions as to design of continuously-reinforced pavements.

DEVELOPMENT OF TRANSVERSE CRACKS

One of the distinguishing characteristics of a continuously-reinforced pavement is that transverse cracks develop at very close intervals, the frequency of crack occurrence varying with the amount of longitudinal steel. The cracking pattern near the ends of a long continuously-reinforced slab is similar to that in conventional pavements, but the frequency gradually increases away from the ends and, in long sections, reaches a maximum, which is fairly constant throughout the central portion of the slab (2, 3, 4).

Figure 1 shows the distribution of cracking throughout each of the sections of 7-in. pavement at the age of 3 years and again at 10 years. It will be seen that in 1950 and 1957 the frequency for successive 100-ft increments increased at a reasonably uniform rate from the ends of the slab to a maximum at a distance ranging from 200 to 500 ft from the ends, then decreased uniformly for several hundred feet, and finally became fairly stable over the central 1,600 to 2,000 ft.

It is interesting to note the obvious relationship between crack frequency and the percentage of longitudinal steel. The data for 1950 and 1957 show definitely that the

TABLE 1

RESULTS OF MILL TESTS OF REINFORCING STEEL

Bar No. (in.)	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

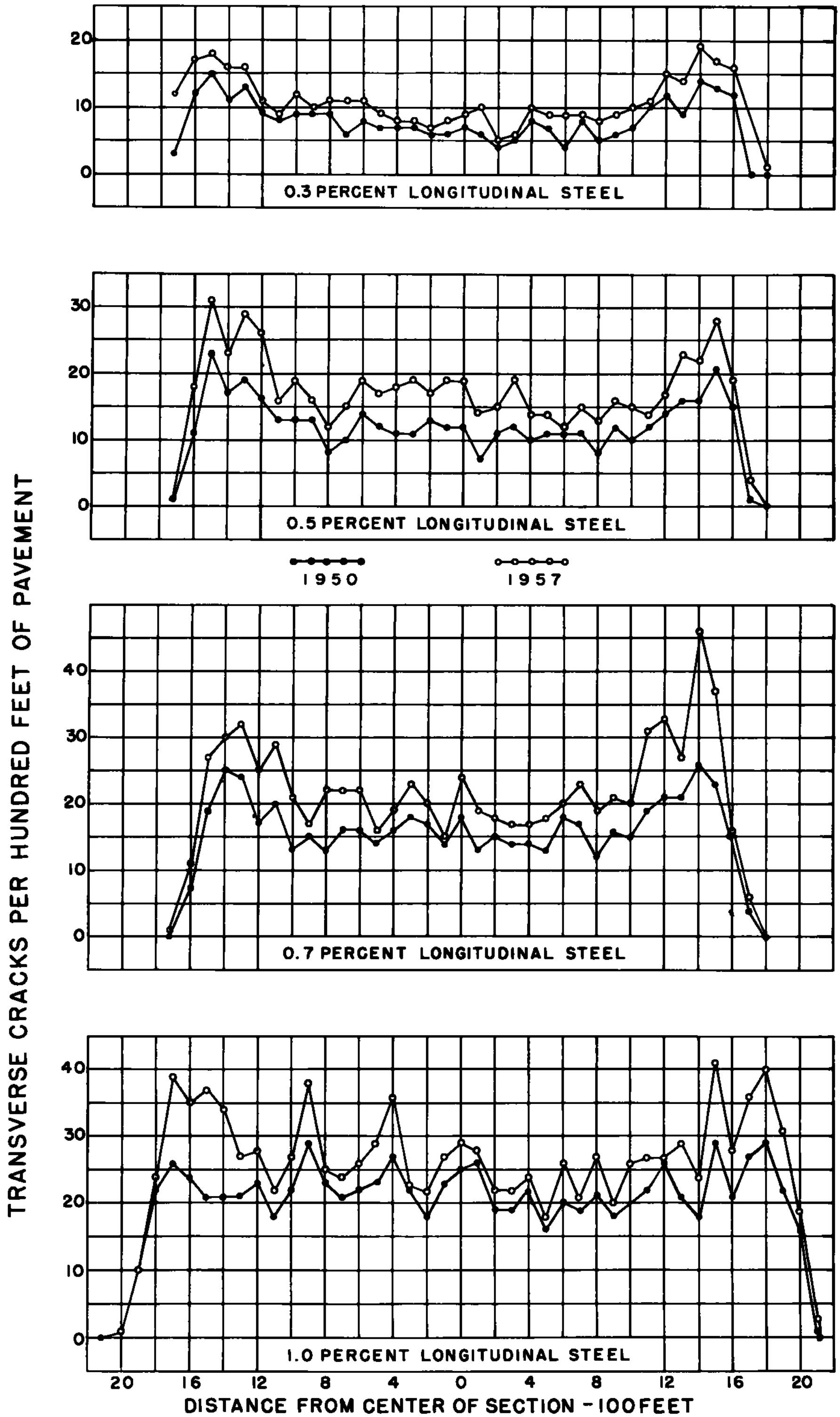

Figure 1. Comparison of frequency distribution of transverse cracks for all sections, 7-in. pavement, 1950 and 1957.

number of cracks increased with the increase in the amount of longitudinal steel. This relationship is illustrated more graphically in Figure 2, which shows the average crack intervals in the four sections of 7-in. pavement at six ages. The first survey was made

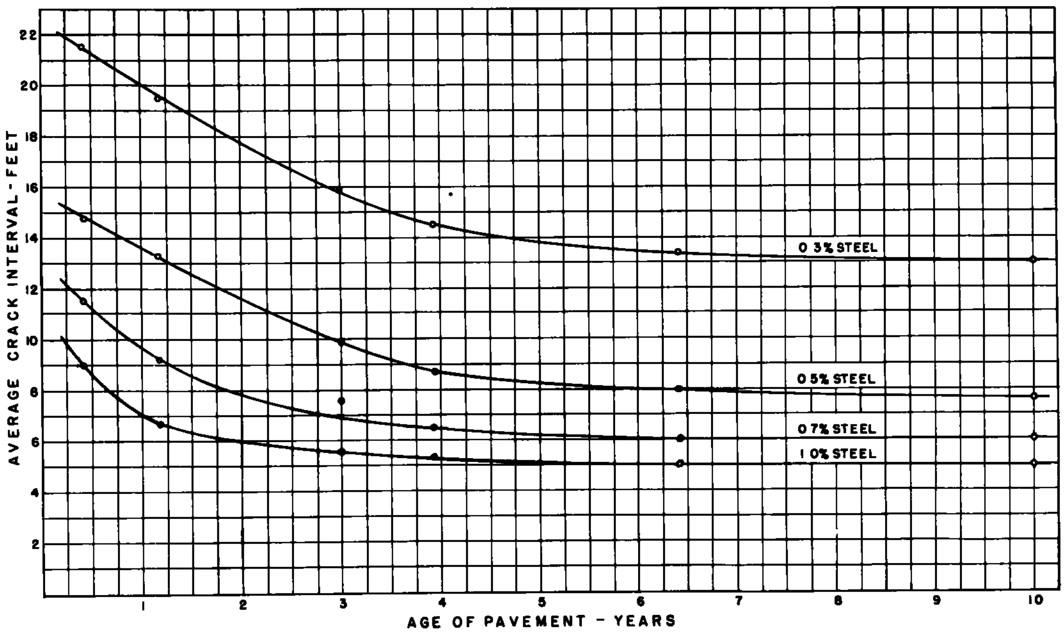

Figure 2. Relationship between average transverse crack interval and age, 7-in. pavement.

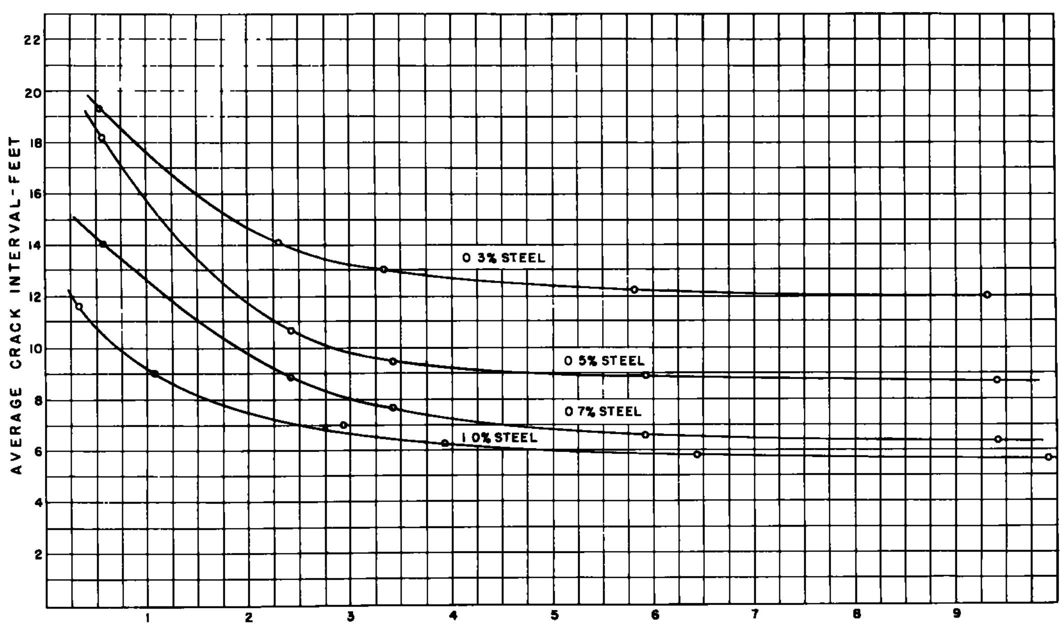

Figure 3. Relationship between average transverse crack interval and age, 8-in. pavement.

in March 1948 after the sections had gone through one winter and were less than six months old. Other surveys were made at approximately 1, 3, 4, 6, and 10 years. This chart shows a definite relationship between frequency of cracks and percentage of steel. Taking the results of the first survey as an example, the average crack intervals were 21.5 ft for the 0.3 percent section, 14.8 ft for the 0.5 percent section, 11.5 ft for the 0.7 percent section, and 9.0 ft for the 1.0 percent section. Expressed as frequency, these values correspond to 4.6, 6.8, 8.7, and 11.1 cracks per 100 ft. Similar relationships are also apparent for the data from the other surveys.

Another significant relationship shown by Figure 2 is that between crack development and age. It is seen that for each section the average crack interval decreased at a rather rapid rate for the first 3 or 4 years and afterward cracks developed at an increasingly slower rate until, at about 6 years the curves became very flat. It can be concluded, therefore, that transverse cracking has practically reached its equilibrium after 10 years and, barring unforeseen structural failures, few new transverse cracks may be expected to occur during the service life of the pavement. It is also apparent from Figure 2 that the development of transverse cracks approached an equilibrium somewhat earlier in the more heavily reinforced sections. The behavior of the 8-in. pavement sections with respect to crack development has been similar to that of the 7-in. sections, as shown by Figure 3.

CRACK WIDTH

Perhaps the requirement most necessary to the success of continuously-reinforced pavement is that the steel reinforcement hold transverse cracks to a narrow width. There are three reasons why narrow transverse cracks are essential. First, they must be narrow to prevent the progressive infiltration of incompressible materials, such as soil, which eventually might cause excessive compressive stress to develop in the pavement and produce blowups. Secondly, the transverse cracks must not admit appreciable amounts of surface water to the subgrade and, by the same token, if the pavement happens to be built directly on soils which are of the potentially pumping types, such as is the case with the Illinois pavement, the cracks must be maintained so tight that pumpable material cannot be ejected through them. Thirdly, the cracks must be held tightly closed so as to maintain effective aggregate interlock between the crack interfaces. The importance of the latter will be shown later in the discussion of failures that occurred at construction joints where no aggregate interlock was present.

The accurate measurement of the width of transverse cracks is difficult. At the time the Illinois pavement was constructed a series of brass reference plugs on 10-in. centers was installed along the edge of the central 30 ft of each test section for the purpose of measuring the widths of cracks which subsequently might form in the areas. Unfortunately, enough cracks did not form between the plugs to give reliable data. Foreseeing that this might occur, a measuring microscope capable of measuring to 0.001 in. was procured and, following the first crack survey in March 1948, 60 transverse cracks were selected from each test section for a program of width measurements, which would be continued as long as the pavement was under active observation.

The cracks were chosen to be representative of those throughout the entire length of the test section by selecting a group of 10 at each end of the section, another 10 at each quarter point, and a group of 20 cracks at the center of each section. Measurements have been made periodically since July 28, 1948, when the first set of readings was taken.

In an attempt to obtain measurements which would be more nearly representative of the true width of the cracks than would be the case if surface widths were measured, the microscope was focused some distance down in the crack and the width was measured at that point. Usually it was possible to register on the matching faces of a broken piece of aggregate. Although this method may not be extremely accurate, it nevertheless is believed that it gave fairly reliable results, although the widths so measured may be somewhat greater than the actual widths.

The data on crack width measurements at the 60 representative cracks are shown in Figure 4 for the 7-in. sections and in Figure 5 for the 8-in. sections. The data for the 8-in. sections are not as complete as those for the 7-in. sections because on several occasions it was not possible to take measurements on all the sections. Nevertheless, they show similar trends.

Except for the July 7, 1949, series of readings, in the case of both 7- and 8-in. pavements and the August 10, 1951, series, in the case of the 8-in. pavement, there is

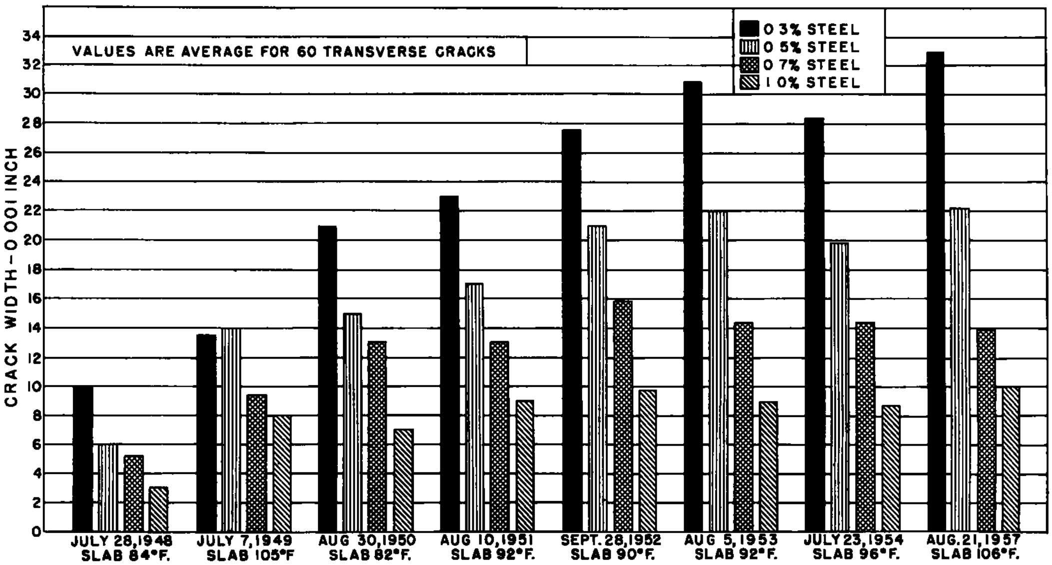

Figure 4. Effect of amount of longitudinal reinforcing steel on width of representative cracks, 7-in. sections.

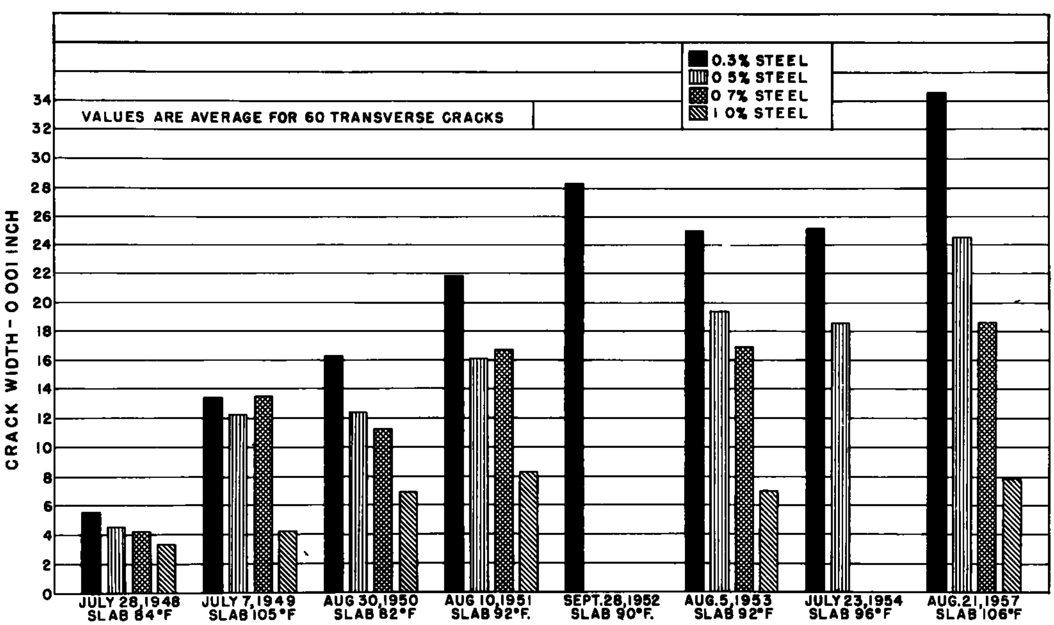

Figure 5. Effect of amount of longitudinal reinforcing steel on width of representative cracks, 8-in. sections.

a definite correlation between crack width and the percentage of longitudinal steel, the crack widths becoming progressively smaller as the percentage of steel increases.

It is readily apparent from these charts that the cracks have become wider with time and also that the influence of time is less in the more heavily reinforced sections. In the 9 years since the first measurements were made on the 7-in. sections, the average crack width in the 0.3 percent section has been increased by 0.023 in.; that in the 0.5 percent section by 0.016 in.; that in the 0.7 percent section by 0.009 in.; and that in the 1.0 percent section by 0.006 in.

The corresponding values for the 8-in. pavement are 0.028 in. for the 0.3 percent steel; 0.020 in. for the 0.5 percent steel; 0.014 in. for the 0.7 percent steel, and 0.0045 in. for the 1.0 percent steel. Except for the section with 1.0 percent steel, the cracks in the 8-in. pavement have shown greater increase in width than those in the 7-in. pavement. The probable reason for this is that 7-in. sections, being built in the fall of 1947, were on the average approximately 6 months older than the 8-in. sections (except Section 7 with 1.0 percent steel, which was also built in 1947) when the initial readings were taken. This difference in age may account for the crack widths at the time of the initial readings being greater in the 7-in. pavement than in the 8-in. pavement.

The appearance of the cracks for the most part is good. Although there has been some raveling along the edges, their appearance is by no means unsightly, even on close inspection. In fact they are not visible from a vehicle travelling 30 mph, except at a few locations where maintenance forces, misunderstanding instructions, poured a number of cracks with asphalt. Figure 6 shows a typical crack in each section of 7-in. pavement at the age of 3 years and the same cracks when the pavement was approximately 10 years old.

What is of real significance is whether the cracks have become so wide that they fail to meet the criteria previously stated; that is, do they permit infiltration of soil, do they allow entrance of appreciable amounts of surface water, and have they lost effective aggregate interlock?

The value of the steel in preserving aggregate interlock appears to be unquestioned. Except at a relatively few cracks in the 0.3 percent sections, no faulting has occurred at transverse cracks. This is in sharp contrast with the serious faulting which has developed at a number of construction joints in the lighter reinforced sections.

Whether the cracks meet the other two requirements is a difficult question to answer. Based on the measurements made, it appears that many of the cracks, particularly those in the 0.3 and 0.5 percent sections, now are so wide that they would allow infiltration of fine incompressible material and entrance and ejection of water. It has been stated previously that because of the method of measurement the measured widths may be somewhat wider than the true widths.

Cashell and Teske (5) has stated: "The real widths of all cracks are many times smaller than their respective surface widths." They reported a surface width of approximately 0.08 in. in pavement with 1.0 percent of steel at the age of 10 years. It is understood that these measurements were made at the surface and thus were influenced to a greater extent by raveling than the measurements made on the Illinois pavement. From limited measurements made midway down the edge of the pavement at the age of 10 years, Indiana found the real widths of cracks in the sections containing 1.82, 0.45, and 0.24 percent steel to be 0.004, 0.011, and 0.013 in., respectively. Based on these data, it would appear that the measured widths obtained in Illinois probably exceed the true widths.

It is understood that cores recently drilled through cracks in the New Jersey continuously-reinforced pavement (6) showed that cracks which appeared very wide on the surface tapered sharply to a fine crack a short distance from the surface. To determine whether such was the case in the Illinois pavement, a core was drilled through at least one of the representative cracks in each test section. Examination of these cores indicated that the cracks were widest at the surface, tapered gradually in width to the approximate depth of the steel, then remained fairly constant in width to the bottom of the pavement. For example, microscope measurements of a core from Section 1 (0.3

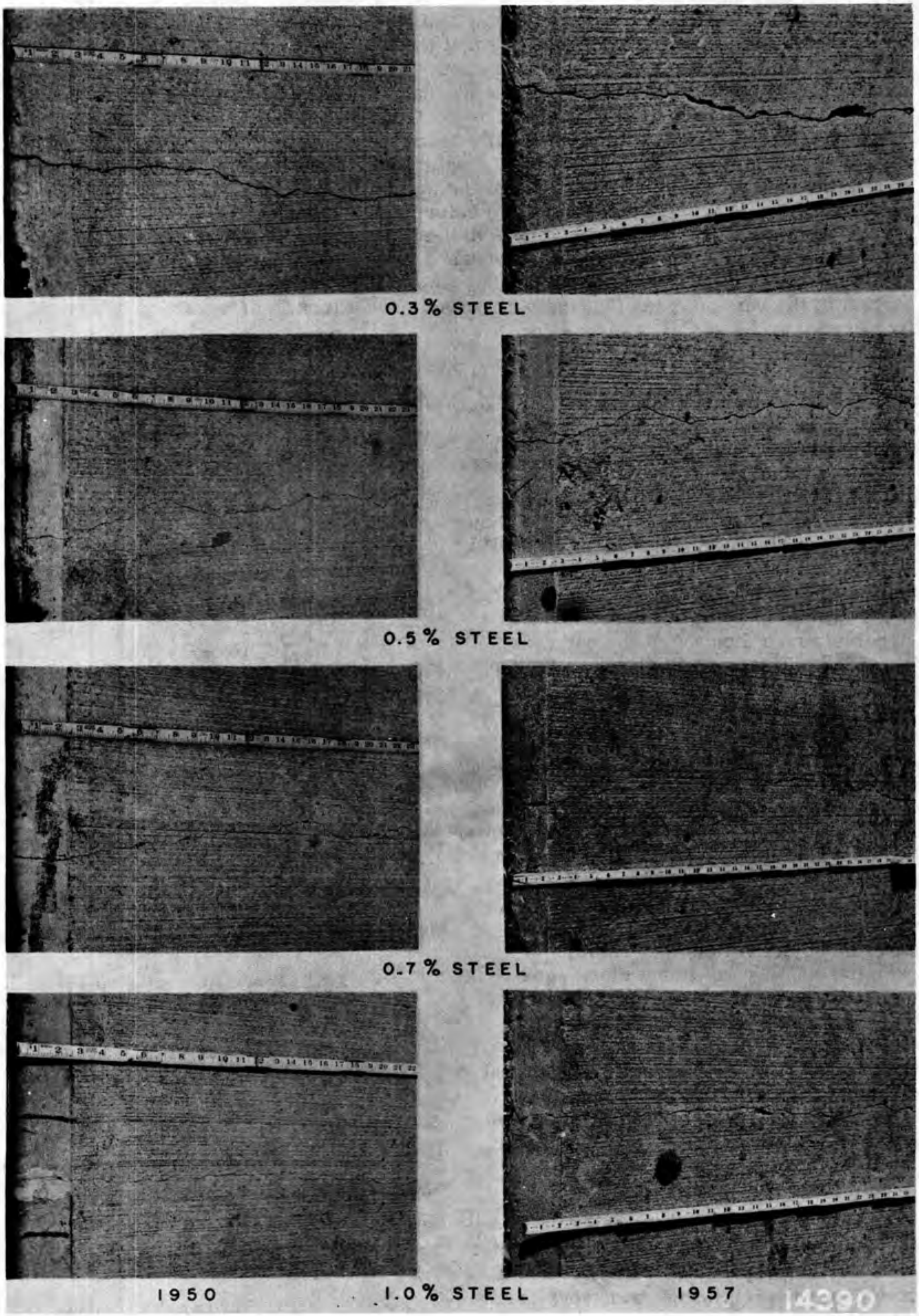

Figure 6. Typical transverse cracks in 7-in. sections, 1950-57.

percent, 7-in.) showed a width of 0.040 in. slightly below the surface, 0.025 in. at 1-in. and 2-in. depths, 0.020 in. at 3 in., and approximately 0.015 in. for the remainder of the depth. It appears, therefore, that the real width of cracks is less than the values given in Figures 4 and 5. On the basis of limited data, perhaps a reasonable approximation of the real width would be one-half the measured surface width.

The cores tend to verify that soil has collected in the cracks. A thin layer of soil was found in the crack in all but one of the cores, and it appears that the permanent opening of the cracks may be due to this condition.

A 4-in. core was taken through one of the cracks in Section 1 which has become quite wide and where faulting has occurred, to determine whether the reinforcing bars have failed, as was suspected. The core was drilled so as to intersect one bar. The bar was found to be necked down and fractured where it intersected the crack. Although there was some evidence of corrosion, the presence of deformations on the "necked down" portion of the bar indicates that the bar had suffered a tensile deformation before it fractured.

Although there are a number of these wide cracks in the 0.3 percent sections, they are by no means representative. To check on the condition of the steel at a more representative crack, a 2-in. core was taken so as to intersect a bar at a crack just 13 ft from the one in which the steel was broken. The crack width at the depth of the steel was approximately 0.015 in. and the bar appeared to be tightly bonded in the concrete, even in the region of the crack, in spite of the presence of a sizeable air void on top of the bar near the crack. This core is shown in Figure 7.

Figure 7. Core through representative transverse crack in 7-in. pavement with 0.3 percent steel.

An interesting sidelight of the coring was that it showed how closely the steel was set to the spacings given in the plans. Eleven 2-in. cores and one 4-in. core were drilled. The locations for ten of the cores were selected so as to fall between two bars and the other two to intersect a reinforcing bar. These locations were determined by measurements based on the planned spacings of the bars. In every case the objective was successfully accomplished. The apparent close agreement between planned and actual spacings thus demonstrated is undoubtedly due to the ingenious design of the chair bars, which supported the longitudinal steel and provided positioning stops at regular intervals for controlling the location of the longitudinal bars.

LONGITUDINAL CRACKS

As has been stated, the pavement was constructed without the conventional center joint for the purpose of finding out how a pavement with continuous transverse reinforcement would act. The results have been quite variable. The five sections constructed in the fall of 1947 all developed a considerable amount of longitudinal cracking the first year. In some locations there were two parallel cracks and in at least two places there were three.

On the other hand, the three sections constructed in the spring of 1948 have developed relatively little transverse cracking. Table 2 gives the projected centerline lengths of longitudinal cracks in each section at six ages. The cracking naturally does not coincide with the centerline of the pavement, but follows an irregular path, meandering as much as 3 ft on either side of the centerline.

In the early years there was a quite definite relationship between the length of longitudinal cracking and the amount of longitudinal steel in the 7-in. sections, and this is still apparent after 10 years in the case of the 0.3, 0.7, and 1.0 percent sections. But for some unknown reason the 0.5 percent section does not follow the same pattern.

The surface width of the longitudinal cracks is probably greater than any of the transverse cracks, although no attempts have been made to measure them because of the extreme hazard involved. However, one core drilled through a representative longitudinal crack in Section 8 (7-in., 0.3 percent) showed that the crack was several times wider than that in the core through a representative transverse crack. The longitudinal cracks are easily visible from a vehicle travelling at normal driving speeds. They probably are more noticeable because there is a continuity of vision and the eye never loses them. Aside from presenting an undesirable appearance, they seem to have had no apparent undesirable effects. Some engineers in the Division have expressed the opinion that they should be sealed, but there seems to be no real need for this. Sealing would be a hazardous task and, unless a material was used whose color blended with that of the concrete, the unsightliness would be increased. Based on this experience, it is believed that a continuously-reinforced pavement should have a joint along the centerline.

PAVEMENT PUMPING

At the time the pavement was planned it was believed, largely on the basis of the behavior of the Indiana pavement and on analytical concepts, that properly designed continuously-reinforced pavement, by reason of the absence of joints, the tightness of cracks, and an inherent flexibility, which would permit it to conform closely to the subgrade, should develop little or no pumping even when constructed over a pumpable subgrade. Also it was realized that if this were true the elimination of a granular subbase would tend to offset the higher cost of the heavy reinforcement. It appeared

TABLE 2

LONGITUDINAL CRACKING IN TEST SECTIONS

Pavement Thickness (in.)	Section No.	Date Constructed (inclusive)	Longitudinal Steel (%)	Longitudinal Cracking (%)					
				March 1948	Dec. 1948	Sept. 1950	Sept. 1951	March 1954	Sept. 1957
7	1	9/25/47 9/30/47	0.3	0	28	34	42	54	64
	2	9/30/47 10/3/47	0.5	0	31	49	72	86	94
	3	10/3/47 10/7/47	0.7	0	42	58	64	66	72
	8	10/14/47 10/17/47	1.0	46	80	82	95	96	97
8	4	4/30/48 5/20/48	0.3	-	0	1	2	3	6
	5	4/26/48 4/30/48	0.5	-	0	<1	3	5	6
	6	11/6/47 12/3/47 4/22/48 4/26/48	0.7	-	0	<1	4	12	13
	7	10/14/47 10/17/47	1.0	52	74	79	83	84	85

logical, therefore, to build the pavement on the natural subgrade in order to confirm these assumptions.

The subgrade is composed of soils of the following groups in the amounts given:

	%
Silty clay and clay of the A-7-4 group	35
Silty clay loam, silty clay, and clay loam of the A-4 group	33
Clay loam of the A-4-2 group	11
Clay of the A-6 group	10
Sandy loam of the A-2 group	11

The A-4, A-6, and A-7 soils are well known for their pumping characteristics when water is present and heavy vehicles in sufficient number travel on a pavement built over them. Hence, 89 percent of the pavement is built over potentially pumping soils.

There is no question about volume and weight of vehicles being sufficient to produce pumping. Table 3 shows the average 24-hr traffic volumes (both directions) in 1950, 1953, 1956, and 1957 through August. It is significant that in that period the number of tractor-semitrailer units has increased from 300 to 1,020 vehicles per day, or 240 percent. Table 4 shows the monthly rainfall from the time the pavement was opened to traffic. It is apparent, therefore, that all the factors necessary to produce pumping have been present many times during the 10-year period.

Serious pumping has occurred at the expansion joints, which separate the sections, and at some of the construction joints, which were built at the end of each day's run. Pumping has been so severe at the expansion joints that the ends of the adjacent slabs at some of them have broken so badly that the concrete had to be removed and replaced with concrete patches. Although pumping at these joints has created a maintenance problem, it should be remembered that they are not a necessary part of a continuously reinforced pavement, having been installed in the experimental pavement only for the purpose of permitting the various test sections to act independently of one another, and the distress at these locations should be disregarded in any evaluation of the performance of this pavement.

Pumping has occurred at one time or another at most of the construction joints in the 0.3 and 0.5 percent sections in both the 7- and 8-in. pavements. At some of these locations major maintenance, such as removal and replacement of small areas, has been required. When the pavement was last surveyed on October 28, 1957, all three of the construction joints in Section 1 (7-in., 0.3 percent steel), two of the three construction joints in Section 2 (7-in., 0.5 percent steel), and all three of the construction joints in Section 4 (8-in., 0.3 percent steel) showed evidence of rather severe pumping.

TABLE 3

TRAFFIC VOLUMES[1] ON EXPERIMENTAL PAVEMENT

Type of Vehicle	1950	1953	1956	1957[2]
Passenger cars	2,060	3,100	2,885	2,850
Single-unit trucks	305	340	335	330
Semitrailers	300	760	1,030	1,020

[1] Average 24-hr count.
[2] January through August.

TABLE 4

MONTHLY RAINFALL IN AREA OF EXPERIMENTAL PAVEMENT[1]

	Rainfall (in.)												
Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual Total
1948	2.26	2.00	7.17	0.87	3.72	6.86	6.07	1.64	2.96	3.95	4.97	2.81	45.28
1949	7.65	3.46	2.97	0.83	4.61	5.01	2.00	1.86	3.97	7.91	-	4.45	
1950	9.06	4.85	2.38	5.00	1.44	4.46	3.25	3.50	1.69	1.38	3.75	0.98	41.49
1951	2.41	4.07	2.73	2.88	1.82	11.54	8.32	1.59	3.03	3.12	3.00	2.67	47.18
1952	0.95	1.56	5.44	3.60	1.90	2.20	4.12	1.35	3.85	0.19	3.00	2.02	30.18
1953	1.03	1.46	4.79	4.74	2.30	3.13	2.33	0.43	0.93	1.84	0.68	1.23	24.89
1954	2.01	0.98	1.21	3.06	2.28	-	-	3.71	1.90	-	0.78	1.62	
1955	2.36	3.02	2.38	4.08	4.17	2.63	6.42	3.30	3.20	4.03	2.92	0.22	38.73
1956	0.97	4.28	1.28	3.20	5.21	3.11	4.00	3.02	1.19	0.94	3.00	4.14	34.42
1957	1.47	2.26	1.73	8.41	9.33	11.58	10.77	3.47	-	-	-	-	

[1] Data from U. S. Weather Bureau station at Vandalia Airport, about 2 mi north of experimental pavement.

All of the joints in Section 1 were open about ¼ in. and it is quite probable that the reinforcing bars are broken. One of the four construction joints in Section 5 (8-in., 0.5 percent steel) was pumping slightly. All construction joints in the sections containing 0.7 and 1.0 percent steel were in excellent condition, were not faulted, and showed no evidence of pumping. This suggests that bars across construction joints should be at least No. 5 at 6-in. centers. Another way of strengthening construction joints would be to use auxiliary plain round dowels. Figure 8 shows pumping along the south edge of the pavement in the vicinity of the construction joint at Sta. 190+40 in Section 1.

In an earlier report (2) it was stated that relatively few cracks in the 8-in. section with 0.3 percent steel, and the 7-in. section with 0.5 percent steel, had shown soil stains, which might be considered as incipient pumping, but that the condition had later improved. It was thought at the time that perhaps the soil collecting in the narrow cracks had acted as a seal. This was confirmed recently when cores drilled over a number of cracks showed a deposit of soil in the bottom of the cracks. Pumping since has been observed at some of the wider cracks in the 0.3 and 0.5 percent sections, but this is by no means a representative condition, as these cracks are ones where broken steel is suspected. In the 0.7 and 1.0 percent sections no pumping has been observed through cracks.

A little edge pumping away from expansion joints and construction joints has been observed in some sections, particularly those with 0.3 and 0.5 percent steel. This pumping is not extensive or severe and does not seem to have affected the performance

Figure 8. Edge pumping near construction joint in 7-in. pavement with 0.3 percent steel.

of the pavement. It seems to occur particularly where the shoulders have become considerably lower than the pavement surface.

In a previous report (2) mention was made of the presence of vertical holes in the earth shoulders where they meet the edge of the pavement. At that time the conclusion was reached that these holes were not caused by pumping, as there was no evidence that subgrade material had been ejected through them. Because at that time they were predominately along the south edge of the pavement, where it was known that the shoulder material had not been as well compacted as along the north edge, it was thought that the holes were the result of water flowing down the edge of the pavement to fill cavities in the soil caused by subsequent settlement of shoulder and subgrade material, and also the space between the subgrade and the slab when the edges of the pavement are curled upward. Later, many of these holes disappeared and it was thought that shoulder maintenance tended to correct the condition.

These holes are still found, particularly along the 0.3 percent sections. They appear to come and go, being more prevalent at one time than another. Careful examinations of these holes have shown no evidence that subgrade material has been ejected. A number have been carefully dug out, and in no case have cavities been found under the edge of the slab. In fact, the slab was observed to be in close contact with the subgrade.

Although pumping, except at expansion joints, which would not be incorporated in a normal continuously-reinforced pavement, and at construction joints, whose weaknesses are now recognized and can be corrected by providing additional load transfer through the use of heavier reinforcing bars or dowel bars, has not been extensive, some pumping has occurred outside these areas.

It is well known that pumping, even though it starts in a small way, is likely to become progressive, and it probably would be a wise practice in anything but an experimental pavement to use a layer of granular material under a continuously-reinforced pavement as a preventive measure where high volumes of heavy vehicles are anticipated. Considering that pumping was not general or of serious consequence away from joints, it is believed that in general a 3-in. layer of granular material would be sufficient.

CHANGE IN LENGTH

The 4-in. expansion joints, which separated the test sections from one another and from the standard pavement adjoining each end of the experimental pavement, were installed so that each section would have considerable freedom to change length independently. Permanent reference monuments were provided at the ends and at selected locations along each section for measurement of longitudinal and vertical movements. Changes in length which have been measured on the sections of 7-in. pavement at various times are shown in Figure 9. Similar data for the 8-in. sections are shown in Figure 10.

It is obvious that the sections change length seasonally, being longer in summer and shorter in winter. Generally speaking, the sections were from 1 to 2 in. shorter in any given winter than they were when the previous summer measurements were taken. One of the concepts of continuous reinforcement is that a large central portion of a long continuously-reinforced pavement remains under restraint and movements occur near the free ends only. When it is considered that the test sections, if entirely free of restraint of any kind, would be expected to undergo seasonal changes in length of 12 to 15 in. due to temperature changes alone, it is apparent that the test sections are under considerable restraint.

Of more significance, however, is the permanent increase that has occurred in the length of every section. Figures 9 and 10 indicate that all the sections are now on the average approximately 4 in. longer than their original lengths. In the summer of 1957 the expansion joints, which were nominally 4 in. wide when constructed, appeared to be tightly closed. In fact, at several of these joints large spalls have occurred due to localized compression where the joint faces were irregular. Some of these areas were so extensive that major repairs were required.

The cause of this growth cannot be stated with any certainty. It is conceivable that the permanent increase in length might be due at least in part to growth of the concrete. Data from the Indiana project indicated that at 10 years the concrete had developed a

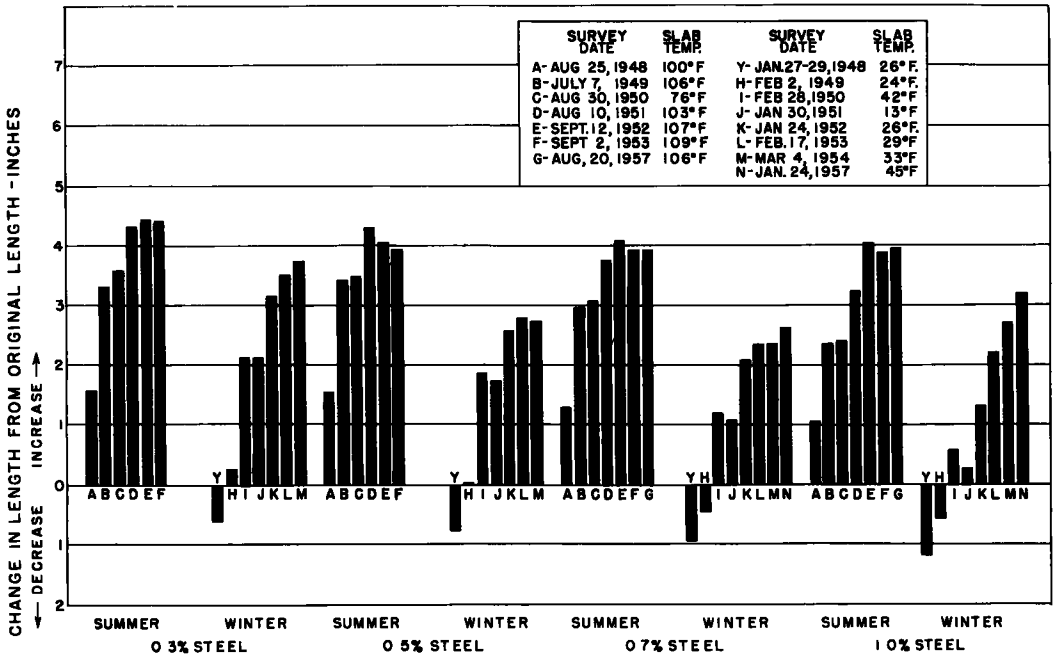

Figure 9. Seasonal changes in length of 7-in. test sections containing various percentages of reinforcing steel.

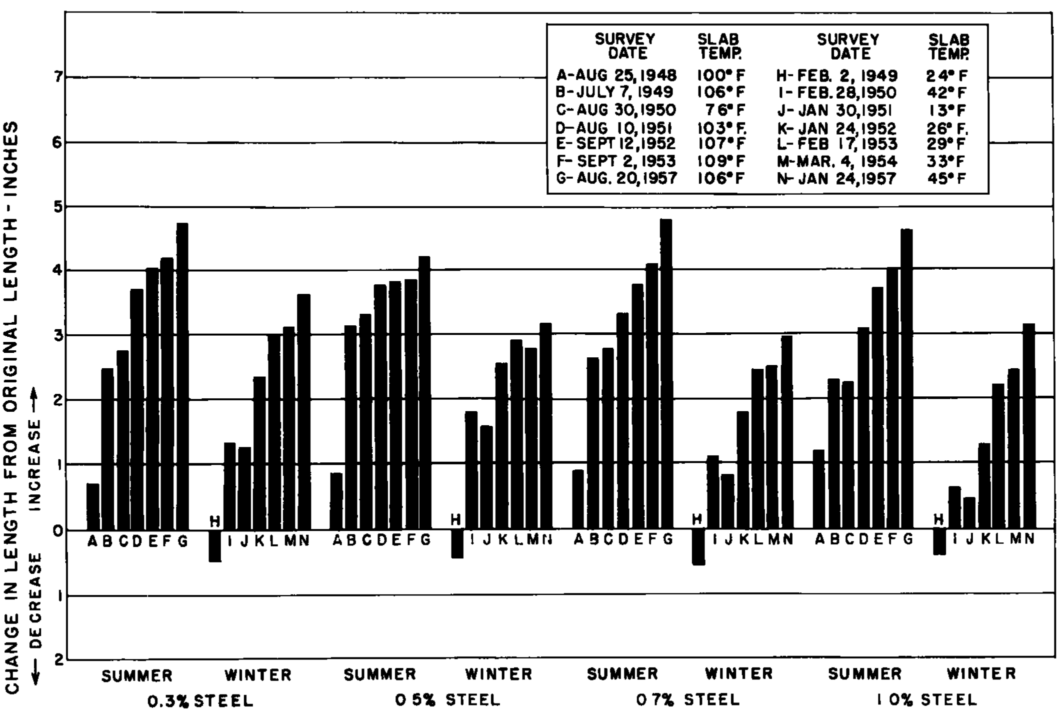

Figure 10. Seasonal changes in length of 8-in. test sections containing various percentages of reinforcing steel.

growth equivalent to $\frac{1}{16}$ in. per 100 ft of pavement (5). A growth of $\frac{1}{4}$ in. per 100 ft was observed at 10 years in the non-reinforced sections in the Bureau of Public Roads investigation at Arlington (5). Thus, the possibility of growth in the concrete cannot be overlooked.

To check this theory a review was made of the Whittemore strain gage measurements which were made periodically between the series of brass plugs set on 10-in. centers along the edge of the pavement in the 30-ft central portion of each test section. Although there are some inconsistencies in the data which need further study, even after eliminating from consideration those readings which were influenced by the presence of transverse cracks, there are strong indications that the concrete has not grown appreciably. The readings taken in August 1957 varied from the initial readings, in most cases, by only 0.001 to 0.002 in., and there was as many minus values as there were plus ones.

It was thought that perhaps the growth may be associated with a permanent opening of the transverse cracks. The cracks, once they form, do not close fully thereafter, even in periods of high temperatures when the pavement is under compression. Whether this permanent opening of cracks is due to infiltration of soil, which may appear logical, is only speculative. Emphasis is given to this speculation by the fact that cores drilled through transverse cracks showed a thin layer of packed soil in the crack. Furthermore, an analysis of crack width data and the permanent increase in length of sections lends some credence to this speculation. When the change in average crack width for a particular section is multiplied by the total number of cracks in the section a value is obtained which, though somewhat greater, is reasonably close to the increase in length of the section. When it is considered that the measured width of cracks, due to the method used, is likely to be somewhat greater than the true width, the agreement is even closer.

Considering the test sections collectively, the increase in length of the 5½ mi of pavement has been between 30 and 40 in. One wonders what would have happened had the entire 5½ mi been built continuous and without any provision for expansion. Would the additional restraint imposed on the pavement have resulted in less over-all growth? Or would excessive compressive stresses have been developed that would have caused blowups? These are questions which cannot be answered on the basis of present knowledge. Neither is it possible to determine how much provision for expansion is required nor the manner in which it should be provided.

RIDING QUALITY OF SURFACE

One of the important claims for continuously-reinforced pavement is that it provides an unusually smooth riding surface. It is believed that there is better opportunity to build the surface to a higher degree of smoothness because it is unnecessary to halt construction and finishing operations for the installation of joints, as is necessary in the case of the conventional pavements in which plate-type joints or groove-type (other than sawed) joints are used. Furthermore, these joints, even when properly installed, may later reduce riding comfort due to curling of slab ends, faulting of slab ends, the joint opening being too wide, and to excess sealing materials which result in a ridge.

The experimental pavement at Vandalia has been recognized for its excellent surface smoothness ever since it was opened to traffic. Some engineers in the Division of Highways have referred to it as the smoothest concrete pavement in the state. Unfortunately, no equipment for measuring surface variations or for determining relative riding quality was available at the time the pavement was opened to traffic; therefore, until recently when the Division of Highways procured a roughometer of the type developed by the U.S. Bureau of Public Roads, any evaluation of riding quality had to be a matter of individual judgment. It was generally agreed that the experimental pavement was exceptionally smooth when built and has retained this characteristic.

Table 5 gives the results of roughometer measurements made August 13, 1957, on the the experimental pavement and also on a 2.32-mi section of standard pavement, which adjoins the experimental pavement of the west and was constructed at the same time and by the contractor who built the experimental pavement. The standard pavement is 10-in. thick, reinforced with 78-lb wire fabric, with plate-type contraction joints at 100-ft intervals.

The readings on the experimental pavement varied from 60 to 90 and averaged 72 in. per mile. Good uniformity existed throughout the length of the experimental pavement, and there were no significant differences between test sections. The average roughness index on the standard pavement was 81 in. per mile. It is doubtful that the difference between the two is of real significance. It is generally considered that an index under 90 in. per mile indicates satisfactory surface smoothness and assures good riding qualities.

TABLE 5

ROUGHNESS INDEXES FOR EXPERIMENTAL PAVEMENT AND ADJOINING STANDARD PAVEMENT
(Readings taken August 13, 1957)

Test Section	Sub-Section	Length (ft)	Roughness Index (in./mi) Westbound Lane OWP	Westbound Lane IWP	Eastbound Lane OWP	Eastbound Lane IWP
(a) Continuously-Reinforced Pavement, Section 0-2						
1	A	1,752	69	72	78	75
	B	1,752	75	66	78	72
2	A	1,752	78	63	72	72
	B	1,752	72	63	69	72
3	A	1,752	66	66	69	75
	B	1,752	63	63	69	72
4	A	1,752	78	66	75	69
	B	1,752	78	69	66	66
5	A	1,752	81	66	72	66
	B	1,752	90	72	69	75
6	A	1,754	87	72	72	69
	B	1,754	84	90	78	81
7	A	2,116	75	75	60	65
	B	2,116	82	77	65	67
8	A	2,116	72	80	62	65
	B	2,116	72	75	65	65
Avg.			77	71	70	70
(b) Standard Reinforced Pavement, Section P-2						
Avg.			80	78	86	80

PAVEMENT SLAB MAINTENANCE

Maintenance of the surface of the experimental pavement has been confined almost exclusively to the repair of distressed areas that developed adjacent to 4-in. expansion joints and at construction joints. These areas of failure are further restricted to the test sections containing 0.3 to 0.5 percent reinforcement. Because, in a regular continuously-reinforced pavement, expansion joints would be installed only infrequently and would be specially designed for the service, no consideration should be given to the maintenance costs at the 4-in. expansion joints. Furthermore, experience has shown that the weakness inherent in the construction joints can be corrected easily, so maintenance at those points would not be a factor in a properly designed continuously-reinforced pavement. Based on the experience in Illinois, therefore, and eliminating the joints from consideration, it can be stated conclusively that the continuously-reinforced pavement has required practically no surface maintenance in the 10-year period since its construction. Surface maintenance costs in the same period for approximately 3.4 mi of adjoining standard pavement west of the experimental pavement amounted to approximately $85 per mile per year. It would appear, therefore, that surface maintenance costs on a properly designed continuously-reinforced pavement could be expected to be low and to compare favorably with those for a pavement of conventional design.

ECONOMIC CONSIDERATIONS

There are three major factors involved in the economic evaluation of continuously-reinforced pavement; namely, first cost, maintenance cost, and service life. Although the experience in Illinois does not as yet give definite data on these factors, it nevertheless furnishes information which permits some general observations.

No accurate information is available to the Division of Highways as to the contractor's true costs for constructing the experimental pavement. The only comparison available is based on his bid prices, and an analysis of these shows that he bid approximately the same unit price for the 7-in. pavement with 0.7 percent steel, the 8-in. pavement with 0.5 percent steel, and the standard 10-in. wire fabric reinforced pavement with 6-in. granular subbase. It is reasonable to assume that the contractor made generous allowances in his unit prices for the continuously-reinforced pavement to cover uncertainties due to the experimental nature of the project. The special requirements for research, the close supervision to be expected from engineers, and the fact that neither his organization nor the Division of Highways had had previous experience with this type of construction. Statements by key personnel of the contractor, which indicated that they were very happy with the manner in which the work proceeded and that their construction costs were well below anticipated costs, tended to confirm this.

It seems reasonable to assume, therefore, that when costs are not influenced by the complications of a research project and when contractors become familiar with the type of construction, the cost of continuously-reinforced pavement of the design recommended hereinafter should compare favorably with the cost of standard 10-in. wire fabric reinforced pavement. It would seem that more conclusive information as to relative costs will be obtained from the projects recently constructed in which a more conventional and perhaps a lower cost method of installing the continuous reinforcement was used.

Practically no surface maintenance has been required on the Illinois pavement outside the areas immediately adjacent to the expansion and construction joints, which are not representative of a regular continuously-reinforced pavement. Furthermore, there is nothing to indicate that any unusual maintenance will be required in the foreseeable future. It appears, therefore, that the cost of maintaining a properly designed continuously-reinforced pavement quite likely will be less than that for a pavement of conventional design.

Both the experimental pavement in Illinois and the adjoining standard pavement are in excellent condition after 10 years of service. There is no evidence now that one will outlast the other. It appears, however, that when a properly designed continuously-reinforced pavement reaches the age when it is no longer adequate as a surface course, it will provide a better base for bituminous resurfacing than a pavement of conventional design.

Summarizing this discussion, it would appear that on the basis of construction cost, maintenance cost, and serviceable life, continuous reinforcement is economically sound.

RECOMMENDED DESIGN FOR FUTURE PAVEMENTS

The data from the several experimental pavements which have been under observation for varying periods of time do not permit formulation of a rational theory of design. The behavior of the various test sections in the Illinois pavement, however, provides an empirical approach to the selection of proper values for certain variables and to the solution of other design features.

Considering first of all pavement thickness, there has been no noticeable difference in performance between the comparable sections of 7- and 8-in. pavements. Therefore, it might be concluded that a 7-in. thickness is adequate. But when one considers the sharp increases which have occurred in recent years in the volume of heavy vehicles on the highways and that this increase is likely to continue, conservatism appears to be the wise course, and 8-in. thickness is recommended.

A more difficult problem is the selection of the minimum percentage of longitudinal steel. The experimental pavement shows definitely that pavement behavior and present condition vary with the amount of longitudinal steel; the 1.0 percent sections being better than the 0.7 percent sections, the 0.7 percent sections being superior to the 0.5 percent sections, and the 0.5 percent sections out-performing the 0.3 percent sections. These four percentages were selected because it was believed they would provide sufficient range in performance that the optimum practical amount could be determined.

It was thought that 0.3 percent would prove wholly inadequate and that those sections would fail extensively and early. It was further believed that 1.0 percent was ultraconservative. In view of the poor expectations for the 0.3 percent sections, their behavior has been amazing. Generally speaking, and with the exception of apparent broken steel and faulting at some wide cracks and failures at expansion joints and construction joints, which, as has been previously stated, should be disregarded in this evaluation, the 0.3 percent sections are in good condition and should continue to give satisfactory performance for a considerable number of years. This is not to say that 0.3 percent is adequate, but it does suggest that the steel requirement may be less than was first supposed.

Stress measurements during the first winter (2) indicated that the steel in the 7-in. section with 0.7 percent steel was at times under a stress of approximately 62,000 psi, which was close to the yield point of the rail-steel bars used. This stress is in excess of minimums specified by ASTM for rail-steel bars, new billet hard grade bars, and wire fabric reinforcement, the logical materials for this purpose. At first glance, this would suggest that perhaps an amount somewhat greater than 0.7 percent, perhaps 0.8 percent, should be recommended.

One cannot disregard the fact that the sections with 0.5 percent have given good performance and in general are in excellent condition. Therefore, 0.5 percent is suggested as an absolute minimum, but 0.6 percent is recommended as a more conservative value. Also, because there is some indication that the steel in pavements built at summer temperatures is subject to higher stresses than when construction temperatures are lower, perhaps 0.7 percent steel should be used in pavements constructed in extremely hot weather.

The unsightly longitudinal cracks which developed in the Illinois pavement leave no doubt about the need for a controlled center joint.

Transverse reinforcement is a matter of what is required to support the longitudinal steel from the subgrade, if the method of installation employed in the Illinois pavement is used. Based on the experience in Illinois, No. 3 bars at 18-in. centers are adequate. If welded fabric or bar mats are used, the size of the transverse members would be dependent on manufacturing considerations.

The requirements for expansion joints are difficult to determine. Certainly some provision for expansion would be required adjacent to structures. But whether expansion joints would be required at regular intervals in long stretches of continuously-reinforced pavement is presently indeterminate. Further research is needed in which the movements in a long (4- or 5-mi) consinuously-reinforced pavement without expansion joints could be studied.

Although pumping has not been a serious problem except at expansion joints and construction joints, some has occurred, and it is recommended that a granular subbase be used. It is believed that a 3-in. layer would be sufficient to control pumping, but greater depths might be required in some cases to counteract the effect of frost action in frost-susceptible soils.

CONCLUSIONS

The information obtained from observations and study of the Illinois experimental pavement support the following conclusions:

1. Large numbers of transverse cracks develop in continuously-reinforced pavements, their frequency being proportional to the amount of longitudinal steel.
2. Cracks are relatively wide-spaced near the ends of the long continuously-reinforced slab; their frequency increases at a fairly uniform rate to a maximum at a distance of 200 to 500 ft from the ends, from whence the frequency remains fairly constant over the long central portion
3. A large number of transverse cracks occurs very early, and thereafter the development of cracks is a function of age, the rate of development decreasing with time. It appears that transverse cracking in the Illinois pavement has reached an equilibrium at the end of 10 years and that cracks can be expected to occur at a very slow rate in the future.
4. After 10 years the average crack intervals for the sections with 0.3, 0.5, 0.7, and 1.0 percent longitudinal steel, respectively, are approximately 12, 8, 6, and 5 ft, considering the 7- and 8-in. pavements collectively. There is no significant difference between the two thicknesses of pavement.
5. The width of cracks is an important factor in the performance of continuously-reinforced pavement. Cracks must be narrow to minimize infiltration of soil and water and to maintain effective aggregate interlock between crack interfaces.
6. Crack width is a function of the amount of longitudinal steel, the greater the percentage of steel the smaller the crack. It also increases with age.
7. At the age of 10 years the crack width measured by focusing a measuring microscope on the point below the pavement surface, so as to reduce the effect of raveling, averages 0.034, 0.023, 0.016, and 0.009 in. for the 0.3, 0.5, 0.7, and 1.0 percent sections, respectively, considering both 7- and 8-in. pavements collectively. There are no significant differences between the two thicknesses.
8. Cores drilled through representative cracks in each section indicate that the real width of the cracks is less than that measured by the microscope. On the basis of limited data, perhaps a reasonable approximation would be that the real width is about one-half of the measured width.

9. Raveling and spalling along the edges of the cracks is of no serious consequence, except at a few cracks in the 0.3 percent sections which are excessively wide and where apparently the steel is broken. The cracks are not visible from a vehicle traveling at 30 mph.

10. Meandering longitudinal cracks will occur in a continuously-reinforced pavement built without a controlled center joint. Although they may be of little significance structurally, they are likely to be considerably wider than the transverse cracks, will ravel more, and become unsightly.

11. Continuous reinforcement will reduce the tendency for pumping, judging from behavior of the Illinois pavement. However, in view of the fact that some pavement pumping occurred, it appears that it would be wise to use a thin granular subbase under this type of pavement as an added safety factor.

12. Construction joints are points of potential weakness and need provision for adequate load transfer. This can be accomplished by using at least No. 5 bars at a maximum of 6-in. centers, or by providing auxiliary dowel bars when a lesser amount of reinforcing steel is used.

13. Sections of continuously-reinforced pavement of the lengths used in this investigation, and separated by wide expansion joints, will show a permanent increase in length with age. The cause of this growth is not readily apparent. Whether a similar increase would occur in sections several times that length is not known.

14. Continuously-reinforced pavement can be built to a high standard of surface smoothness, and can be expected to possess good riding qualities throughout a long life.

15. A properly designed and constructed continuously-reinforced pavement will require little maintenance and will compare favorably in this respect with pavement of conventional design.

16. It appears that on the basis of first cost, maintenance cost, service life, and salvage value, continuously-reinforced pavements are economically sound.

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