THREE-YEAR PERFORMANCE REPORT ON EXPERIMENTAL CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN ILLINOIS

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SYNOPSIS

This paper covers the performance of the experimental continuously reinforced concrete pavement in Illinois during the period since its construction in 1947–48. Details of construction were previously reported at the 27th Annual Meeting of the Highway Research Board.

The performance of all the test sections has been highly satisfactory and compares favorably with that of the adjacent 4½ mi. of standard pavement built at the same time by the same contractor. Two localized structural failures developed in 1948 in the eastbound lane of Section 4 (8-in. pavement with 0.3 percent steel) and progressed to the point where it was necessary, early in 1950, to replace the damaged concrete with concrete patches, approximately 8 ft. and 4 ft. in length, respectively. The exact cause of these failures has not been definitely determined.

Some pumping has occurred, principally at expansion joints separating the test sections, at construction joints and at a relatively few transverse cracks, particularly in Section 4 (8-in. pavement with 0.3 percent steel). Pumping, first observed in the fall of 1948, was a matter of considerable concern because it was feared that it might become progressive. This, however, has not been the case; in fact, little pumping has been observed during the past year except at expansion joints.

SR-4 strain gages, which were installed on selected reinforcing bars in two sections, have become unreliable, but not before they yielded valuable data on the stresses in the reinforcing bars. Maximum stresses observed were 62,400 and 42,000 p.s.i., respectively, in the 7 and 8-in. sections with 0.7 percent steel.

The average interval between transverse cracks has become progressively shorter with age and bears an inverse relationship to the amount of longitudinal steel. In other words, the frequency of transverse cracks becomes greater as the percentage of steel increases. This is true of both 7- and 8-in. sections. Average crack interval ranges from 6 to 16 ft.

Repeated measurements of 60 representative transverse cracks in each test section show a tendency for the cracks to become wider with age. Generally speaking the average crack width is greater the lower the percentage of longitudinal steel. The behavior of both 7- and 8-in. pavements is similar. Average crack width ranges from 0.007 to 0.021 in.

All the test sections have become progressively longer with age and, in the case of the 7-in. sections, the growth bears an inverse relationship to the amount of longitudinal steel. The inverse proportionality is not quite so apparent for the 8-in. sections, possibly because a period of about 7 months elapsed between the construction of the first and last sections and, therefore, the average slab temperature at which the initial measurements for the several sections were made varied quite widely. The permanent growth ranges from about 2 in. to 3.6 in.

A considerable amount of longitudinal cracking has developed in the 7-in. sections, all of which were built during the fall of 1947. Except for the 8-in. section containing 1.0 percent longitudinal steel, which was also constructed in the fall of 1947, very little longitudinal cracking has developed in the 8-in. sections. The other three sections of 8-in. pavement were constructed in April and May of 1948.

The longitudinal cracks naturally do not coincide with the centerline of the pavement, but follow an irregular path, meandering as much as 3 feet from the centerline. At several locations there are two approximately parallel longitudinal

cracks, and at one location there are three. In those sections having a considerable amount of longitudinal cracking, the amount appears to be proportional to the percent of longitudinal steel.

Some spalling has occurred at both longitudinal and transverse cracks but not to a serious extent. It appears that spalling may be a function of the crack width, with more spalling at the wider cracks.

This is a report on the performance of the experimental continuously reinforced concrete pavement in Illinois during its first three years of life. A previous report[1] gives the details of design and construction of the pavement so it will be necessary to include here only sufficient description to familiarize the reader with the essential features of the pavement.

The experimental pavement was constructed in 1947 and 1948 by the Illinois Division of Highways, in cooperation with the U. S. Bureau of Public Roads, just west of Vandalia, Illinois, on U. S. Route 40. A recent survey showed that the average 24-hr. traffic count was 2250 vehicles, consisting of 1700 passenger cars and 550 commercial vehicles, of which 300 were tractor semitrailers.

The pavement is 22 ft. wide, about 5½ miles long and is divided into eight test sections, ranging in length from 3500 to 4200 ft., separated from each other by poured expansion joints, whose original width was 4 in. Four of the sections have a uniform thickness of 7 in. and four a uniform thickness of 8 in.

Four percentages of longitudinal steel, namely, 0.3, 0.5, 0.7 and 1.0 percent, based on the gross cross sectional area of the concrete, were used with each thickness of pavement. The longitudinal steel is continuous from end to end of each section, the reinforcing bars being lapped 30 diameters, and was placed about 3-in. below the top surface of the pavement. The transverse reinforcement is continuous across the width of the pavement and consists of ⅜-in. bars spaced 12 in. apart in one-half of each section and 18 in. apart in the other half. All reinforcing bars met the requirements of ASTM Designation: A305-47T for the deformations of deformed steel bars for concrete reinforcement. Rail steel bars were used for longitudinal reinforcement and intermediate grade, new billet steel bars for transverse reinforcement.

[1] H. W. Russell and J. D. Lindsay "An Experimental Continuously Reinforced Concrete Pavement in Illinois" *Proceedings*, Highway Research Board, Vol. 27 (1947).

The pavement was constructed directly over the natural soil as graded, without granular replacement or special subgrade treatment of any kind. About 90 percent of the pavement is on soils which are susceptible to pumping.

Performance of the Pavement—The pavement has been under close observation since its construction and a large amount of data have been collected on steel stresses, frequency of transverse cracks, width of transverse cracks, dimensional changes of the test sections and the physical condition of the pavement. Since the average age of the test sections is less than three years, five sections having gone through three winters, and the other three only two winters, no definite predictions can be made as to their ultimate performance or of the behavior of continuously reinforced pavement in general. The data, however, show some highly significant tendencies and trends, which are of interest and worthy of consideration.

Stresses in Longitudinal Steel—Steel stresses are perhaps of greatest immediate significance because they give the earliest indication of the probable ability of the pavement to perform satisfactorily. Successful performance cannot be expected unless the stress in the reinforcing bars remains below the yield point of the steel. If the stress exceeds the yield point, the bars will stretch, the cracks will become progressively wider and eventually the steel may break. Since it is entirely possible that near maximum steel stresses may occur during the first winter, especially if the pavement is built during periods of high temperature, it may be readily seen that stress measurements can be of great significance.

Stress was measured by means of electrical resistance wire strain gages, commonly called SR-4 gages, mounted on selected longitudinal bars in both the 7 and 8-in. sections which contain 0.7 percent of steel. Gages were arranged to measure the average stress at each of eight transverse cross sections, 2½ ft. apart, near the middle of the test section. A depressed

contraction joint was installed along one line of gages so that the stress could be measured where the steel received no assistance from the concrete, as is the case at transverse cracks.

Many stress readings were taken on both test sections during the first month after they were built; later, readings were taken periodically, but less frequently. Representative stress data for the two test sections are shown in Figures 1 and 2.

assistance in resisting the axial forces acting on the pavement. The curve at the bottom of the chart gives the average temperature of the pavement for corresponding stress readings.

As noted on the chart, concrete was poured at the location of the gages at 1:00 P.M. on October 6, 1947. The zero or "no stress" condition was determined from readings taken immediately after the concrete was poured and subsequent readings were referred to this as a base. It will be noted there is little differ-

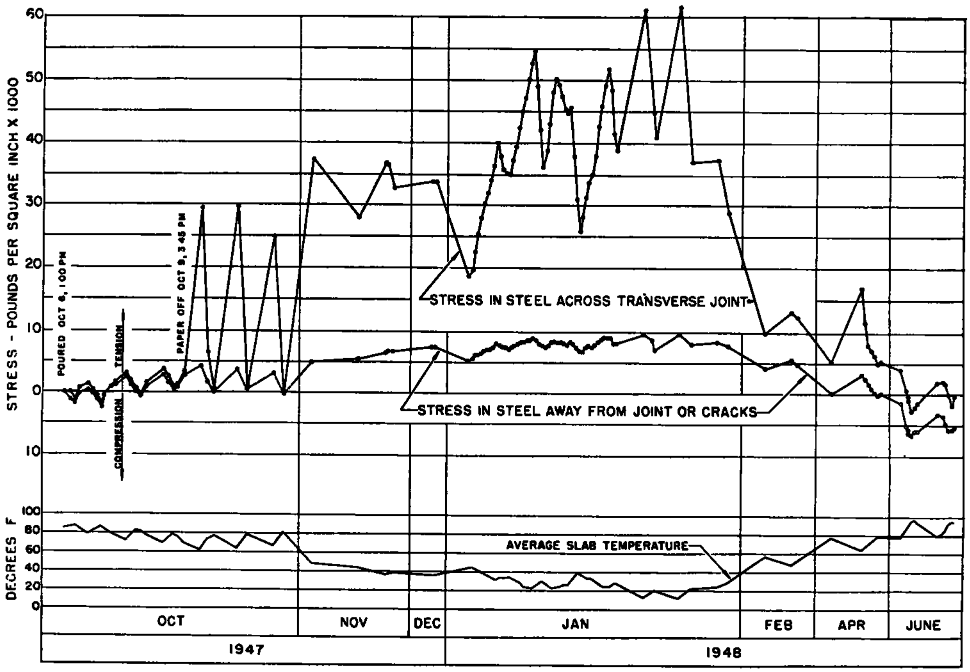

Figure 1. Stresses Measured in Longitudinal Steel in Section 3 (7-in. Pavement, 0.7 Percent Steel).

Figure 1 shows typical stress readings for Section 3 (7-in. pavement, 0.7 percent steel). The top curve, with closed circles marking the measured points, indicates the unit stress in the steel along the depressed transverse joint, where the bars receive no assistance from the concrete and must resist all the tensile force. The value at each point is the average of the readings from all reliable gages along the depressed joint.

The curve with open circles gives the unit stress in the longitudinal steel at points away from the depressed joint or from transverse cracks, where the concrete affords maximum

ence between the two stress curves for the first three days, when the pavement was covered with curing paper. The stress in the steel during this time ranged from 4500 lb. per sq. in. tension in the mornings, when the temperature was minimum, to 3000 lb. per sq. in. compression in the afternoons, when the temperature was maximum.

When the curing paper was removed on the afternoon of the third day, the only evidence of the depressed joint was a fine vertical crack down each edge of the pavement. Early next morning, however, a crack had opened to a width of 0.010 in. for the full length of the

joint, and the average stress in the steel at this location was approximately 30,000 lb. per sq. in. tension. The maximum stress away from the joint remained approximately 4500 lb. per sq. in. tension. By midafternoon, the stress at the joint had fallen to about zero, and the crack over the joint was again tightly closed.

Thereafter, the stress at the joint fluctuated widely in response to daily and seasonal temperature changes, being high when the temperature was low and low when the temperature was high. The stress away from the joint also responded to temperature changes, but to a much less degree.

The observed maximum average stress at the joint was 62,400 lb. per sq. in. on January 21, 1948, when the average slab temperature was 13 deg. F., or about 70 deg. lower than that for which the zero or "no stress" condition was determined. As an example of the reliability of the strain gages at that time, 9 of the 12 gages originally installed were used in determining the average stress and none of their readings varied more than 4 percent from the average. Since the yield point of the rail steel bars in this section is about 70,000 lb. per sq. in. the maximum observed stress is within the critical range, and this may indicate that less than 0.7 percent of steel is inadequate.

From this point a seasonal rise in temperature began, accompanied by a decrease in the general stress level. Gages began to give erratic results during the spring thaw and by June 22 only 4 of the original 12 gages were reliable. By that time the steel at the joint was generally in compression.

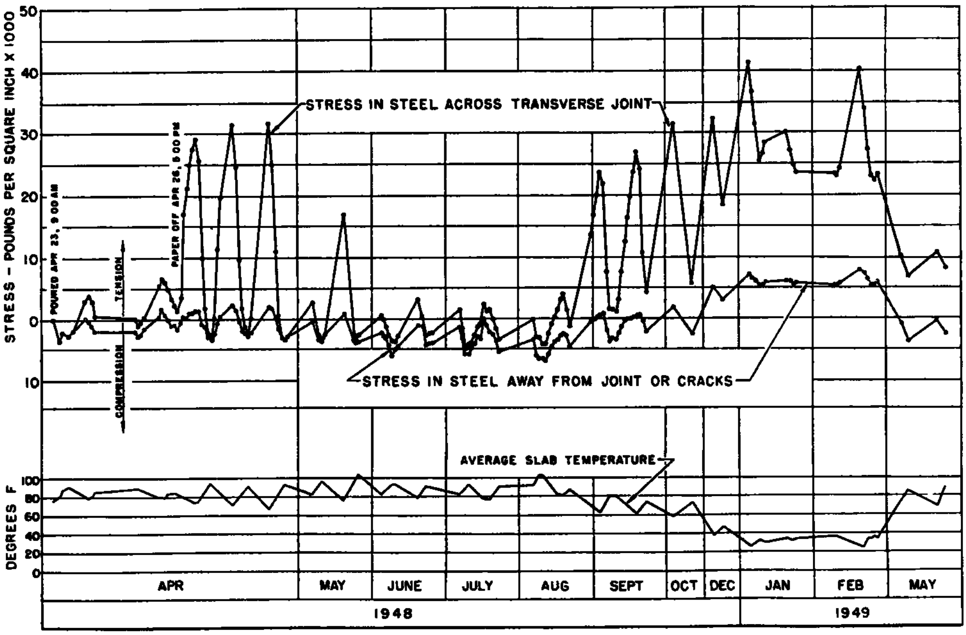

Figure 2. Stresses Measured in Longitudinal Steel in Section 6 (8-in. Pavement, 0.7 Percent Steel).

It is interesting to note that the stress away from the joint or transverse cracks was never higher than 10,000 lb. per sq. in. This indicates that as long as the concrete is structurally continuous it will carry a large part of the total stress. It is only when the concrete cracks that the steel is required to carry high stress and even then the concrete absorbs this stress within a short distance from the crack, if the bond is good. As an example of the efficiency of the bond, one line of gages showed no measurable effect from a transverse crack only 10 in. away.

Figure 2 shows stress data for Section 6, an 8-in. pavement with 0.7 percent steel laid in

April 1948. The curves show the same trends as those in Figure 1 and no detailed discussion is necessary. The most significant feature is that the observed maximum tensile stress in Section 6, measured in January 1949, was about 42,000 lb. per sq. in. as compared to 62,400 lb. per sq. in. for Section 3, which was laid in October 1947. The most probable reason for this difference is that Section 3 was subjected to a temperature range about 25 deg. F. greater than Section 6, due to the difference in temperature at the time the sections were constructed, and the difference in temperature when the maximum stresses in the two sections were measured.

It is apparent from this discussion 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 across joints or cracks responds rapidly to daily changes in temperature and may show a large variation during a 24-hour period. When the general temperature level is higher than that at which the pavement was constructed, the steel stress is likely to be low and probably on the compression side. Maximum tensile stresses occur in the winter and are proportional to the temperature drop and inversely proportional to the amount of steel. It is at this time that the stress across joints and cracks will be critical, if the steel is not adequate. It appears that critical stress in the steel will not occur where the concrete is unbroken, a condition which assists the steel in carrying stress.

Unfortunately, the SR-4 strain gages in both sections became erratic in a relatively short time so it was not possible to obtain stress measurements for more than one cycle of seasonal contraction. However, during the time the gages remained reliable very valuable information was obtained which more than justified the cost of the gage installations. It has not been possible to determine the cause of the gage failures, but it is believed most likely that moisture entered the gages in spite of efforts to waterproof them. The experiment points to broad possibilities for the application of the SR-4 gages in this type of work if they can be successfully moistureproofed, and suggests that further study be given to this problem.

Frequency of Transverse Cracks—The theory of continuous reinforcement assumes that the occurrence of frequent transverse cracks will afford stress relief and absorb a great part of the changes in length which occur daily and seasonally in long slabs. According to theory, if the pavement contains adequate steel, frequent cracks are induced by an alternate interchange of stress between the steel and the concrete. When the concrete cracks, the steel assumes the full stress at that location, but, by virtue of bond, the stress is quickly transferred back to the concrete, and another crack eventually develops. The behavior of all the test sections has been according to theory in this respect, large numbers of transverse cracks having developed early in each section and new cracks are developing continually.

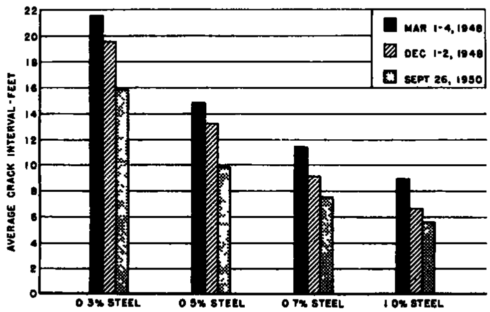

Figure 3. Effect of Amount of Longitudinal Reinforcing Steel on Average Transverse Crack Interval—7-in. Sections

Figure 3 shows the results of three surveys made to determine the frequency of transverse cracks in the four sections of 7-in. pavement. The first survey was made in March 1948, after the sections had gone through one winter; the second was made in December 1948, during the following winter; and the third in September 1950. In this chart, for the purpose of comparison, frequency is plotted in terms of the average interval between cracks instead of the number of cracks for a given length.

Similar data are shown in Figure 4 for the 8-in. sections. Only Section 7 (8-in., 1.0 percent steel) was included in the March 1948 survey; the other three sections were not constructed until later.

The most significant thing about these data is the very obvious relationship between crack

interval and the amount of longitudinal reinforcing steel. It is clearly evident that the average crack interval increased as the amount of steel decreased, or the frequency of cracks increased with the amount of steel. The charts also show there was a decrease in the crack interval, or an increase in the number of cracks, in each section with each successive survey.

It will be seen from Figure 3 that in March 1948, the average crack intervals for the 7-in. sections with 0.3, 0.5, 0.7, and 1.0 percent steel, respectively, were 21.6, 14.9, 11.5, and 9.0 ft.; in December 1948 these had decreased to 19.6, 13.3, 9.2 and 6.7 ft.; and in September 1950 the values were 15.9, 9.9, 7.6 and 5.6 ft. Figure 4 shows that in March 1948 the average crack interval in the 8-in. section with 1.0

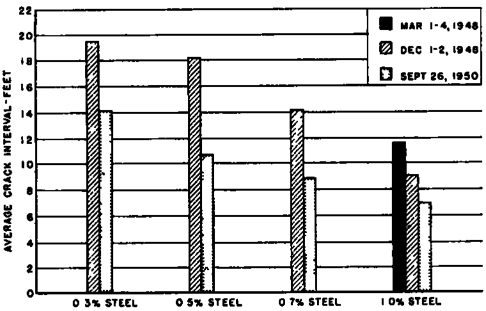

Figure 4. Effect of Amount of Longitudinal Reinforcing Steel on Average Transverse Crack Interval—8-in. Sections

percent steel was 11.6 ft. In December 1948 the average intervals for the 8-in. sections with 0.3, 0.5, 0.7 and 1.0 percent steel, respectively, were 19.5, 18.2, 14.2 and 9.1 ft.; and in September 1950, 14.2, 10.7, 8.9 and 7.0 ft. While, in general, the intervals are somewhat less for the 7-in. sections than the 8-in., it is believed that difference in age and the fact that the 7-in. sections went through one more winter than three of the 8-in. sections, rather than thickness of pavement, may be responsible for such differences as exist.

Data from the Stilesville, Indiana, project[2] show that in the heavily reinforced sections (1.82 percent steel) the transverse crack frequency after 10 yrs. was lowest near the ends of the sections and increased uniformly to a maximum at the center of the sections. This was true for sections ranging from 600 to 1310 ft. in length.

However, as will be seen from Figure 5, crack distribution follows a somewhat different pattern in the Illinois experiment. Beginning at the ends, the frequency for successive 100-ft. increments increases at a fairly uniform rate to a maximum at a distance ranging from 300 to 500 ft. from the ends of the sections, from whence it decreases uniformly for several hundred ft. and finally remains fairly uniform over the central 1600 to 2000 ft. of the section, although there appear to be indications of secondary peaks in this central portion as the amount of reinforcement increases, particularly for the 0.7 and 1.0 percent sections.

It is realized that the data from the two projects are not comparable because of the difference in age and perhaps because of the much heavier reinforcement in the Indiana sections. The cracking pattern may change in the Illinois sections within another 7 yrs. However, the present pattern points to the possibility that perhaps the Indiana sections were not long enough to be representative of the action in slabs of much greater length. The fact that the maximum frequency occurs at from 300 to 500 ft. from the ends of the Illinois sections would indicate that maximum frequency would develop at the center of slabs approximately 1000 ft. long, in which case even a 1310 ft. slab, the longest in the Indiana project, might be expected to follow that pattern.

Width of Transverse Cracks—Of equal importance, and closely related to steel stress, is the width of transverse cracks which develop in a continuously reinforced pavement. For effective performance the cracks must be narrow, preferably so small that they will not carry appreciable amounts of water to the subgrade. Crack width is a function of steel stress and effectiveness of the bond between concrete and steel in the immediate vicinity of the crack.

Figure 6 shows a typical crack in each of the four 7-in. test sections, containing 0.3, 0.5, 0.7, and 1.0 percent of longitudinal steel, respectively. These photographs indicate that crack width varies inversely with the amount of steel. The crack in the 0.3 percent section

[2] Harry D. Cashell and Sanford W. Benham "Experiments with Continuous Reinforcement in Concrete Pavements," *Proceedings*, Highway Research Board, Vol. 29 (1949).

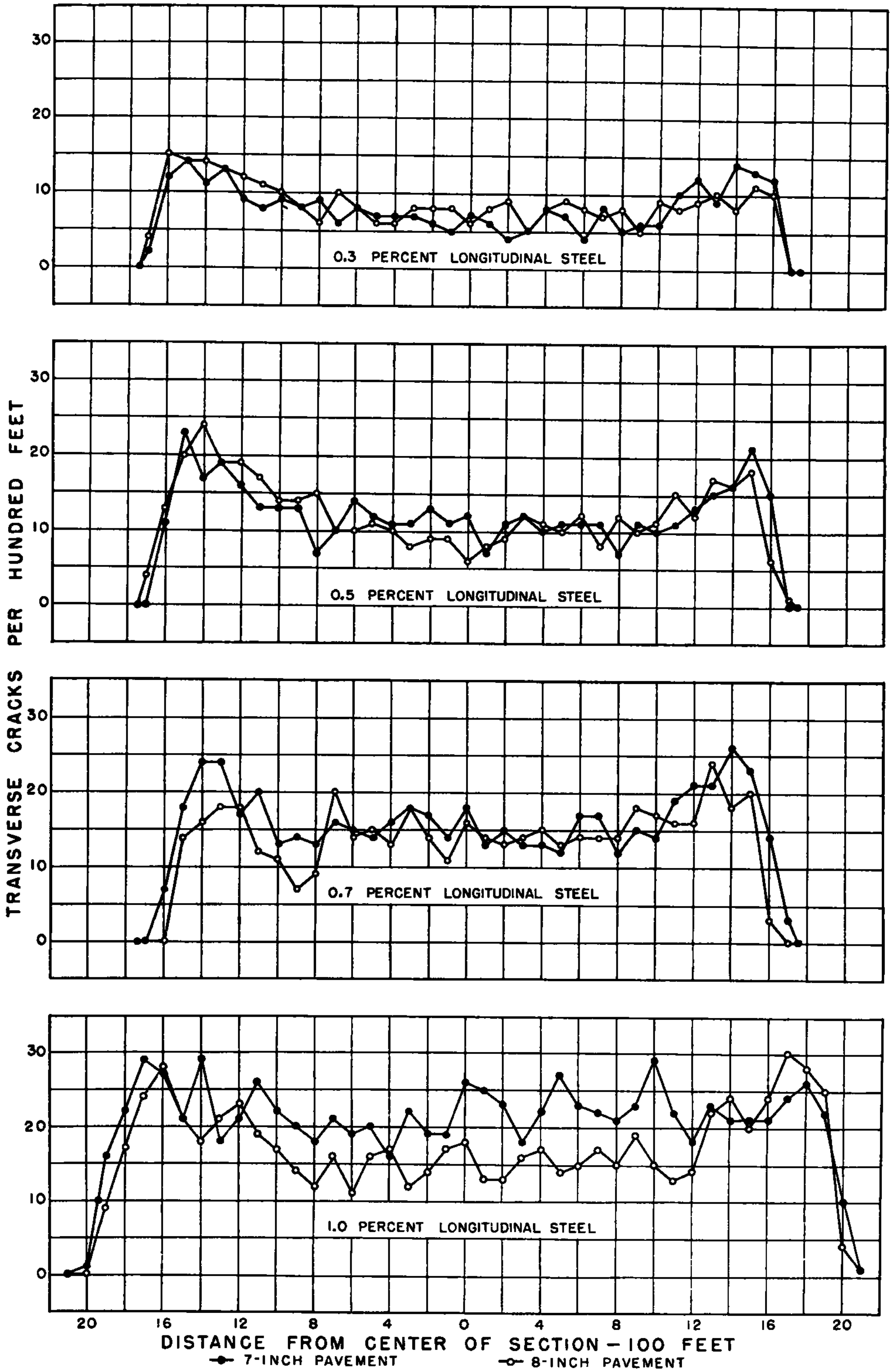

Figure 5. Frequency Distribution of Transverse Cracks for All Sections—Sept. 1950

is definitely much wider than the others, and it is obvious the crack in the 1.0 percent section is the tightest. The photographs show practically no spalling along the cracks and, in this respect, are also typical, except for the section with 0.3 percent steel, where some spalling exists at a considerable number of cracks. These photographs are also representative of the cracks in the respective sections of 8-in. pavement.

A better comparison of the width of cracks is shown in Figures 7 and 8, in which the average surface width of 60 representative cracks, measured with a microscope, is plotted for each of the four sections of 7- and 8-in. pavement, respectively. Five sets of measurements were taken from July 1948 to August 1950.

Both charts show that, in general, there is an inverse relationship between average crack width and the amount of longitudinal steel, the cracks being widest in the 0.3 percent sections and becoming progressively narrower as the percentage of steel increases. Referring to Figure 7, the crack widths on August 30, 1950, were 0.021, 0.015, 0.013 and 0.007 in. in the 0.3, 0.5, 0.7 and 1.0 percent sections of 7-in. pavement, respectively, and from Figure 8, corresponding values for the 8-in. sections were 0.016, 0.012, 0.011 and 0.007 in.

The charts show that crack width varies seasonally, cracks being relatively wider in winter, when the pavement is contracted and the steel presumably is under greater tension, than in the summer, when the pavement is expanded. However, a matter of considerable concern is the fact that the representative cracks are becoming progressively wider with age and the cracks do not close tightly in the summer when the pavement expands, but appear to be remaining further open with each successive summer. Three factors may be contributing to this behavior. Incompressible material entering the cracks may prevent them from closing fully; the steel bars may have a somewhat higher coefficient of expansion than the concrete and the greater expansion of the steel may tend to keep the cracks from closing; and permanent shrinkage of the concrete may be sufficient to produce a permanent opening at cracks. It would appear most likely, however, that the first factor would be largely responsible for the progressive opening of the cracks.

The question is often asked whether the

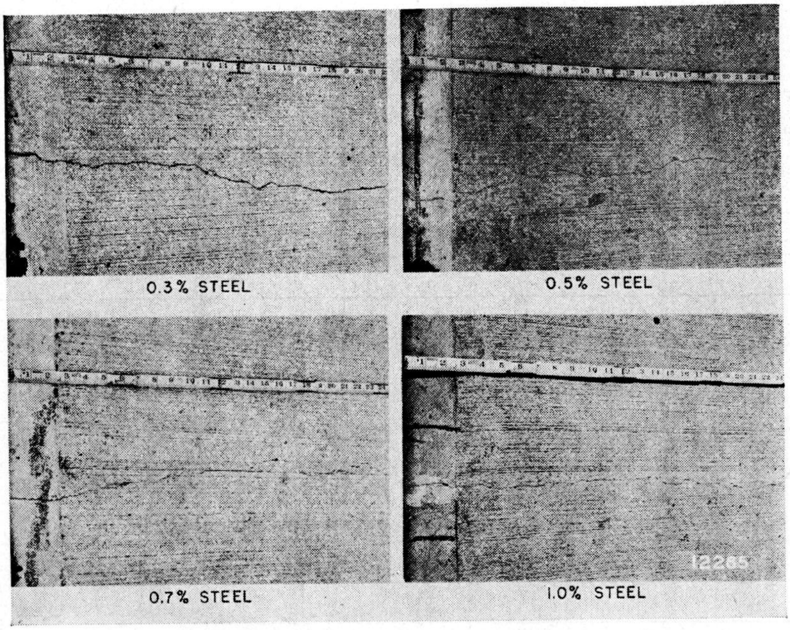

Figure 6. Typical Transverse Cracks in 7-in. Sections

progressive opening of cracks would occur if the experimental pavement were continuous from one end to the other; that is, if the various test sections were not separated from each other by wide expansion joints, which provide relief and permit the sections to move independently. The expansion joints between the sections are becoming progressively narrower, and the action of the cracks, after the joints become fully closed and greater resistance to movement is built up in the slabs, may provide the answer. However, more reliable information in this respect probably would be obtained from a study of much longer continuously reinforced slabs.

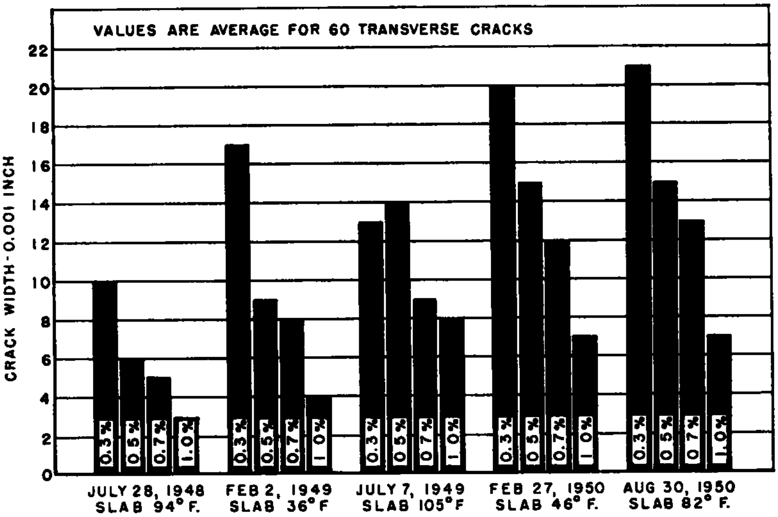

Figure 7. Effect of Amount of Longitudinal Reinforcing Steel on Width of Representative Cracks—7-in. Sections.

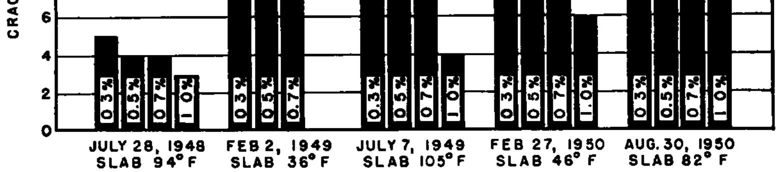

Figure 8. Effect of Amount of Longitudinal Reinforcing Steel on Width of Representative Cracks—8-in. Sections.

It is not definitely known how wide a crack must be before it will permit the passage of serious amounts of water to and from the subgrade. Many factors are involved, and what might be a critical width in one case probably would not be in another. Water has been observed seeping up through some cracks in all sections of our pavement, particularly when the frost comes out of the subgrade in

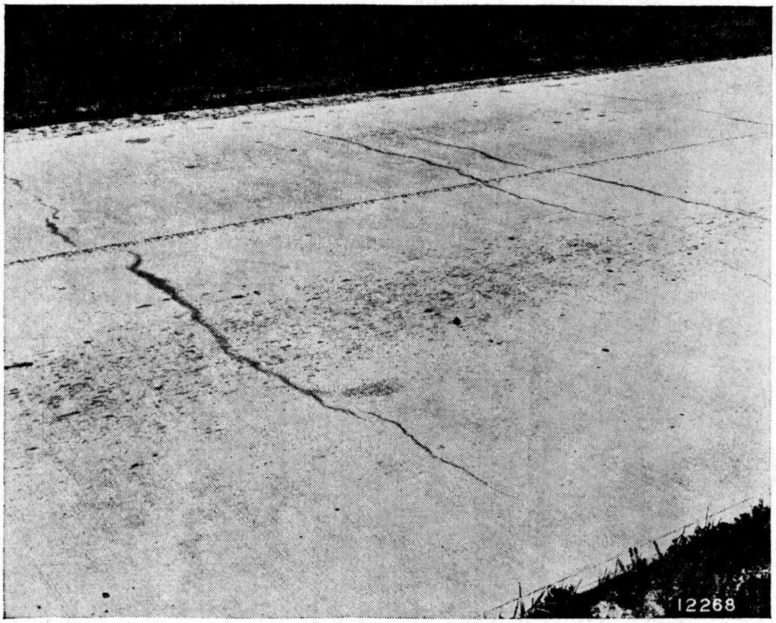

Figure 9. Stained Cracks in 7-in. Pavement Containing 0.7 Percent Steel—Pavement is in a cut.

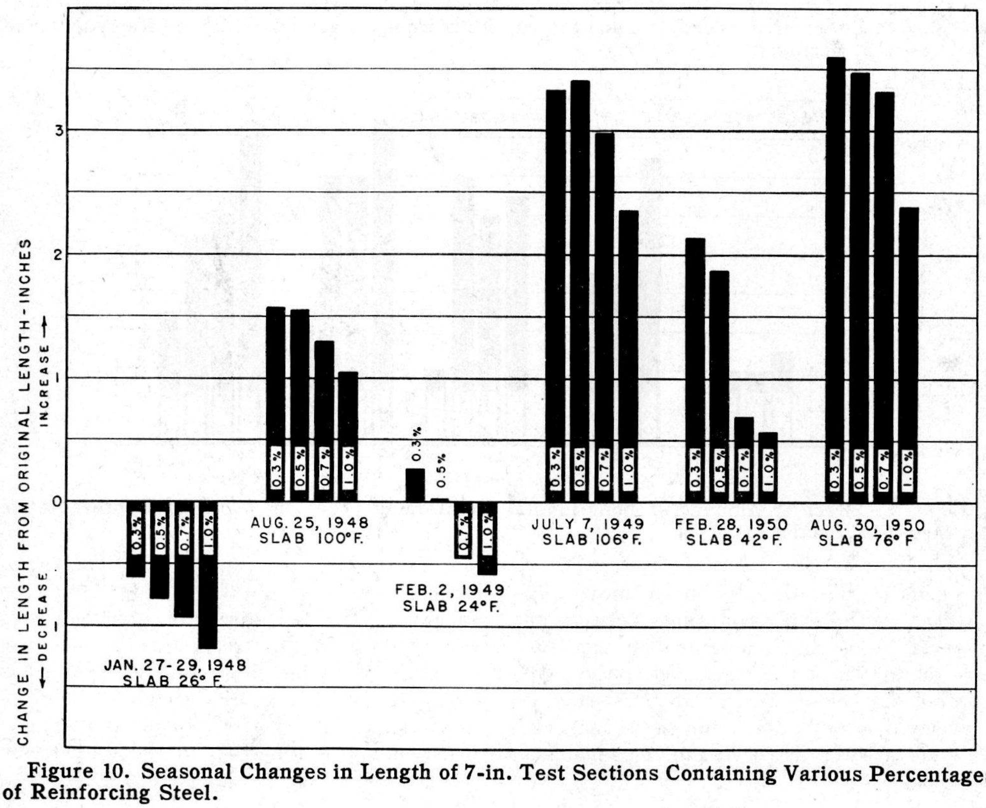

Figure 10. Seasonal Changes in Length of 7-in. Test Sections Containing Various Percentages of Reinforcing Steel.

the spring. Some of these cracks are permanently stained as shown in Figure 9. The pavement at this point is in a cut and undoubtedly the water table at times is high.

Soil stains have been observed in a number of cracks, particularly in the 8-in. section with 0.3 percent steel at a time when the cracks averaged about 0.015 in. in width. This will be discussed in greater detail later; it is mentioned here to show the necessity for narrow, tight cracks. A very few cracks have a reddish stain at the surface, which might suggest that the steel is corroding; however, the stain may come from some coloring in the soil. As yet there is no conclusive evidence as to the optimum width of crack.

Change in Length of Sections—A study of seasonal movements of the test sections is important because the trend of these movements indicates how successfully the pavement is performing. Figure 10 shows the changes in total length of the four 7-in. sections at six different times, three when the temperature was low and three when the temperature was high. Figure 11 shows similar data for the four 8-in. sections at five different times, three in the summer and two in the winter.

Two things are immediately apparent from these charts. First, the length varies inversely with the amount of steel—the more steel the greater the contraction in winter and the less the expansion in summer. Second, all the sections are experiencing a permanent increase in length. Referring to the first, third and fifth groups of bar graphs in Figure 10, and the second and fourth groups in Figure 11, it will be noted that, in general, during cold weather, the decrease in length of each section became greater as the percentage of steel increased; that is, the greater the amount of steel the shorter the section became. The second, fourth and sixth groups in Figure 10, and the first, third and fifth in Figure 11, show that during each summer the sections with the most steel had the least increase in length—the more steel the shorter the section.

That the sections become shorter in winter as the amount of steel increases, can perhaps be explained by the fact that the cracks in

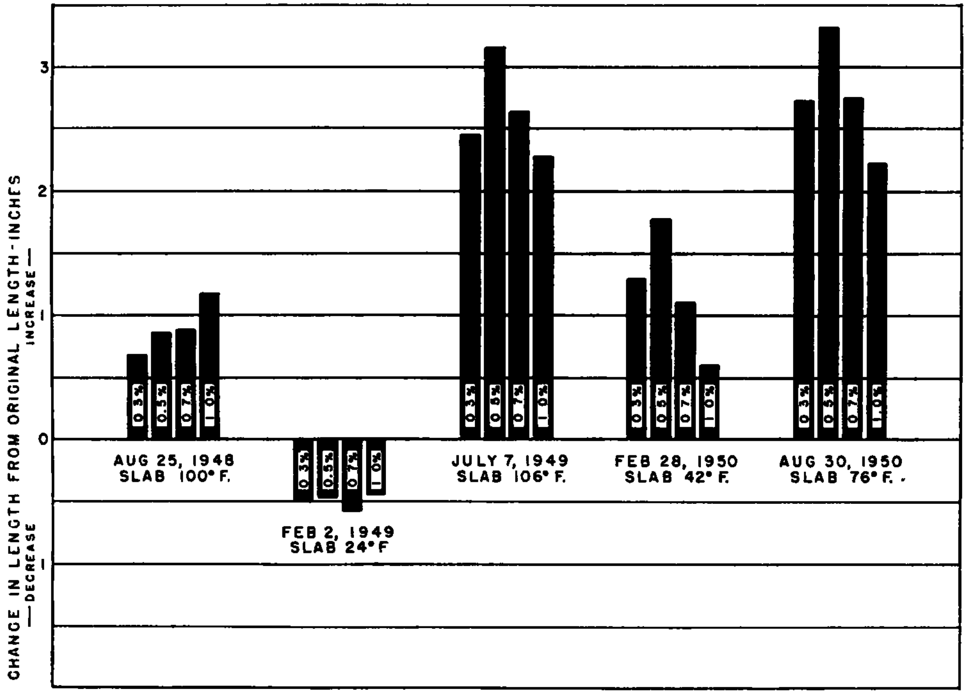

Figure 11. Seasonal Changes in Length of 8-in. Test Sections Containing Various Percentages of Reinforcing Steel.

the lighter reinforced sections, being wider, take up a relatively larger part of the total contraction. The situation in the summer, with the sections being generally longer the less steel they contain, is difficult to explain since it would seem that, if temperature were the only factor, all sections should expand the same amount. However, it may be that infiltration has a relatively greater effect on the sections with lighter steel which have the widest cracks in winter.

All the sections have become progressively longer, both in summer and winter, and the only apparent explanation for this is that incompressible material entering the cracks has rendered them less effective in absorbing expansion and contraction. That the cracks remain partially open in the summer was shown by Figures 7 and 8.

TABLE 1
LONGITUDINAL CRACKING IN TEST SECTIONS

Pavement Thickness	Section No.	Date Constructed (inclusive)	Longitudinal Steel	Length of Cracks Projected on Centerline, Percent of Section Length		
				March 1948	Dec. 1948	Sept. 1950
in.			%	%	%	%
7	1	9-25-47 9-30-47	0.3	0	28	34
	2	9-30-47 10- 3-47	0.5	0	31	49
	3	10- 3-47 10- 7-47	0.7	0	42	58
	8	10-14-47 10-17-47	1.0	46	80	82
8	4	4-30-48 5-20-48	0.3	—	0	1
	5	4-26-48 4-30-48	0.5	—	0	<1
	6	11- 6-47 12- 3-47 4-22-48 4-26-48	0.7	—	0	<1
	7	10-17-47 11- 6-47	1.0	52	74	79

Until recently, measurements indicated a remarkable stability in a large central portion of each section, no appreciable movement having been observed in the central 2700 ft. of those sections which are 3500 ft. long, or in the central 3400 ft. of those sections which are 4200 ft. long, all the movement being produced in the first 400 ft. from the ends of the sections. However, both daily and seasonal movements have recently been observed beyond 400 ft. from the ends of the sections, indicating that the erstwhile stable central portions are losing some of their former stability. This also suggests that cracks in the central portion may be losing some of their ability to absorb movement, probably because of infiltration of incompressible material.

Physical Condition of Pavement—Generally speaking, the condition of the pavement is excellent and compares favorably with that of the adjacent 4½ miles of uniform 10-in. standard pavement which was constructed as a part of the same contract. The standard pavement has a 6-in. granular sub-base, 78-lb. wire fabric reinforcement, contraction joints at 100-ft. intervals and metal center joint with tie bars.

Transverse cracks are for the most part invisible to motorists, even when traveling at moderate speeds, except where maintenance workers misinterpreted instructions and sealed a number of cracks with asphalt. The pavement surface is smooth and has unusually good riding qualities. There is none of the rhythmic thud and pitching motion often experienced on pavements with closely spaced joints.

Since the pavement was built without a center joint, longitudinal cracking has developed, as expected. This 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. At several locations there are two approximately parallel longitudinal cracks, and at one place there are three. These cracks are noticeable only when one drives slowly.

The projected centerline lengths of the longitudinal cracks observed in the various test sections during several surveys are given in Table 1 in percent of the length of the section. It is immediately apparent from these data that there is a marked difference between the sections that were constructed in the fall of 1947 and those constructed in the spring of 1948. Longitudinal cracking is considerable in the former and almost entirely absent in the latter. Although most of the cracking occurred in the 7-in. sections, all of which were constructed in the fall, this appears to be coincidental, inasmuch as the 8-in. section with 1.0 percent of steel, which was the only 8-in. section constructed in the fall, shows about the

same cracking as the corresponding 7-in. section. Age does not appear to be a factor, since all of the fall construction showed considerable cracking at the age of 1 yr., while those sections which were constructed in the spring have practically no longitudinal cracking after 2½ yr. Apparently there is something about the time of construction that affected the development of longitudinal cracks, but at the present time an explanation for this behavior is not known.

It was expected that some failures would occur in the pavement, since, in order to establish the minimum steel for proper performance, two sections were purposely designed with less steel than previous experience indicated might be adequate. Only two serious structural failures have occurred in the pavement so far, both being located in the 8-in. test section containing 0.3 percent steel. They were highly localized and involved relatively small areas of pavement.

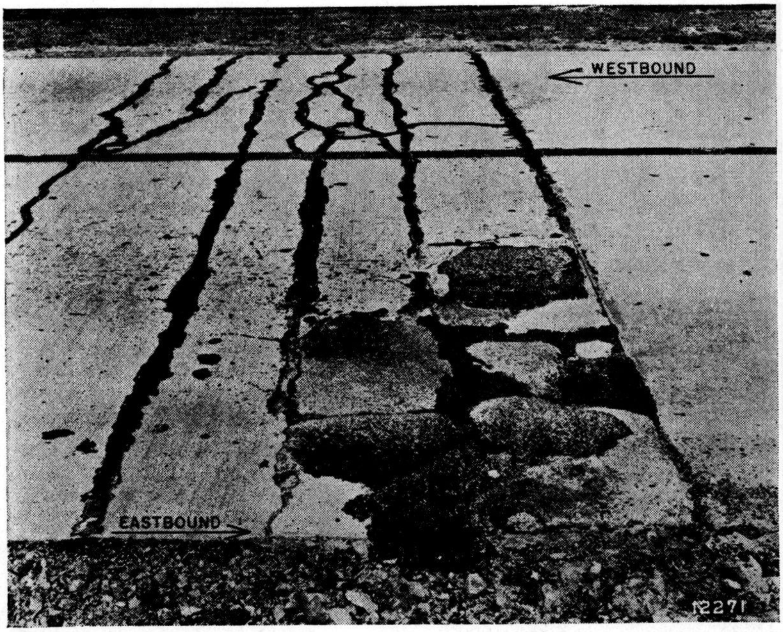

Figure 12. Localized Failure Near Sta. 292 + 75 as Seen from South Side of Slab (8-in. Pavement, 0.3 Percent Steel).

It will be noted that in the case of the sections constructed in the fall of 1947, the amount of longitudinal cracking increases with the percentage of longitudinal steel. The reason for this is not presently apparent, but it is believed that the longitudinal steel, in restraining the longitudinal warping, may also restrain the transverse warping in somewhat the same manner.

There is no apparent difference in the cracking where the ⅜-in. transverse bars are spaced 12 in. on centers and where they are 18 in. on centers. It does not appear that the longitudinal crack will be objectional unless its width increases or it develops unsightly spalling.

The larger of the two occurred adjacent to a construction joint at Station 292 + 75 and involved a length of about 8 ft., which included four closely spaced transverse cracks. It was first observed on October 15, 1948, only 5 months after construction. Pumping had developed at the construction joint and at two adjacent transverse cracks and severe spalling had developed at the transverse cracks. In November 1948, this area was undersealed with asphalt in an attempt to reduce the pumping. The failure became progressively worse, and in March 1950 the eastbound lane had developed to the condition shown in Figure 12.

Figure 13 shows the condition in the westbound lane at the same location and at the same time. It will be noted that the pavement in this lane is in comparatively good condition and there is none of the serious distress that developed in the other lane. The reason for this difference is not known since it does not appear that traffic or soil conditions in the two lanes are relatively different. It does, however, emphasize the local nature of the failure.

Additional evidence of the localized nature of this failure is seen in Figure 14. This photograph shows the excellent condition of the pavement east of the construction joint. The first transverse crack is 47 ft. east of the joint. Figure 15 shows that the pavement west of the joint is also excellent. The pavement shown on both sides of the damaged area is representative of the general condition of the remainder of this test section and, for the most part, the other sections are equally as good or better. It is needless to say, therefore, that the general condition of the entire pavement is good.

While the major causes of this failure are thought to be insufficient steel and poor subgrade, the failure is also related to unusual conditions prevailing at the time of construction, which deserve discussion. On May 10, 1948, paving, which progressed from east to west in this section, was terminated at the construction joint shown in Figures 12 and 13. Paving was not resumed until May 19, when almost 1500 lineal ft. of concrete was laid. In the meantime the concrete immediately east of the construction joint had an opportunity to gain considerable strength and for its shrinkage to stabilize. Shortly after construction the four transverse cracks shown in the photographs developed within 8 ft. of the construction joint, due probably to the resistance to shrinkage offered by the reinforcing steel which was securely anchored in the older slab. Undoubtedly the closely spaced cracks and poor subgrade made the pavement extremely flexible, which resulted in unusually large deflections under heavy vehicles. Considering this condition and the characteristics of the subgrade, it is not surprising that pumping occurred.

The other failure occurred at Station 282 + 40 and involved two transverse cracks about

Figure 13. Localized Failure Near Sta. 292 + 75 as Seen from North Side of Slab (8-in. Pavement, 0.3 Percent Steel).

Figure 14. View Looking East from Sta. 292 + 75 Showing Excellent Pavement Beyond Localized Failure.

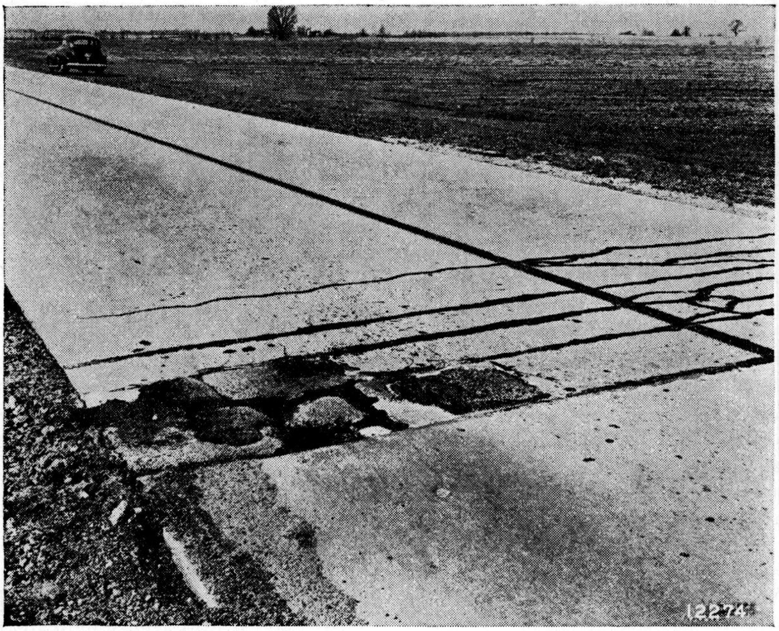

Figure 15. View Looking West from Sta. 292 + 75 Showing Excellent Pavement Beyond Localized Failure.

3 ft. apart in the eastbound lane. Excessive deflection and pumping occurred also at this location, with resulting deep spalling of the transverse cracks.

About May 1, 1950, the condition of the pavement had become so bad at both locations that it became necessary to remove the broken concrete and replace it with concrete patches. At the larger failure the patch was 8 ft. long for the full width of the eastbound traffic lane. The other patch was about 3 ft. long at the centerline and 6 ft. long at the south edge of the pavement. In neither case was it necessary to patch the westbound lane.

It was possible to remove the old concrete without a great deal of damage to the reinforcing bars. Only one bar was broken at each location during removal of the old concrete. At the larger failure four bars had broken under traffic. An inspection of the concrete removed from the larger failure showed it to be of inferior quality, which points to the possibility that the concrete placed in that area, the first batch poured that day, may have been excessively wet when placed or poorly proportioned.

At both locations the subgrade was found to be wet and unstable for a depth of 7 in. and this material was removed and replaced with sand. A French drain was constructed across the shoulder at each location to provide drainage for the area. An excellent job was performed in placing both patches, indicating that this type of pavement can be properly repaired if failures occur.

There have been rumors of wide spread pumping on the experimental pavement, but to date nothing really alarming has been found, except for the two distressed areas just described. During a thawing period in February 1949, it was observed that about 5 percent of the cracks in the 8-in. section containing 0.3 percent steel, and less than 1 percent of the cracks in the 7-in. sections containing 0.5 and 0.7 percent steel, were stained with soil. Although there was no visible ejection of slurry as heavy vehicles passed over the cracks, this condition indicated that some of the cracks are wide enough to permit the passage of water-borne soil and, therefore, it should be considered as incipient pumping.

The condition at that time had no apparent effect on the performance of the pavement and, while it was thought that it might develop further, subsequent observations following heavy rains indicate that this has not been true and, in fact, the condition at these cracks appears to have improved. During a recent survey pumping was observed only at some of the 4-in. expansion joints which separate the test sections, and no pumping was found at transverse cracks in any of the test sections. Pumping at the expansion joints is to be expected because the joints interrupt the structural continuity and provide channels for easy access of water to and from the subgrade. These joints are not a necessary part of a continuously reinforced pavement and their performance should not be considered when judging the behavior of the experimental pavement.

An unusual condition, and one which may have been the foundation for the pumping rumors, is the presence of many vertical holes in the earth shoulders where they meet the edge of the pavement. Casual observation might easily lead to the conclusion that these holes are produced by pumping, but a more thoughtful consideration of the facts indicates this is not true.

A characteristic pumping hole usually has a deposit of subgrade material built up around its rim, much like a crawfish hole, caused by the outward flow of slurry. But the tops of the holes along the experimental pavement are erroded, indicating that the flow has been inward and not outward. Near the bottom of the slab, the hole divides into a number of smaller passages which branch out from the main channel. It is believed the holes are the result of surface 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. This is confirmed by the fact that there are comparatively fewer holes along the north edge where the paving equipment operated and resulted in better compaction of the shoulder. It also appears that the holes are less numerous than when first observed, indicating that shoulder maintenance has tended to correct the condition.

A similar condition has recently been observed along the edges of pavements placed on granular sub-base in this as well as other states and investigations indicate that these may have resulted from the causes cited above. While these holes are undesirable, particularly on a pavement constructed over natural subgrade, because they provide chan-

nels through which water can easily reach the subgrade, and in this way may eventually contribute to pumping, there is no evidence to indicate that they were caused by pumping or are pumping at the present time.

Conclusions—The experimental pavement is not old enough to justify a definite conclusion as to its ultimate performance but, based on its present behavior, it would appear that continuously reinforced concrete pavements properly designed and constructed for their environments would give good performance. In view of the high steel stresses measured in 0.7 percent steel, it may be necessary to provide more steel than was indicated by earlier experiments. It also may be found that continuous reinforcement, while it may be a factor in controlling pumping, cannot by itself economically solve this problem and, therefore, it may be necessary on future experiments to revise the design accordingly. Perhaps, overall performance would be improved by a combination of continuous reinforcement and a thin granular sub-base. These possibilities should be studied in other projects.

In the final analysis the criterion by which this type of construction is judged will be an economic one. Whether pavements of this type are practical will depend on their original cost, service life and the cost of maintenance. In other words, will it give more and improved service for the money expended than other types of pavement.

A study of the contract unit prices for the experimental pavement and the adjacent standard pavement, built under the same contract, shows that the contractor 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. mesh reinforced pavement with 6-in. granular sub-base. Neither the experimental nor the standard pavement has required much maintenance to date and, of course, no estimate can be made at this time of the probable service life of the various test sections of continuously reinforced pavement. It appears that further study and additional experimental projects will be required before the question of economics can be answered.

REPORT ON EXPERIMENT WITH CONTINUOUS REINFORCEMENT IN CONCRETE PAVEMENT—NEW JERSEY

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SYNOPSIS

This paper reports the observed behavior and performance of two sections of continuously-reinforced concrete pavement constructed in New Jersey in 1947. These sections, which are each approximately one mile long, carry relatively heavy truck traffic. Some of the significant observations are as follows:

1. Both sections undergo an annual over-all change in length of approximately 2 in.
2. Except for minor movements limited to a few hundredths of an inch, all interior points located more than 700 ft. from the ends of the sections have undergone no longitudinal changes in position—that is, in each section there is an extensive central region which has remained essentially at constant length.
3. Except for distances ranging from 64 ft. to 177 ft. at their ends, both sections now contain hundreds of transverse cracks spaced from less than 6 in. to approximately 20 ft. apart. All of these cracks are essentially at right angles to the pavement. There has been no apparent longitudinal cracking.
4. There is a closer spacing of the cracks in the section containing the higher percentage of steel.
5. The cracks have almost doubled in number during the past three years.
6. The crack widths in both sections are extremely variable.
7. The maximum measured width of crack (at gauge plugs) is .03 in.