DEPARTMENT OF DESIGN

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EXPERIMENTS WITH CONTINUOUS REINFORCEMENT IN CONCRETE PAVEMENTS

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SYNOPSIS

For some years highway engineers have been interested in the practicability of concrete pavements constructed without transverse joints and reinforced longitudinally with continuous bonded steel in sufficient amount to hold all cracks closed

In the fall of 1938 a number of continuously reinforced sections, ranging from 20 to 1,310 feet in length, were constructed near Stilesville, Indiana, on US Route 40 as a cooperative research project to study the effects of varying amounts of longitudinal steel in sections of various lengths

The behavior of the sections during the first 10 years of service life conclusively shows that continuous reinforcement can be depended upon to prevent the opening of transverse cracks in concrete pavements In the long, heavily reinforced sections many fine cracks have developed in the central region. These cracks have not opened and have raveled only slightly with traffic and exposure, a condition that has required no maintenance and may be considered superficial The sections have remained strong, durable structural units.

The concrete appears to be sound throughout, there has been no spalling and there is a complete absence of longitudinal cracking above the bars In fact, the manner in which the steel has held closed all cracks, especially those in the more heavily reinforced sections, is believed to have been conducive to distributed interfacial pressure at the cracks which should tend to minimize damage to the concrete from concentrations of pressure such as sometimes develop at cracks in plain concrete pavements

Pumping has developed at many of the transverse joints but, with two exceptions, has not been observed at any of the vast number of transverse cracks. This indicates that a concrete pavement without transverse joints and containing adequate longitudinal reinforcement is not nearly so susceptible to pumping as pavements of other designs.

In spite of the many transverse cracks that have developed in the long sections, the riding quality of the pavement has remained excellent and the pavement itself has been protected from damaging impact forces such as tend to develop where the surface alinement is not maintained.

Recently two other experimental projects have been built with continuously reinforced concrete pavement, one in New Jersey and the other in Illinois These contain sections of greater length than the longest section of the Stilesville experiment and the amount of longitudinal steel used is less. (See *Proceedings* Highway Research Board, Vol 27 (1947). Papers by Woolley, Van Breemen, and Russell)

Since 1938 the Bureau of Public Roads and the State Highway Commission of Indiana have cooperated in making detailed observations of an experimental concrete pavement containing a wide range of continuously reinforced sections Three published reports[1] have described the scope of the study, the construction of the project and the observed behavior of the pavement during the first 5 years of service. The present report, which

[1] Earl C Sutherland and Sanford W. Benham "Experiments with Continuous Reinforcement in Concrete Pavements," *Proceedings*, Highway Research Board, Vol. 19 (1939); also *Public Roads* Vol. 20, No. 11, January 1940 (*Continued p 45 col 1*)

may be considered the major report of the investigation, describes the performance of the various sections over a period of 10 years. Although previously published information will be avoided as much as possible, certain essential data will be repeated for clarity and completeness.

The experimental pavement is a 9-7-9-in. thickened-edge type, 20 ft. wide and approximately 6 miles long. It is located near Stilesville, about 30 miles west of Indianapolis, and was constructed during September and October of 1938 as part of the eastbound lanes of the divided highway US 40.

Traffic counts indicated an average annual daily volume (in both directions) of 3,500 vehicles in 1941 when the pavement was 3 years old. Of these, 1,125 were trucks and busses, the maximum daily gross load being at least 48,000 lb In 1948, when the pavement had been in service for 10 yr, the average annual daily traffic volume had increased to 5,100 vehicles, trucks and busses comprising 1,280 of the total. At this time the maximum daily gross and axle loads were at least 51,000 and 20,400 lb, respectively.

TABLE 1
DETAILS OF REINFORCEMENT IN THE EXPERIMENTAL PAVEMENT
RAIL STEEL BARS (DEFORMED)

Number of Sections for Each Length[1]	Length of Each Section[2]	Calculated Maximum Stress in Steel	Reinforcement Size and Spacing		Percentage of Longitudinal Steel[3]	Average Tensile Strength of Longitudinal Steel	
			Longitudinal	Transverse		Yield Point	Ultimate
	ft	*lb per sq in*			*Percent*	*lb per sq in*	*lb per sq in*
2	600 840 1,080 1,320	25,000 35,000 45,000 55,000	1-in round bars, 6 in c to c	½-in round bars, 24 in c to c.	1 82	63,300	113,200
4	340 470 610 740	25,000 35,000 45,000 55,000	¾-in round bars, 6 in c to c	½-in round bars, 24 in c to c.	1 02	64,400	113,300
4	150 210 270 330	25,000 35,000 45,000 55,000	⅝-in round bars, 6 in. c to c	½-in round bars, 24 in c to c	45	68,800	115,300
6	80 120 150 180	25,000 35,000 45,000 55,000	⅝-in round bars, 6 in c to c	½-in round bars, 24 in c to c	26	66,700	93,600
6	40 50 60 80	25,000 35,000 45,000 55,000	½-in round bars, 6 in c to c	½-in round bars, 12 in. c to c	11	60,300	84,600

[1] The term "section" as used in this report refers to a lane or 10-ft width of pavement, thus the number "2" indicates a pair of sections, one being on each side of the center joint.
[2] The lengths of the longer sections are nominal lengths and may be either 5 or 10 ft greater than the actual length in cases where a pair of bridge-type joints were installed.
[3] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of t e concrete slab

Briefly, the experimental pavement consists of sections ranging in length from 20 to 1,310 ft. Incorporated in these sections are various amounts of steel for each of three types of reinforcement. The number and range in length of the individual sections, together with pertinent data on the reinforcing steel used in each, are given in Tables 1, 2 and 3. The lengths of the sections necessary to develop the steel stresses shown in the tables were calculated on the basis of certain assumptions as to the resistance offered by the subgrade as the pavement expands and

Harry D Cashell and Sanford W Benham, "Progress in Experiments with Continuous Reinforcement in Concrete Pavements," *Proceedings*, Highway Research Board, Vol. 20 (1940); also *Public Roads*, Vol 22, No 3, May 1941.

Harry D Cashell and Sanford W Benham, "Experiments with Continuous Reinforcement in Concrete Pavements—A Five-Year History," *Proceedings*, Highway Research Board, Vol. 23 (1943) (Condensed).

contracts. It was assumed that the resistance would be constant and could be expressed as a coefficient equal to 1½ times the weight of the pavement.

The maximum steel stresses were intended to be such that the elastic limit of the reinforcement would be approached in the longest section of each group, producing, under repeated stressing, inelastic elongation with consequent opening of the cracks.

Since a wide range in slab end movements was expected in the sections of various lengths, several different widths of transverse joint opening were provided. The shorter sections were separated by conventional dowel-type joints having widths of either ¾ or 1 in. A joint of a type similar to that frequently used at bridge approaches and designed to permit a 1½-in. movement in each direction was placed between intermediate-length sections; whereas for the longer sections provision was made for approximately twice this amount of movement by means of a pair of the bridge-type joints spaced 10 ft. apart.

In addition to the regular sections of the experimental pavement; that is, the sections containing continuously bonded steel, four special sections each 500 ft. long were included. In these, weakened-plane joints were spaced at 10-ft intervals and the bond between the longitudinal steel and the concrete was broken purposely for a distance of 18 in. on each side of each transverse joint.

The discussion of the 10-yr. performance of the pavement will be presented in six parts as follows: (1) periodic elevation changes of the regular sections; (2) daily, annual and progressive changes in the length of the regular sections, (3) development, distribution and present condition of cracks in the regular sections, (4) behavior of the 500-ft. special sections; (5) occurrence of pumping; and (6) smoothness of the pavemnt.

TABLE 2
DETAILS OF REINFORCEMENT IN THE EXPERIMENTAL PAVEMENT
BILLET STEEL BARS (DEFORMED)
Intermediate Grade

Number of Sections for Each Length[1]	Length of Each Section[2]	Calculated Maximum Stress in Steel	Reinforcement Size and Spacing		Percentage of Longitudinal Steel[3]	Average Tensile Strength of Longitudinal Steel	
			Longitudinal	Transverse		Yield point	Ultimate
	ft.	*lb per sq. in.*			*Percent*	*lb. per sq. in*	*lb per sq. in*
2	360 600 840 1,080	15,000 25,000 35,000 45,000	1-in round bars, 6 in. c to c	½-in round bars, 24 in c to c	1 82	46,900	78,000
4	200 340 470 610	15,000 25,000 35,000 45,000	⅝-in. round bars, 6 in c to c	½-in round bars, 24 in c to c.	1 02	49,100	78,500
4	90 150 210 270	15,000 25,000 35,000 45,000	½-in round bars, 6 in c to c	½-in. round bars; 24 in c to c.	45	51,400	78,600
6	50 80 120 150	15,000 25,000 35,000 45,000	⅝-in round bars, 6 in c to c.	⅝-in. round bars, 24 in c to c	26	55,500	81,900
6	20 40 50 60	15,000 25,000 35,000 45,000	½-in round bars, 6 in c to c.	½-in round bars, 12 in c to c.	.11	56,900	77,300

[1] The term "section" as used in this report refers to a lane or 10-ft. width of pavement, thus the number "2" indicates a pair of sections, one being on each side of the center joint
[2] The lengths of the longer sections are nominal lengths and may be either 5 or 10 ft greater than the actual length in cases where a pair of bridge-type joints were installed
[3] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab

PART 1.—PERIODIC ELEVATION CHANGES OF THE SECTIONS

Three sets of precise elevation measurements have been made over the entire length

of the experimental pavement, as follows: (1) late fall of 1938 or shortly after construction of the sections, (2) fall of 1939 or approximately 1 yr. after construction; and (3) severe winter of 1939–40 when the frost had penetrated the ground to a depth of about 20 in In addition, during the first 5 yr., elevation measurements were made at more frequent intervals over selected sections. All such measurements were made on reference points installed in the right-hand or heavily traveled lane of the pavement.

At the end of the first year the majority of the elevation changes were very small. Specifically, only 7 percent of the 487 mid-lane locations at which measurements were made showed a change in elevation greater than ¼ in. when compared with the base elevations established soon after construction.

During the peak of the severe winter of 1939–40 increases in elevation were generally within the range of 0.2 to 1 0 in. as compared with the elevations determined the previous fall. It was observed also that, in most instances, heaving was greater at the expansion joints than at points elsewhere in the sections, averaging 0 47 in. at 151 expansion joints and 0.33 in. at 185 points elsewhere in the sections. These data emphasize the importance of tightly sealed joints in pavements exposed to freezing conditions.

TABLE 3
DETAILS OF REINFORCEMENT IN THE EXPERIMENTAL PAVEMENT
WIRE FABRIC (COLD DRAWN WIRES)

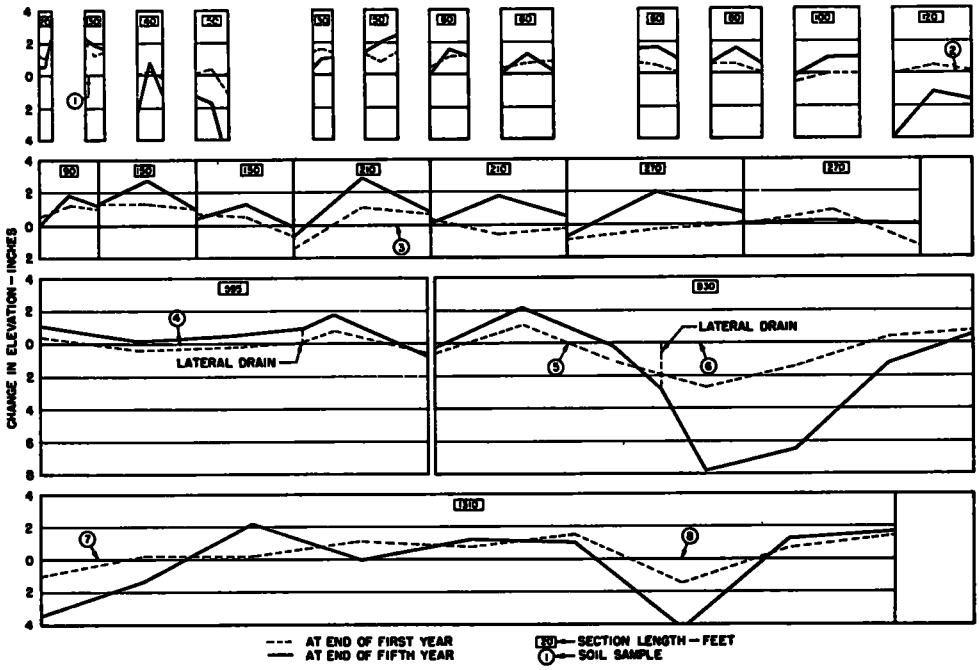
Number of Sections for Each Length[1]	Length of Each Section[2]	Calculated Maximum Stress in Steel	Weight of Reinforcement	Reinforcement Size and Spacing		Percentage of Longitudinal Steel[3]	Average Ultimate Tensile Strength of Longitudinal Steel
				Longitudinal	Transverse		
	ft	*lb per sq in.*	*lb per 100 sq ft*			*Percent*	*lb. per sq in*
6	140 190 250 310	25,000 35,000 45,000 55,000	149	No 4-0, d = .3938 in., 4 in. c to c.	No 3, 12 in. c to c	.42	81,800
6	90 130 170 200	25,000 35,000 45,000 55,000	107	No 4-0, d = 3938 in , 6 in c to c	No 3; 12 in c o c.	.28	80,300
6	80 110 140 170	25,000 35,000 45,000 55,000	91	No 3-0, d = 3625 in., 6 in c to c	No 4, 12 in c to c	24	89,100
6	60 80 100 120	25,000 35,000 45,000 55,000	65	No 0, d = 3065 in , 6 in. c. to c	No 6, 12 in c to c	.17	83,700
6	30 50 60 80	25,000 35,000 45,000 55,000	45	No 3, d = 2437 in., 6 in. c to c	No 6, 12 in c to c	.11	81,000
6	20 30 40 50	25,000 35,000 45,000 55,000	32	No. 6, d = .1920 in., 6 in c to c	No 6, 12 in. c. to c	.07	88,700

[1] The term "section" as used in this report refers to a lane or 10-ft width of pavement, thus the number "2" indicates a pair of sections, one being on each side of the center joint.
[2] The lengths of the longer sections are nominal lengths and may be either 5 or 10 ft greater than the actual length in cases where a pair of bridge-type joints were installed
[3] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab.

Examples of typical changes in pavement elevation are shown in Figures 1 and 2.

The elevation changes in Figure 1 are those observed on selected sections in the fall at the end of the first and fifth years of pavement life, using as a base the elevations established shortly after construction. Subgrade soil data applicable at the time the pavement was placed are also given in this figure.

The data of Figure 1 indicate that the elevation changes at the end of 5 yr. were appreciably greater in magnitude and were less uniform than at the end of the first year.



SOIL SAMPLE NO.	1	2	3	4	5	6	7	8
SILT – PERCENT	38	43	60	58	63	39	46	43
CLAY – PERCENT	16	18	18	18	13	12	19	18
LIQUID LIMIT	32	34	36	37	43	28	35	29
PLASTICITY INDEX	14	11	12	12	19	11	14	11

SOIL SAMPLE NO	1	2	3	4	5	6	7	8
	MOISTURE CONTENT – PERCENT							
0 – 3 INCH DEPTH	16.0	11.0	13.6	11.5	12.9	19.2	16.8	9.8
3 – 12 INCH DEPTH	18.0	13.0	18.0	14.6	15.8	13.5	18.7	15.7
12 – 24 INCH DEPTH	18.0	19.0	18.0	13.1	20.9	13.5	19.6	12.8

Figure 1. Changes in Elevation of Selected Sections at the End of the First and Fifth Years of Pavement Life; Also, Physical Characteristics of the Subgrade Soil at the Time Pavement was Placed

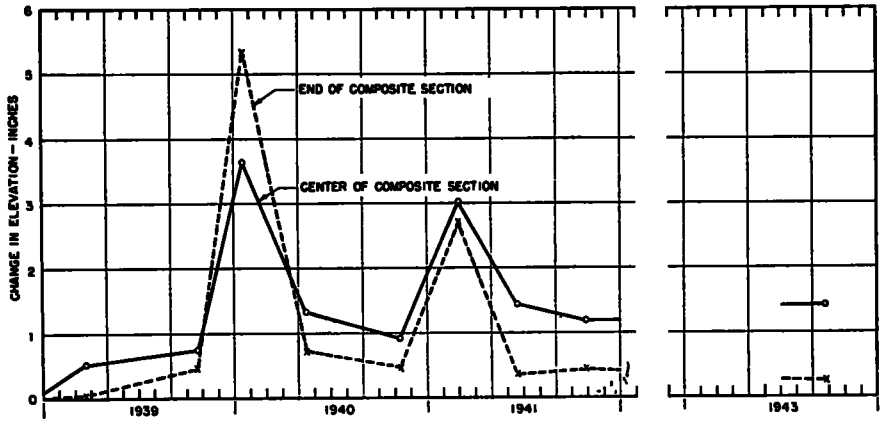


Figure 2. Changes in Elevation of the Center and End of a Composite Section (Average of 24 Representative Sections). Base Measurements Obtained in December 1938

The greatest change was a settlement of 0.8 in near the center of the 830-ft. section However, measurements taken in this area at the end of 10 yr. showed virtually no change with respect to the 5-yr profile

Figure 2 shows the changes in elevation of a composite section, this figure being prepared from measurements made from time to time at the centers and at the ends of 24 representative sections, using as a base the elevations established soon after construction. Thus, the elevation changes shown for the center and for the end of the composite section are, respectively, the average changes in elevation of the 24 centers and of the 48 ends of the representative sections.

The positions of the center and end of the composite section indicate that (1) the pavement as a whole raised slightly with respect to the basic elevation; and (2) excepting in the severe winter of 1939–40, the changes in elevation were greater at the center than at the end, suggesting a permanent downward warping at the end of the composite section with respect to its center. It is believed that soil displacement resulting from pumping or consolidation of the subgrade at the joints may be primarily responsible for this condition of distortion.

PART 2.—DAILY, ANNUAL AND PROGRESSIVE CHANGES IN LENGTH OF THE SECTIONS

Daily, annual and progressive changes in length were measured at the ends of a number of representative sections. These length changes were carefully determined either by measurement to fixed reference points located at the ends of a section or by measurements across joints between sections of equal length Cross-joint measurements give the width changes of joints which, when determined for a joint separating sections of equal length, should approximate the total length change of one of the joining sections All of these measurements were from points installed in the surface of the right-hand or heavily traveled lane of the pavement

It will be recalled that all transverse joints were designed to care for a reasonable amount of slab expansion and, as far as can be determined, the observed length changes of the sections were unaffected by restraint at the joints during the first 5-yr of pavement life Considerable care was exercised during construction in correctly aligning the round steel dowels used in the joints which separate the shorter sections so that restraint from this source was reduced to a minimum. However, over a period of time, the joints, especially the bridge-type joints, gradually became filled with soil and with bituminous material used for their maintenance so, at present, the movements of several of the longer sections may be restrained to some extent during periods of maximum expansion.

For a given temperature or moisture change in the concrete, there should be a proportionate change in section length provided the section remained structurally intact and was not restrained in any manner. Actually, however, the length changes of the sections of this study are affected by restraint that may develop at transverse joints and by such factors as subgrade resistance to slab movement, differences in the thermal coefficients of steel and concrete, moisture changes of the concrete and not of the steel and changes in width of existing transverse cracks. Just how much influence each of these exerts cannot be determined, but it is believed that the subgrade resistance is the most important. Since the thermal coefficients of steel and concrete are nearly the same, that for steel being somewhat the greater, it seems reasonable that this factor would have little effect on the length changes of the sections, particularly during expansion As for transverse cracking, it will be shown later in the report that very few cracks occurred in sections having lengths less than 120-ft and the cracks that developed in the longer, more heavily reinforced sections are extremely fine Thus, for either a daily or annual time period, the cumulative change in width of all transverse cracks in a given section should be small compared to the overall length change of the section.

Daily Length Changes of the Sections Measured—

In considering the daily length changes it will be recalled that all measurements at the joints were made at the level of the upper surface of the pavement and were, therefore, affected in some degree by changes in the condition of warping at the end of the slabs. In this investigation it was found that the cor-

rections for daily warping ranged from 0.003 to 0.005 in. for days when a large temperature change occurred in the pavement. These corrections were applied only to sections having lengths of 60 ft. or less because in the longer sections the magnitude of the length changes was such as to make the correction unimportant.

In Figure 3 are shown the relations between section length and change in section length as found for a daily mid-depth pavement temperature drop of 24 deg. F. and a daily mid-depth pavement temperature rise

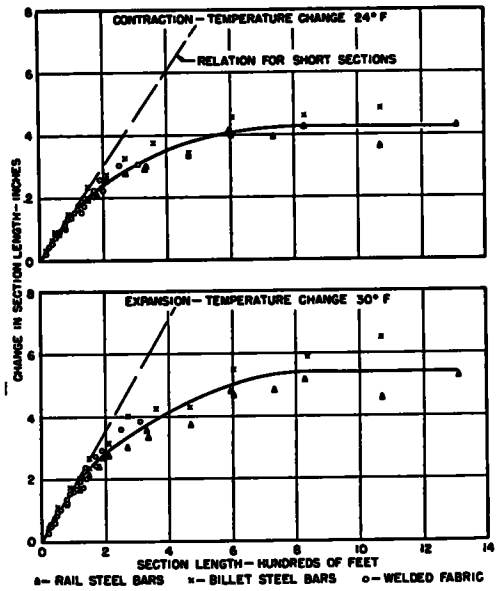


Figure 3. Relation Between Section Length and Daily Change in Length

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of 30 deg. F. The data for this figure were obtained from 64 sections that cover the rang of section lengths for all percentages of long tudinal steel included in each of the three types of reinforcement The slopes of the lightweight straight lines shown in the figure were computed from the change in length of 19 uncracked sections, 20 to 60 ft. long, and thus represent, for the temperature changes mentioned, the rates of length change for short sections that are comparatively free to expand and contract. From these the coefficients of daily length change for short sections, which should approximate the thermal coefficient of the concrete, are readily obtained.

It is apparent from Figure 3 that sections up to approximately 75 ft. long move with as much freedom as the very short sections. The change in length of sections greater than about 75 ft. is restrained by subgrade resistance and perhaps other factors and this restraint, the effect of which is shown as departure from the slope line established by the short sections, increases rapidly as the sections become longer. After a section length of about 800 ft. is reached, the curves level off, indicating that the maximum restraint has been developed and that sections whose lengths are greater than this may be expected to show total length changes of equal magni-

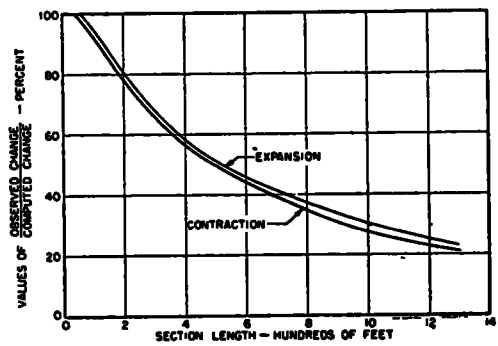


Figure 4. Observed Daily Changes in Section Lengths Expressed as Percentages of the Computed Changes in Length of Equivalent Unrestrained Sections

tude. This suggests that the central portion of sections greater than 800 ft will be completely restrained during quick changes in average concrete temperature. Applied to the 1,310-ft. section, this would mean that the central 500 ± feet of this section did not move during the daily temperature changes for which data are shown in Figure 3

The great influence of subgrade resistance on the relatively rapid daily change in length of the sections is clearly indicated in Figure 4. This figure, prepared from the curves of Figure 3, shows the observed change in length of a section expressed as a percentage of the change in length of an unrestrained section of the same length. The length changes of the unrestrained sections were calculated from the data obtained from the very short

sections. Reduced to this basis, the observed daily length changes were about the same for both the expansion and contraction cycles.

In the discussion of Figure 3 it was suggested that the central portion of long sections may be completely restrained during quick changes in pavement temperature This seems to be confirmed by the data given in Figure 5, data which show the longitudinal movements observed at the center, quarter-points and ends of a 1,310-ft. section for a daily mid-depth temperature drop of 19 deg. F. and a rise of 25 deg. F. The value shown

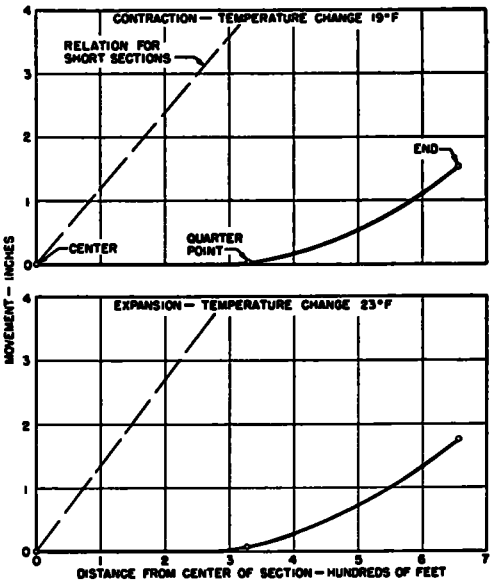


Figure 5. Daily Movement at the Center, Quarter Point and End of a 1310-Foot Section

for the quarter-point and for the end is in each case the average value obtained from measurements at both quarter-points and at both ends of the section.

These data indicate that during contraction the movement at the ends of the 1,310-ft section was about 20 percent and at the quarter-points about 1 percent of the movement which would be found in an unrestrained section of the same length. Corresponding values for expansion were about 20 and 2 percent It will be noted that between 500 and 550 ft. of the central portion of the 1,310-ft. section did not move during the daily cycle, this length being comparable to the 500±-foot length that was estimated from the results of Figure 3.

The manner in which certain sections respond to a daily rise in pavement temperature is shown in Figure 6. The basic measurements for this figure were obtained in the early morning of a summer day and subsequent measurements were made at intervals until late afternoon

In the case of both the 470- and 1,310-ft. sections the relation between increase in temperature and change in overall section length remained linear until a total elongation of approximately 0.2 in. was attained, after which the rate of length change increased progressively with temperature, being more pronounced for the longer section. For example, during the 28-deg. F temperature

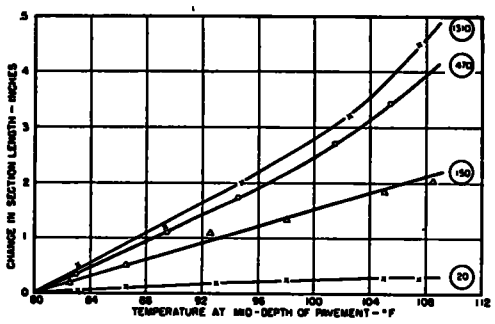


Figure 6. Effect of a Daily Rise of the Mean Pavement Temperature on the Change in Length of Several Sections. Figures in Circles Indicate the Length of Sections in Feet

rise shown in Figure 6, the 1,310-ft. section moved 0.19 in for the first 14-deg. temperature increase and 0.27 in. for the second 14-deg. increase. Since the movements of the sections are intimately related to the restraint offered by the subgrade, the increase in the rate of length change of the long sections suggests that, after a certain amount of slab displacement, the total or accumulated subgrade resistance continues to increase, but at a progressively decreasing rate.

Daily changes in length of a limited number of sections were observed each summer over a 4-yr. period on days when a large temperature change occurred in the pavement This 4-yr. period extended from the second through the fifth year of pavement life and, therefore, it is believed that the movements of the

sections were not affected by restraint at the transverse joints Table 4 gives the daily length change data for the various section lengths, reduced to unit values per deg. F. These values indicate that the coefficient of the daily length change of a given section did not change appreciably from year to year.

TABLE 4
SUMMARY OF VALUES OF COEFFICIENTS OF DAILY LENGTH CHANGE
(Based on Changes in Overall Section Length)

Section Length	Unit Change in Section Length per deg F × 10^{-7}					
	July 1940	June 1941		June 1942		July 1943
	AM to PM	PM to AM	AM to PM	PM to AM	AM to PM	AM to PM
ft						
20	49	49	49	41	52	
150		46	43	44	41	
335	29	32	29			
470		25	25	22	24	
600	21	24	24	21	23	
1070	13	13	14			
1810	9	11	11	8	10	10

The average of the five values obtained from the observed daily length change for the 20-ft. section (Table 4) is 0 0000048 per deg F. Because the 20-ft. section is free to expand and contract when subjected to a temperature change, this value should approximate the thermal coefficient of the concrete. As a matter of supporting data the slopes established by the short sections, as shown in Figure 3, when divided by the temperature change of the concrete give coefficient values of 0 0000053 per deg. F. for contraction and 0.0000049 per deg. F. for expansion.

Annual Length Changes of the Sections Observed—

Figure 7 contains the annual length change data for the various sections for the first, second, third and fifth years of the life of the pavement The annual change in length of a section was computed from data obtained in the morning of a midwinter day and in the afternoon of a midsummer day and, consequently, includes the length change that occurred between the morning of a winter day and the morning of a summer day plus the daily length change that occurred between the morning and afternoon of the aforementioned summer day. Since an effort was made to obtain these data during the coldest period of winter and the hottest period of summer,

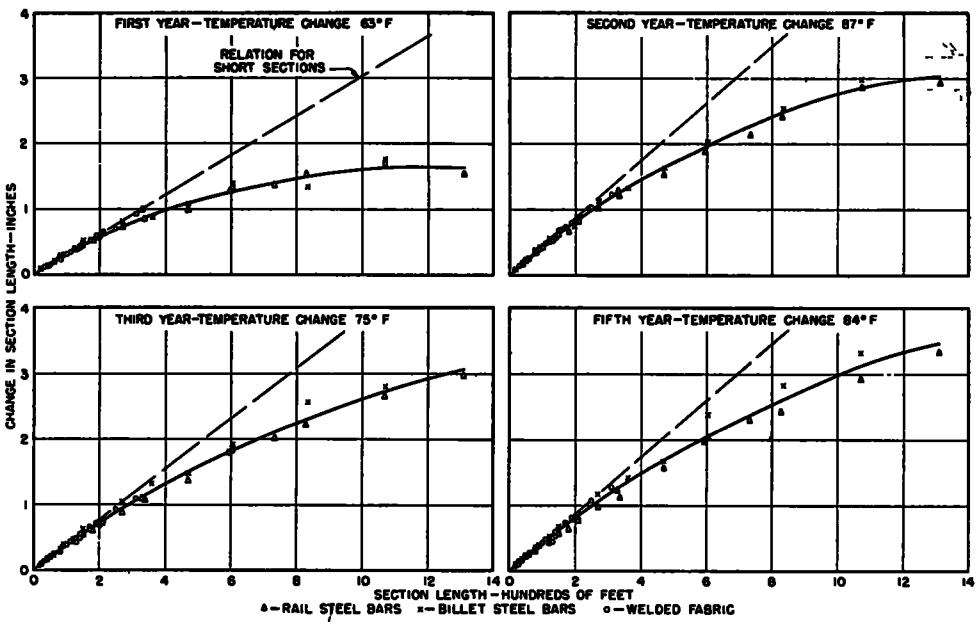


Figure 7. Relation Between Section Length and Annual Change in Length

the length changes shown are approximately the maximum for the annual cycle. The slopes of the lightweight dash lines represent respective annual relations determined from 19 uncracked short sections.

The type of reinforcement used in the various sections is denoted by symbol. There appears to be some tendency for sections containing billet steel bars to develop slightly greater annual length changes than equivalent-length sections reinforced with rail steel bars. This same tendency was noted in the daily expansion and contraction data of Figure 3 The cause for this apparent difference is not known.

It is indicated by the four curves of Figure 7 that sections up to approximately 150 ft long move with as much freedom during an annual cycle as do the very short sections. The length changes of sections greater than 150 ft, however, are restrained by the subgrade and this restraint increases progressively with increase in section length. The data of Figure 3 indicated that daily restraint to free movement was first noticeable in sections about 75 ft. long. It should be remembered that the annual length change data considered above include the effects of one daily cycle also A probable explanation for the observed difference just mentioned, is that, under the slowly developed temperature rise from winter to summer, sections up to, at least, 150 ft long moved freely because they encountered less restraint from the subgrade than obtains during the more rapid daily cycle of length change. Hence, the small amount of daily restraint to free movement of sections between 75 and 150 ft long, as shown in Figure 3, while present is not apparent in the curves of Figure 7.

In connection with this study of the annual length changes of the sections it is of interest to note the symmetry of movement that was found in the long sections For example, during the fifth annual period, the observed movement at one end of a 1,070-ft section was 1 50 in while at the other end the movement was 1.42 in ; likewise the movements at the two ends of the 1,310-ft section were 1 62 and 1 72 in. respectively.

To provide a more easily visualized comparison of the annual length changes of the various sections, Figure 8 was developed from Figure 7 in the same manner that Figure 4 was obtained from Figure 3 The curves of Figure 8 show not only the magnitude of the restraint in the various sections, but also that the sections expanded more freely progressively for each of the first three annual cycles. Thus, it appears that the sections encountered less subgrade resistance with each successive annual expansion period until, by the end of the third period, a condition of essential stability was reached.

Further evidence is added by the annual movements observed at the quarter-points of the 1,310-ft. section For the first year the annual movement at the quarter-points was about 10 percent of the movement to be expected at the quarter-points of an unrestrained section of equal length. For the

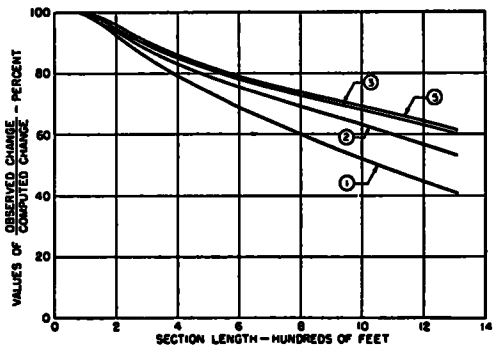


Figure 8. Observed Annual Changes in Section Lengths Expressed as Percentages of the Computed Changes in Length of Equivalent Unrestrained Sections. Figures in Circles Indicate Age of Pavement at Time of Observations.

second, third and fifth years the percentage values were respectively 31, 46 and 45

Sections Move with Greater Freedom During an Annual Period than During a Daily Period—

Table 5 shows the annual length changes of selected sections reduced to unit values per deg. F. These annual coefficients of length change, although expressed as unit values per deg. F, involve temperature, moisture, subgrade resistance and perhaps other factors The factor of moisture will be discussed later.

In comparing the coefficient values of the longer sections of Table 5 with those of Table 4 it is observed that coefficients for an annual expansion period are, in general, much greater than those for a daily expansion period, thus suggesting greater freedom of

movement of the sections during an annual period.

This condition is clearly shown, also, by comparing the curves of Figure 8 with those of Figure 4. For example, during the annual length change cycle for the 1,310-ft. section (fifth year) the observed movement was 62 percent of the theoretical length change for an unrestrained section of equal length; whereas

TABLE 5
SUMMARY OF VALUES OF COEFFICIENTS OF ANNUAL LENGTH CHANGE
(Based on Changes in Overall Section Length)

Section Length	Unit Change in Section Length per deg F $\times 10^{-7}$ for the Following Years of Pavement Life			
	First	Second	Third	Fifth
ft				
20	40	42	43	43
150	40	40	41	41
335	34	35	36	36
470	29	32	34	34
600	29	31	34	35
1070	21	26	28	29
1310	15	21	25	25

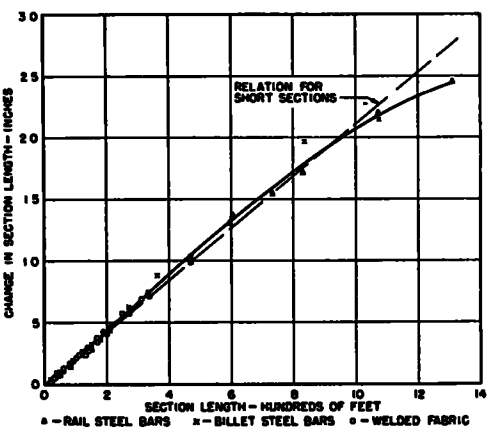


Figure 9. Relation Between Section Length and Change in Length—From the Morning of a Winter Day to the Morning of a Summer Day of the Same Year—Mean Pavement Temperature Change of 45 F

during the daily length change the value was only 22 percent. Hence, it is strongly indicated that the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place

Data lending further support to this observation are given in Figure 9 This figure shows the length changes of the sections that occurred between the morning of a day in February when the mid-depth pavement temperature was 32 deg F. and the morning of a day in late June of the same year when the mid-depth pavement temperature was 77 deg. F Therefore, the data do not include the effect of a quick daily temperature rise, but rather show only the comparative freedom with which sections of all lengths expanded under a slowly developed temperature rise of 45 deg F. These measurements were made during the third year of the life of the pavement, at which time stabilization of annual movement had developed

It appears from the cycle of length change shown in Figure 9 that sections up to about 900 ft long expanded as freely as the very short sections. The free movement of sections greater than 900 ft is restrained and, although the sections included in this study are not long enough to warrant definite conclusions, it is indicated by the rapid increase in restraint that the central portion of sections greater in length than approximately 1,700 to 1,800 ft would be in a state of complete restraint during an annual cycle.

The manner in which the sections up to 900 ft. long expanded from February to June indicates that subgrade resistance did not accumulate in these sections and, as a consequence, residual compression was probably absent on the morning of the June day when the pavement temperature was 77 deg. F. Therefore, it seems logical that as summer advances and the mean pavement temperature gradually rises, sections of considerable length, unless restrained at the joints, expand to their annual maximum without developing appreciable residual compression. If this is the case, then it would be expected that in late summer or early fall the comparatively large, sudden drops in temperature would cause comparatively large, direct tensile stresses to be developed in the sections, larger probably than at any other period during the year.

Again confirmatory evidence is supplied by the movements observed at the quarter-points of the 1,310-ft. section. During the fifth year of pavement life, a 0.61-in movement was recorded at the quarter-points of this section when it had expanded to its approximate maximum length for the annual cycle However, during the early fall of the

same year, after the pavement temperature had dropped approximately 50 percent of its winter to summer rise, the contraction of the section was restrained to the extent that the return movement at its quarter-points was only 0.06 in. or about 10 percent of the movement observed at maximum expansion.

In concluding this study of the annual length changes of the sections, it is of interest to compare sections in which the three different types of reinforcement were used and, also, those in which the maximum stresses in the longitudinal steel presumably varied considerably, in order to determine the effect of these factors on the relation between section length and annual contraction of the sections. These comparisons are shown in Figure 10 for a contraction period since, during such a period, maximum tensile stresses develop in the reinforcement The length changes given in this figure are the result of a 77-deg F fall in temperature that occurred between midsummer and midwinter of the third annual contraction period

A comparison of the three curves of Figure 10 indicates that, for a contraction period, the type of reinforcement had little influence on the observed length changes of the sections. For example, in the case of a 300-ft. section the measured length changes for the rail steel bars, billet steel bars and the welded fabric were 0.92, 0.98 and 1.00 in respectively.

The maximum steel stresses as calculated for the various sections are denoted, in Figure 10, by symbol. For a given section length these stress values may be considered as inverse indices of steel area. The orderly manner in which all points, regardless of symbol, fall on the curves in the figure is evidence that, within the ranges available for comparison in a given section length, the amount of the longitudinal steel exercises no significant control over the length changes. For example, two sections each approximately 600 ft. long, reinforced with rail steel bars, show essentially the same length change although one contains 1 82 percent of longitudinal steel while the other contains but 1 02 percent.

Examination of all of the transverse cracks in the regular sections indicated that, except for three in the 24 sections containing the 32-lb. wire fabric, all were held closed by the longitudinal steel The condition of these cracks will be discussed later

Since after 10 years of heavy-duty service none of the cracks in the regular sections showed evidence of inelastic deformation of the longitudinal steel (except the three just mentioned), it must be concluded that the assumptions used in computing steel stresses in the original design lengths were unduly conservative How closely the elastic limit of the steel has been approached during this period of service remains unknown.

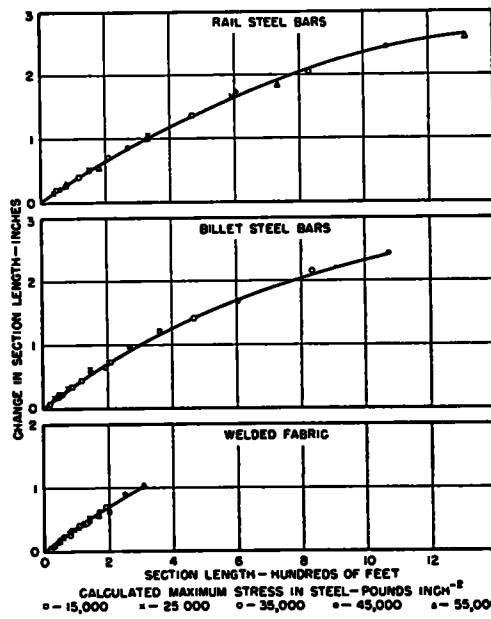


Figure 10. Effect of (a), Type of Reinforcement and (b), Calculated Maximum Steel Stress on the Relation Between Section Length and Annual Contraction—77 F. Temperature Drop

Progressive Length Changes of the Sections Determined—

To evaluate length changes of a progressive or permanent nature, measurements were made at the ends of a number of selected sections every February and August during the first 9 yr. of pavement life The February observations were obtained when the mid-depth slab temperature was approximately 32 deg F. and the August observations when the mid-depth slab temperature was approximately 92 deg. F. The effect of moisture on

the determination of the progressive length changes of the sections was minimized, as much as possible, since there is reason to believe that the moisture content of the pavement remains virtually stable in February and in August. Studies indicate that during these months the absorbed moisture is at the maximum and minimum, respectively, of the annual cycle of moisture change.

The data from the study of progressive or permanent changes in the lengths of the sections are given in Figures 11 and 12.

In Figure 11 the dash lines show the average February to August and August to February unit length changes of a number of short, uncracked sections, plotted with respect to the initial set of measurements obtained in February of 1939. These sections are 20 to 80 ft. long, comprise 340 ft. of pavement and are relatively free to expand and contract.

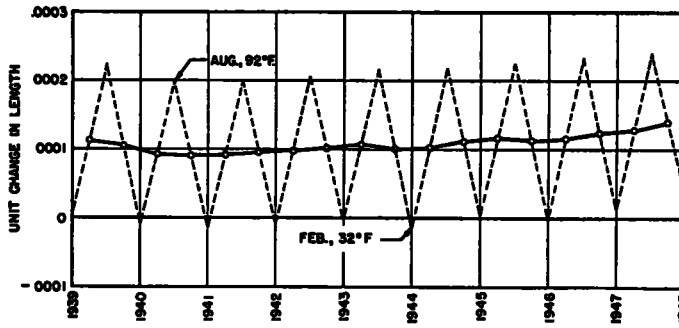


Figure 11. Annual Cycles of Length Change and Progressive Growth of Short, Uncracked Sections Expressed as Unit Changes in Length

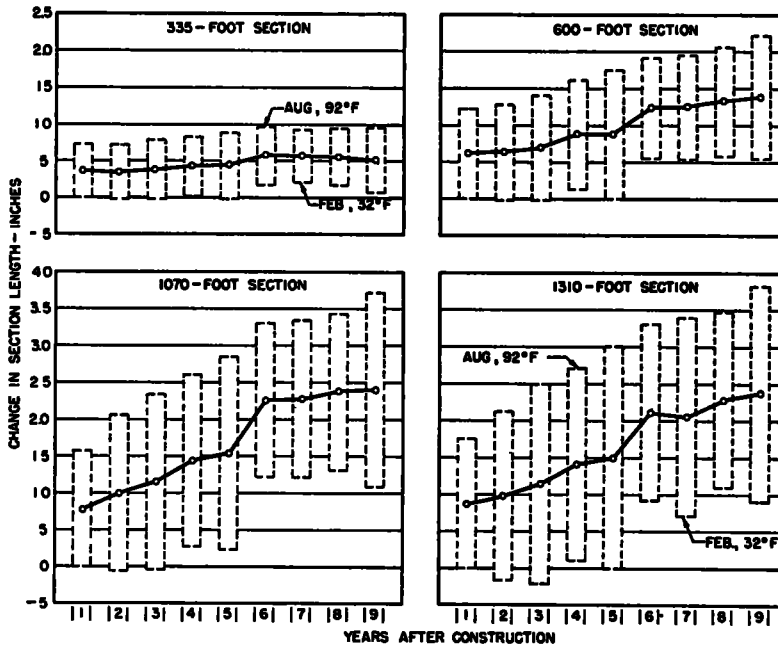


Figure 12. Annual and Progressive Length Changes of Several of the Longer Sections

The length changes are expressed as unit values per 60 deg F. change in slab temperature.

The solid line drawn through the mean points of the cyclic variations of Figure 11 indicates progressive permanent growth or increase in the lengths of the sections, this growth being more pronounced after the fifth year of pavement life In fact, the growth was so small during the early life of the pavement that in the 5-yr. report it was stated that "no definite indication of a permanent change in the length of the short sections was observed." Since these short sections are structurally intact there can be little doubt that a permanent increase in length is developing. This appears to be another instance of the tendency of some concretes, at least, to grow when subjected to repeated cycles of temperature and moisture change.

The difference between the high and low points of the mean line of Figure 11 represents a permanent unit increase of 0 000048 or an increase of approximately $\frac{1}{16}$ in. for a 100-ft. slab. A similar study of permanent growth of concrete pavement was made at the Arlington Experiment Farm, Virginia, by the Bureau of Public Roads[2] on a 40-ft., plain concrete test section. Over a 9-yr. period this test section showed a permanent increase in length equal to approximately $\frac{3}{8}$-in. for a 100-ft. slab. This value is approximately six times greater than the value computed from the data of the reinforced short sections of the Indiana pavement. Whether the reinforcing steel in the Indiana sections restrained the tendency for the concrete to grow in the presence of moisture or whether differences in material and exposure between the sections of the two investigations were responsible for this difference in behavior can only be a matter of speculation.

Effect of Moisture Estimated—

Although the data in Figure 11 were obtained primarily for a study of length changes of a permanent nature, values of winter to summer length changes resulting from the change in the moisture content of the concrete alone can be obtained from them with considerable accuracy For example, the range in the observed unit length changes of the short sections, as determined from the expansion and contraction periods shown in the figure, is 0 000205 to 0 000240 for the 60-deg F. change in slab temperature Inasmuch as the short sections were structurally intact and were relatively free to move, the length changes are principally those caused by changes in the temperature and moisture content of the concrete. It will be recalled that, earlier in the report, the average value of the thermal coefficient of the concrete was estimated to be approximately 0 0000048 per deg F From this, the unit length change of the sections for a 60-deg. F. change in temperature can be calculated and applied as a correction to the observed unit length change yielding the unit length change caused by the change in the moisture content of the concrete

In this investigation the unit length changes caused by the annual cycle of moisture variations were found to range from 0 000048 to 0 000083, these length changes being opposite in sense and partly compensatory for those caused by the annual cycle of temperature changes. For the Indiana pavement, these values correspond to length changes produced by a 10-deg to 17-deg. F. change in pavement temperature. The values should be approximately a maximum for the yearly cycle, since, as remarked before, the data were obtained at times when the maximum and minimum amounts of moisture were present in the concrete.

Again referring to data obtained from the 40-ft., plain concrete test section of the Arlington Experiment[2], it was found that seasonal variations in the moisture content of that concrete caused length changes corresponding to a 20-deg to 40-deg F. change in slab temperature. Hence, it is evident that the effect of moisture on length changes was less for the reinforced sections in Indiana than for the plain concrete section of the Arlington study In this comparison, also, it seems quite possible that the reinforcing steel restrained, to some extent, the tendency of the concrete to change in length with moisture change

As a matter of interest, in tests conducted

[2] L. W Teller and E. C Sutherland, "The Structural Design of Concrete Pavements—Part 2," *Public Roads*, Vol 16, No 9, November 1935

E F Kelley, "The Application of the Results of Research to the Structural Design of Concrete Pavements," *Public Roads*, Vol 20, No. 6, August 1939

by the Minnesota Department of Highways[3] seasonal moisture variations caused length changes corresponding to slab temperatures that averaged 20 deg. F. Also, it was determined in tests by the Michigan State Highway Department[4] that, for a constant temperature of 72 deg. F., the average unit change in length of plain concrete specimens from an oven dry to a saturated state was 0.000246, this value being equivalent to a change in temperature of 46 deg. F It appears that different concretes may vary considerably in this characteristic.

The progressive or permanent changes in the length of several of the longer sections are given in Figure 12 These data were obtained at the same temperatures and on the same days as those of the short sections discussed previously. The lengths of the individual bars indicate overall changes in section length that accompanied a 60-deg. F. winter to summer rise in pavement temperature. The solid line drawn through the mid-points of the individual bars defines the progressive changes in the lengths of the sections.

It is apparent from this figure that: (1) the lengths of all sections increase progressively with time; and (2) the magnitude of these progressive increases becomes greater with increase in section length for sections up to approximately 1,000 ft. long.

The progressive increases of the long sections are the result not only of the tendency of concrete to grow when exposed to cycles of moisture and temperature change, as was the case of the short, uncracked sections; but also of the tendency of transverse cracks to open, however slightly, with time and of the influence of subgrade resistance. The fact that the long sections returned so nearly to their original or base lengths during the early cycles of length change indicates that the initial widths of the many transverse cracks that developed during this period must have been extremely small. Also, when the magnitude of the total increase in section length or growth of these sections is divided by the number of cracks in the section it is evident that the steel reinforcement has prevented any appreciable opening of the individual cracks

[3, 4] Investigational Concrete Pavements, Highway Research Board, Research Reports No 3B (1945)

PART 3.—DEVELOPMENT, DISTRIBUTION AND PRESENT CONDITION OF CRACKS IN THE SECTIONS

Five crack surveys were made over the full length of the experimental pavement during the 10-yr period. The first was made shortly after the sections were placed, with others at the end of the first, third, fifth and tenth years of service. In addition, during the first 3 years of the life of the pavement, certain representative sections were surveyed at more frequent intervals. In every case the surface of the pavement was subjected to a very careful examination in order that all fractures visible to the naked eye might be detected.

Figure 13, traced from the crack survey sheets, shows the number and position of the cracks that have developed in typical sections during the 10-yr. period of service. Considerable care was exercised in accurately plotting each crack on the original survey sheets. Because of the fine character of the cracks, it was necessary to outline each crack on the surface of the pavement before plotting on the sheets.

It will be noted from the examples in Figure 13 that short sections tend to be comparatively free of fractures. At the end of 10 yr., 70 percent of 154 short sections, that is, those whose lengths range from 20 to 120 ft, were still uncracked As the section lengths increase, however, cracking becomes more prevalent until in the central portions of long sections the crack interval is frequently less than 2 ft.

It may be observed also from the crack patterns shown by the survey sheets that: (1) cracks, although somewhat wavy and irregular, are essentially at right angles to the axis of the pavement; (2) cracks in many instances are not continuous across both lanes, either ending completely or being offset slightly at the center joint; (3) longitudinal cracking has not developed in any part of the pavement; and (4) corner breaks at transverse cracks are very rare.

After 10 yr. of service the surface condition of the pavement is excellent. With the excep-

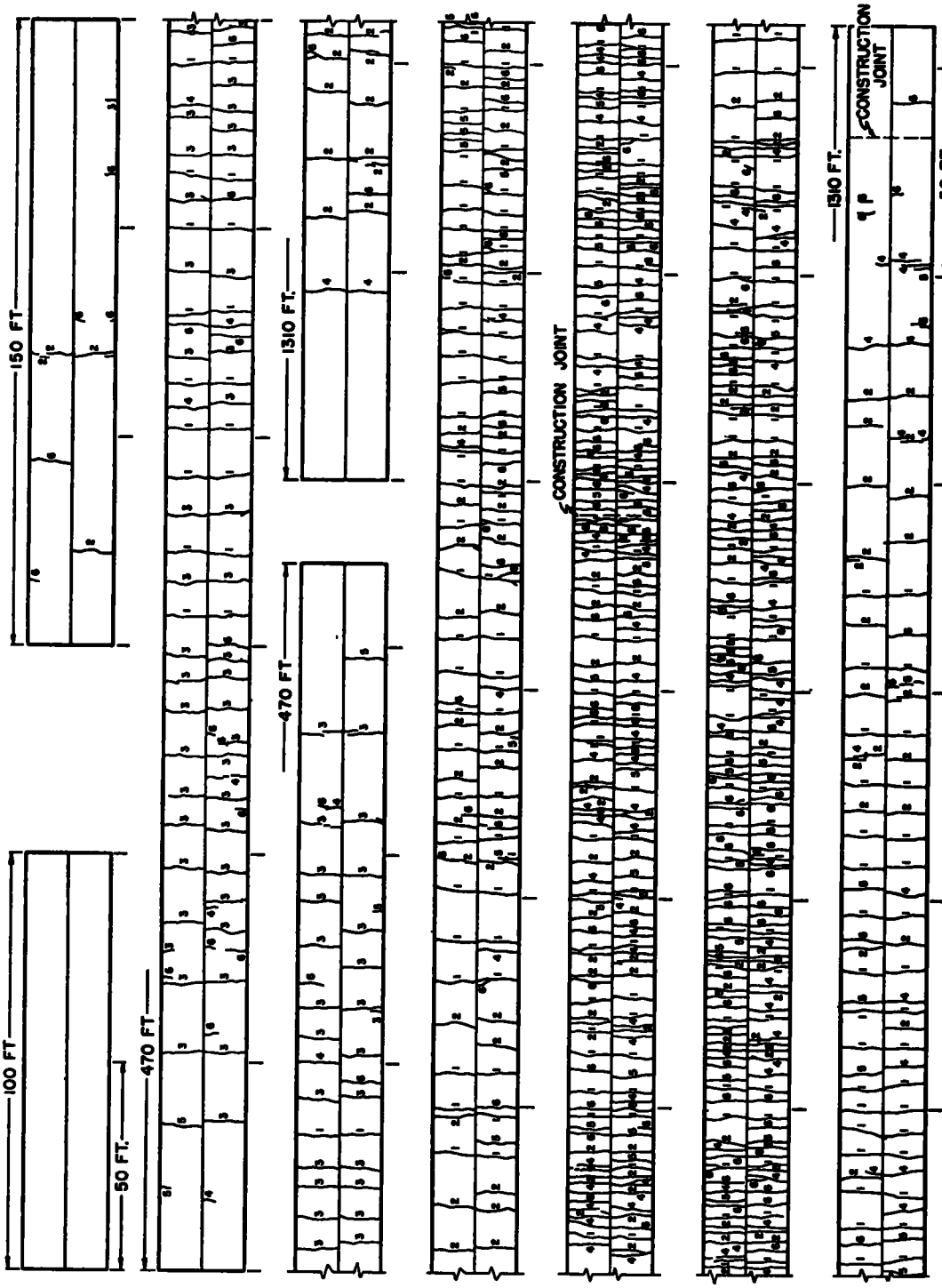


Figure 13. Typical Formation of Cracks During the First 10 Years of Pavement Life — Sections Placed September–October, 1938
Survey: 1—Nov., 1938 2—Nov., 1939 3—Nov., 1940 4—Oct., 1941 5—Oct., 1943 6—Oct., 1948

tion of those in sections reinforced with the 32-lb. wire fabric, all fractures have been held closed by the longitudinal steel. The cracks that formed in the sections containing this light fabric were wider initially than those that appeared in the more heavily reinforced sections and, after about 8 yr., in several cases the steel crossing them broke, probably from shearing forces, resulting in relatively wide openings and some spalling. In all of the other sections there is no evidence of any form of structural damage to the concrete, with the exception of a very slight raveling of the edges of the cracks, probably due to flexure. It is believed that the fineness of the cracks, especially those in the more heavily reinforced sections, is conducive to distributed interfacial pressure, thus minimizing the possibility of blow-ups and other pressure concentration failures that are sometimes observed at cracks in plain concrete pavements.

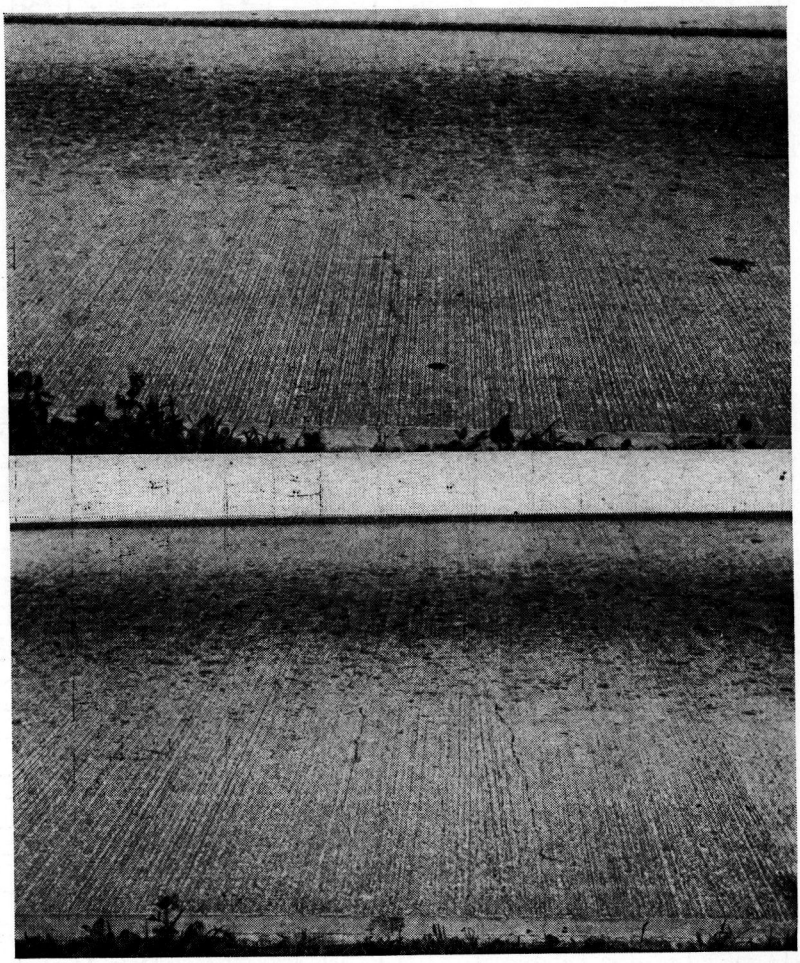


Figure 14. Surface Condition of Pavement in Vicinity of the Widest Cracks Observed in the Central Portion of the Heavily Reinforced Sections. Heavily Traveled Lane After 7 Years of Service

Surface Condition of Pavement at Cracks Studied—

Figures 14 and 15 show typical examples of the surface condition of the pavement at several cracks.

Figure 14 pictures the surface condition of the pavement in the vicinity of several of the widest cracks to be found in the central regions of the heavily reinforced sections. These cracks are in the right-hand or heavily traveled lane of the pavement and were taken after a 7-yr. service period. Unfortunately since the photographs were taken, many of these cracks were inadvertently covered with bituminous material by a maintenance crew. This material did not enter the cracks but did spread over the surface and obscure them.

In Figure 15 are shown close-up photographs of the surface condition of the pavement at two cracks; one in the central portion of the longest section of those reinforced with 1-in. diameter rail steel bars (1.82 percent of longitudinal steel) and the other in a comparable portion of the longest section of those containing ½-in. diameter rail steel bars (0.45 percent steel). Both photographs were taken after the pavement had been in service for 10 yr. and show fractures typical of those that appeared in the heavily traveled lane shortly after construction.

The contrast between the surface widths of the two cracks pictured in Figure 15 is obvious. When cracks such as these first formed, those that appeared in the most heavily reinforced sections were almost microscopic, being discernible only by extremely close inspection. However, as the percentage of reinforcing steel decreased the cracks were, in general, less frequent and more readily seen; those of the lightly reinforced short sections, if present at all, being relatively conspicuous. Over the 10-yr. period of service the action of traffic and exposure have produced some slight raveling and rounding of the edges of the fractures until their surface appearance is as shown in the photographs.

The preceding discussion related to differences between the surface widths of fractures in sections containing different percentages of longitudinal reinforcement. During the surveys, it was observed further that: (1) cracks in the end portion of a given long section generally presented a slightly better surface appearance than those in the central part; and (2) cracks in the central portion of sections containing a given percentage of steel, but of different lengths, showed some slight evidence of a corresponding difference in surface widths, those in the central part of the longest section of each group apparently being wider than those in the central part of the shortest section. It seems reasonable that this should be so.

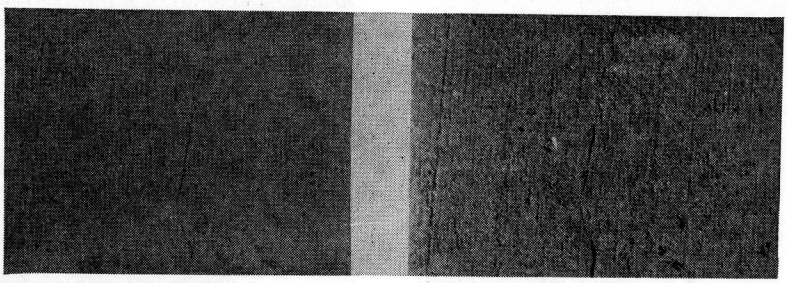


Figure 15. Surface Condition of Cracks Typical of Those That Developed at an Early Age in the Central Portion of the Longest Section Reinforced With: A, 1-inch Diameter Bars (1.82 Percent Steel); and B, ½-inch Diameter Bars (0.45 Percent Steel). Heavily Traveled Lane after 10 Years of Service

At the end of 10 yr., quantitative measurements of the surface widths of cracks, which included raveling and rounding of their edges, were made in the following manner: Starting at the edge of the pavement the width of a segment of crack about 3 ft. long was carefully examined and a width measurement made at a point judged to be average. A similar measurement was made on each of two additional 3-ft. segments of the same crack, thus covering one lane width. The average of the three measurements was considered to be the average surface width of the crack for the particular lane. All measurements were estimated to the nearest 0.01 in. It is realized that this procedure does not establish an exact value for the surface width of an individual crack, but it is believed that the average of a number

of such measured values has significance in relative comparisons.

Measured values of the surface widths of cracks obtained in the manner described are given in Table 6 The values shown, for each percentage of steel, are of fractures that developed at an early age in the central area of the longest section reinforced with either rail or billet steel bars Hence, the computed maximum steel stress was either 45,000 or 55,000 lb per sq. in An average width value represents the combined average of 15 to 20 cracks, measured in sections containing both rail and billet steel bars All data were obtained in the fall of the year when the mean pavement temperature was 58–60 deg F

The comparisons available in the table show that the surface widths of the cracks

TABLE 6
THE SURFACE WIDTH OF CRACKS
(Central Portion of Longest Section for Each Percentage of Steel)

Percentage of Longitudinal Steel[1]	Surface Width of Cracks in Lane Carrying			
	Heavy Traffic		Light Traffic	
	Average	Range	Average	Range
Percent	*in*	*in*	*in*	*in*
1 82	053	02– 11	020	01– 03
1 02	078	03– 15	032	02– 05
45	104	07– 18	038	02– 06
26	117	09– 15	038	02–.07

[1] Calculated maximum stress in steel is either 45,000 or 55,000 lb per sq in

tend to increase with a decrease in the amount of longitudinal reinforcement For example, the average measured width of the cracks in the heavily traveled lane of the selected sections reinforced with 0 45 percent steel is approximately twice the average width of those in sections containing 1 82 percent of steel. The influence of traffic on the surface width of the fractures is also evident, but this effect will be discussed later in the report.

As previously mentioned the values of Table 6 are for sections containing rail and billet steel bars Measurements of the surface widths of cracks also included fractures in the longest section of each group reinforced with the 91- and the 149-lb welded wire fabric The data from these measurements are concordant with those from the sections reinforced with rail and billet steel bars The average surface width of the cracks in the section containing the 91-lb fabric was found to be appreciably greater than that of the section reinforced with the 149-lb. fabric.

Referring to the range in the average surface width of individual cracks (Table 6), it is apparent that the maximum is, in some cases in the heavily traveled lane, slightly more than ⅛-in Also, the width of a crack at isolated points along its length was often observed to be considerably greater than its average width, because of localized raveling The maximum values at such points were 0 3 and 0 7 in , respectively, for sections reinforced with 1 82 and 0 45 percent of steel It should be kept in mind, however, that the depth of raveling along the lengths of all cracks was estimated to be never more than ⅛-in and may be considered superficial

A limited amount of supplementary data on the surface widths of fractures, other than those given in Table 6, were obtained by measurements in the end and central areas of the 1,310-ft. section, in order to establish a comparison of crack widths in those regions. It was found that the surface widths of cracks in the central portion of the 1,310-ft section averaged about twice the width of those near the ends Also, measurements were made of the surface width of cracks that had developed in the central part of the 600-ft. section reinforced with 1 82 percent of steel and such widths averaged about ½ the width of those that formed in the central part of the 1,310-ft. section containing the same percentage of reinforcement

Real Widths of Cracks Determined—

Figure 16 shows close-up photographs of several cracks as observed at the vertical face at the edge of the pavement. These photographs, taken in 1948, show fractures that occurred at an early age in the central area of the longest section of each group reinforced with 1-in diameter rail steel bars, ½-in diameter rail steel bars and a 91-lb wire fabric, the percentage of reinforcement values being 1.82, 0 45 and 0 24 respectively

Cracks, such as those pictured in Figure 16, were almost imperceptible when they first appeared, being visible throughout the depth of the slab only after a drying period following a wetting of the concrete With time, however, they have opened progressively a very

small amount and their edges have raveled slightly.

At the end of 10-yr. of service, measurements were made of the edge-face widths of a number of cracks (located in the central portions of the sections mentioned above) in order to obtain values of the real widths of the cracks themselves; width values which, unlike those taken on the surface of the pavement, did not include raveling and rounding of the crack edges. A 40x shop microscope with a 0.001-in. graduated scale was used to make the measurements, the instrument being focused into the opening of the fracture to eliminate errors caused by surface conditions at the crack edges.

the average width of those in the same lane of the section containing 1.82 percent of steel. Comparison of the data in Tables 6 and 7 shows that the surface width of cracks increases under the same conditions that cause an increase in real width. It is apparent, also, that the surface width of a given crack is many times greater than its real width.

Longitudinal reinforcement is in continuous bond with the concrete until the first transverse crack develops. When this happens the amount of opening of the crack will depend upon the total elongation of the steel which crosses it and this elongation is, in turn, dependent upon: (1) the length that is free to elongate as affected by the bond between the steel and the concrete, and (2) the magnitude of the direct tensile stress in the steel, also dependent upon bond conditions.

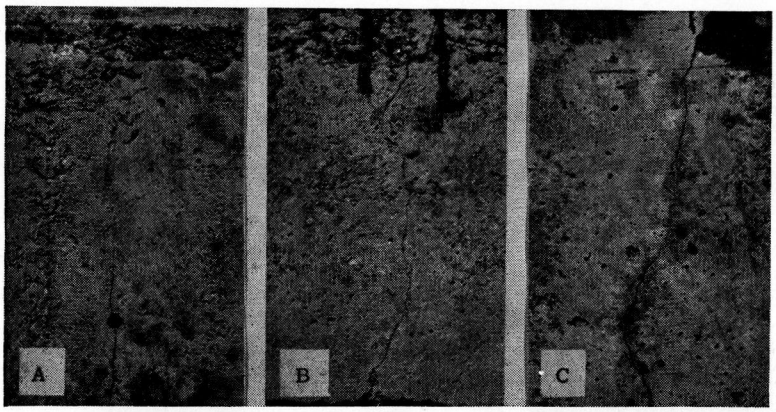


Figure 16. Edge (Vertical Face) Condition of Cracks Typical of Those That Developed at an Early Age in the Central Portion of the Longest Section Reinforced With: A, 1-inch Diameter Bars (1.82 Percent Steel); B, ½-inch Diameter Bars (.45 Percent Steel); and C, 91-pound Wire Fabric (.24 Percent Steel). Heavily Traveled Lane After 10 Years of Service

The data obtained from this study of crack widths in the slab edges are given in Table 7. Each average value of the table is the average for five cracks that developed early in the life of the pavement. The computed maximum steel stress at the site of these cracks is 55,000 lb. per sq. in. All measurements were made at the mid-depth of the slab and in the fall of the year when the mean pavement temperature was 73–74 deg. F.

The values shown indicate that the real widths of the cracks, like their surface widths, increase with a decrease in the percentage of longitudinal reinforcement. For example, the average width of the fractures in the heavily traveled lane of the selected section reinforced with 0.45 percent of steel is nearly three times

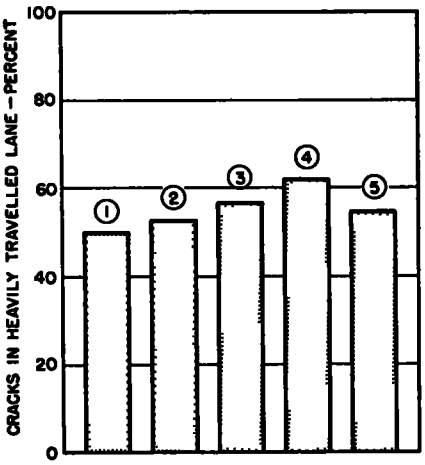
TABLE 7
THE REAL WIDTH OF CRACKS
(Central Portion of Longest Section for Each Percentage of Steel)

Percentage of Longitudinal Steel[1]	Width of Cracks in Lane Carrying:			
	Heavy Traffic		Light Traffic	
	Average	Range	Average	Range
Percent	*in.*	*in.*	*in.*	*in.*
1.82	.004	.002–.007	.002	.001–.003
.45	.011	.007–.018	.009	.007–.010
.24	.013	.005–.018	.010	.006–.013

[1] Calculated maximum stress in steel is 55,000 lb. per sq. in.

In this investigation neither the length over which the steel was not in bond nor the magnitude of the tensile stress in the steel could be determined.

However, it is of interest to examine the crack-width data on the basis of the amount of longitudinal steel present as shown in Table 7. Presumably, at the time of the crack-width measurements the same steel stress was active in the central region of all sections listed in the table. When compared in this way it will be found that for both the heavily



(1) SEPT – OCT., 1938 (SECTIONS PLACED) TO NOV., 1938
(2) NOVEMBER, 1938 TO NOVEMBER, 1939
(3) NOVEMBER, 1939 TO NOVEMBER, 1941
(4) NOVEMBER, 1941 TO OCTOBER, 1943
(5) OCTOBER, 1943 TO OCTOBER, 1948

Figure 17. Effect of Traffic on the Amount of Cracking

traveled and the passing lanes, the average crack width increases directly with a decrease in the percentage of longitudinal steel.

Also of interest is the fact that in the longer sections the surface widths of cracks, and presumably their real widths also, were less in the end than in the central areas of the sections. This is as would be anticipated, since the tensile stress in the longitudinal bars would be expected to decrease as the end of a section is approached.

Effect of Traffic Observed—

In connection with the study of cracking, an opportunity has been afforded to observe the effect of traffic on the development and condition of the cracks. It will be recalled that the experimental 2-lane pavement is one half of a divided highway, consequently, the right-hand lane carries the greater number of vehicles and practically all of the heavy trucks, the left-hand lane being used largely for passing. Also, it is mentioned again that the experimental sections are a part of the transcontinental highway U. S. 40 and are, therefore, subjected to a relatively high frequency of heavy traffic loads.

Although a survey made soon after completion of the pavement showed equal cracking in both lanes, at the end of the first year 51.2 percent of the total number of cracks were found to be in the right-hand lane of the pavement. This percentage value had increased to 52 7 and 53.0 percent at the end of

TABLE 8

Percentage of Longitudinal Steel	Ratio of Right Lane to Left Lane
Surface Width	
0 26	3 1 1
0 45	2 7 1
1 02	2 4 1
1 82	2.7 1
Real Width	
0 24	1 3 1
0 45	1.2 1
1 82	2 0 1

the fifth and tenth years respectively. Thus, it appears that repetition of traffic loads has exerted a slight but only a slight influence on the development of transverse cracks. Since approximately two-thirds of the total or present number of cracks formed during the first year, when only 51.2 percent formed in the right-hand lane, the effect of traffic repetition on subsequent cracking is somewhat more pronounced than is indicated by the preceding percentage values

Figure 17 was prepared to show the influence of traffic repetition on cracking during specific periods of the life of the pavement. For the periods indicated by the circled numbers, each individual bar represents the number of cracks that formed in the heavily traveled, right-hand lane expressed as a percentage of those that formed in both lanes of

the pavement. It will be noted that an equal number of cracks appeared in both lanes of the pavement within the first month or two after construction. During each subsequent period a progressively greater number of cracks formed in the heavily traveled lane than in the passing lane until a maximum value of 62 percent was reached for the period covering the third to fifth years of pavement life. During the last 5 yr. about 55 percent of all new cracks developed in the right-hand lane of the pavement.

Also, as will be seen by an examination of the data in Tables 6 and 7, traffic has had an appreciable effect on both the surface and the real widths of the transverse cracks.

Comparisons of the average widths of cracks in the right-hand lane with those of companion cracks in the passing lane are given in Table 8

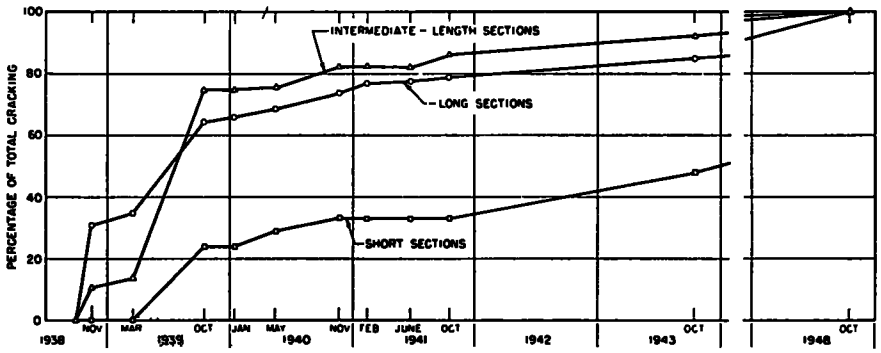


Figure 18. Rate of Crack Development During the First 10 Years of Pavement Life

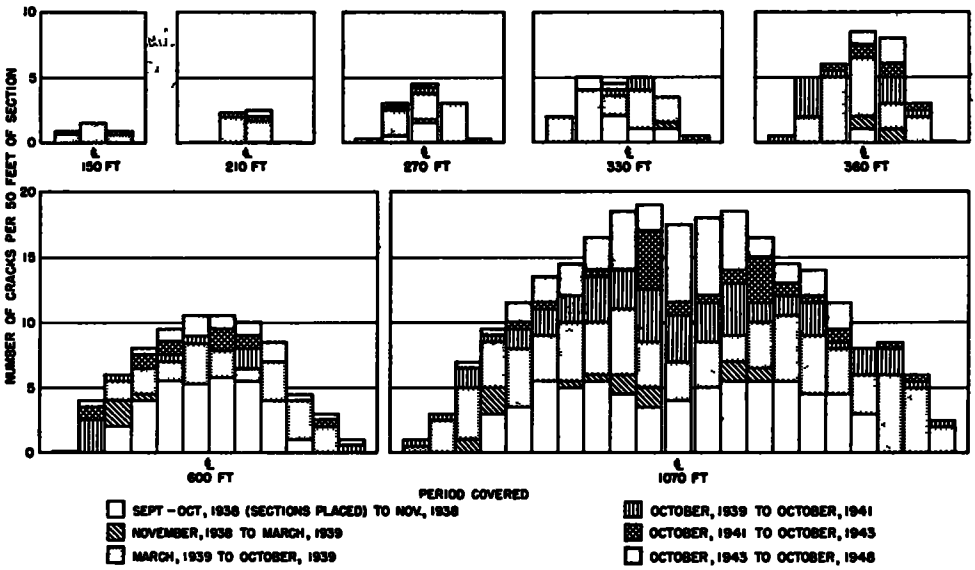


Figure 19. Distribution of Cumulative Cracking Per 50 Feet of Section Length—First 10 Years of Pavement Life

It is apparent that the heavier traffic using the right-hand lane has produced more extensive raveling and other superficial damage at the crack edges and a wider separation of the fractured faces than the lighter traffic on the left-hand lane. This effect of traffic is naturally more pronounced in the case of the surface widths of the cracks.

Rate and Distribution of Cracking—

Figure 18 shows the manner in which cracking has developed with respect to time In this figure the sections were grouped according to length, as follows: (1) short, 20–120 ft , (2) intermediate-length, 120–470 ft , and (3) long, 470–1,310 ft.

Thirty-one percent of the total or present number of cracks in the long sections and 11 percent of those in the intermediate-length sections appeared within approximately one month after construction Few cracks occurred during the first winter, none in the short sections. However, the rate of cracking was quite high for all groups during the period that followed, which included the interval between late March and late October of the first year.

The survey made at the end of this first year of service showed 65 percent of the present cracking had developed in the long sections, 75 percent in the intermediate length sections and 25 percent in the short sections.

On the basis of all transverse cracks that have developed during the 10-yr. period of service, it is of interest that 67 percent had appeared by the end of the first year. After the first year the rate has been quite low and, in general, rather uniform. Between length groups the highest rate has been in the short sections.

In Figure 19 is shown the distribution of cracking for representative sections expressed as the number of cracks per 50 ft. of section. The data indicate that: (1) the number of cracks per 50-ft increment increases from a minimum value at the end of a section to a maximum value in the central area in a generally normal frequency distribution pattern, and (2) the maximum values, as found in the central area of the sections, increase progressively with increase in section length.

It will be noted that the symmetry of cracking in the experimental sections is not only indicative of structural uniformity, but also implies that the nonuniformity of the elevation changes that developed in the pavement, as mentioned earlier, apparently had little effect on the formation of cracks.

The manner in which cracking developed in the sections is, in some respects, shown to better advantage in Figure 19 than in Figure 18, especially since the distribution of cracking for the various time periods is given in the latter figure.

The magnitude and distribution of the cracking that appeared within approximately one month after construction of the sections is shown as the first period During this period no cracks were found in sections having lengths of 210 ft. or less and only a limited number in the central portion of sections with lengths between 270 and 360 ft , but a considerable number were found in the 600- and 1,070-ft. sections at some distance from the ends. Since the cracking during this period appeared only in the central areas of the longer sections, it is believed that it had its origin primarily in the tensile stresses induced by subgrade resistance during shrinkage of the sections either from loss of moisture, decrease in pavement temperature, or both.

The second period covers the first winter after construction The survey at the end of the winter indicated that sections having lengths of 210 ft. or less were still uncracked and only a small amount of cracking, spottily distributed, had developed in sections having lengths equal to or greater than 270 ft. The relative absence of crack development during this period indicates that: (1) the nonuniform changes in pavement elevation caused by frost penetration of the subgrade had little influence upon cracking; and (2) tensile stresses from subgrade resistance were no greater during the winter period than during the preceding fall. This supports the conclusion drawn from the data of Figure 9

In the third period, between late March and late October of the first year, a noticeable change occurred in the crack development in all of the sections. In fact, a large percentage of the cracks now present in sections having lengths of 360 ft. or less and in the end areas of the longer sections formed sometime during this period. Such cracking is believed to have been caused primarily by stresses induced by restrained warping. Unfortunately, the pavement was not surveyed in mid-summer so it is not possible to determine more closely the part of this period during which cracks formed. However, it is suspected that the cracks developed in large part during late spring and early summer when warping stresses are generally highest for the year. The fractures that formed at some distance from the ends of the longer sections may have resulted, also, from stress combinations existent in early fall when the sections were con-

tracting after having attained their maximum annual unrestrained lengths.

During the fourth period, which covers the second and third years after construction, the development of new cracks was confined primarily to the central areas of the long sections. The relatively small number of fractures that appeared during this period and during the succeeding fifth period (fourth and fifth years) greatly reduced the rate of crack development. Within the sixth and last period of this study, from the fifth through the tenth years of pavement service, the greatest number of cracks again have formed in the central areas of the long sections, suggesting a continued high stress condition in those regions.

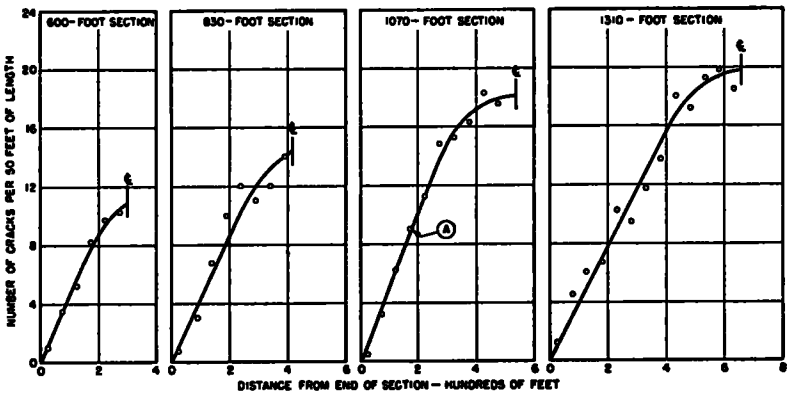


Figure 20. Frequency Distribution of Cracking at the End of 10 Years for Sections Reinforced with 1-inch Diameter Rail Steel Bars (1.82 Percent of Steel)—Average of Both Lanes

Crack Frequency Patterns

Frequency distribution curves for the cracking that existed at the end of 10 yr. in the four sections comprising the group reinforced with 1.82 percent of longitudinal steel (1-in. rail steel bars) are shown in Figure 20.

The ordinate values represent the number of cracks per 50-ft of section and the corresponding abscissas are distances from the end of the section to the centers of the 50-ft. lengths to which the ordinate values apply. For example, at point A in the figure there are nine cracks in the 50-ft. length which lies between 150 and 200 ft. from the end of the 1,070-ft. section.

It is apparent that: (1) for some distance, beginning at the end of a section, the crack frequency for successive 50-ft. increments increases directly with increase in distance; (2) the length, over which the linear relation holds, increases progressively with increase in section length; and (3) the slopes of the linear portions of the curves appear to be nearly the same for the different section lengths. It is believed that the frequency of cracking in the sections reflects, to a considerable extent, the stress distribution in the longitudinal steel as induced by subgrade resistance.

Frequency distribution curves were constructed for all sections having lengths equal to or greater than 150 ft. From these curves the maximum cracking frequency value (number of cracks per 50 ft. of section in the central area) was determined for each section. Figure 21 shows, for each of the three types of reinforcement, the relation established by plotting such frequency values against the corresponding section lengths. In order to show possible effects of the stresses in the steel, the maximum computed steel stresses are indicated by the symbol used.

It is apparent that the maximum cracking frequency increases with an increase in section length, the relation being nearly linear for section lengths between 400 and 1,000 ft For section lengths greater than about 1,000-ft. the curves depart from linearity indicating that a condition of complete restraint is being approached. This suggests that sections having lengths somewhat greater than 1,000 ft., possibly the 1,700–1,800-ft. length mentioned in the discussion of Figure 9, would develop

complete restraint in the central region and that sections of this length or greater would have equal maximum cracking frequencies irrespective of their overall lengths.

The 10-yr. data indicate that an interval between cracks in this region of complete

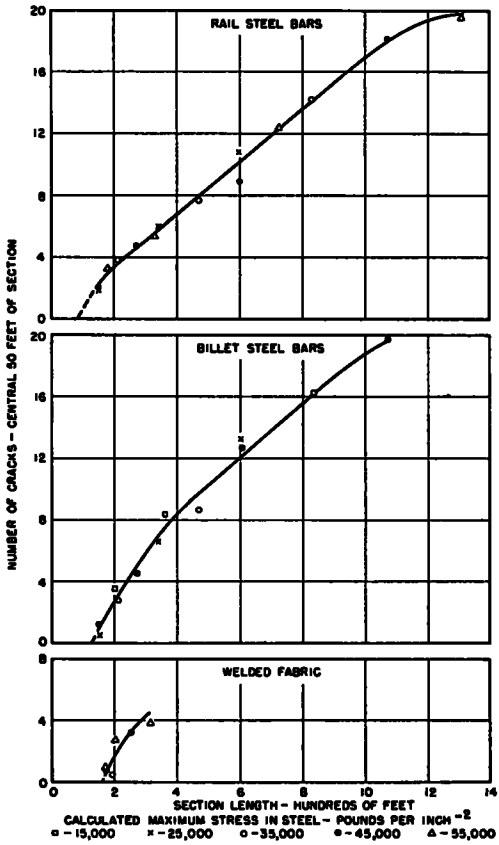


Figure 21. Effect of (a), Type of Reinforcement and (b), Calculated Maximum Steel Stress on the Relation Between Section Length and Maximum Cracking Frequency—Age of 10 Years

restraint might be expected to be approximately 2 0 to 2 5 ft.

The data shown in Figure 21 indicate that the type of reinforcement has only a slight effect on maximum cracking frequency For example, within the length range of sections containing welded wire fabric, the maximum cracking frequency values are only slightly less than those for sections of comparable length reinforced with billet or rail steel bars. Comparing the bar reinforced sections, it appears that the maximum cracking frequency values are slightly greater for billet than for rail steel bars. (At 1,000 ft. these values are 18 8 and 17.0 respectively).

On the other hand, it will be noted that all symbols denoting the various magnitudes of computed steel stresses fall very close to the mean curves, indicating that, within the range of steel percentages in sections of common

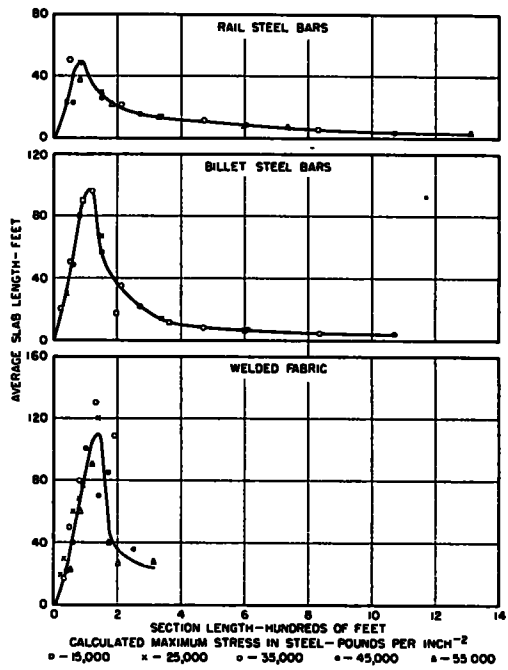


Figure 22. Effect of (a), Type of Reinforcement and (b), Calculated Maximum Steel Stress on the Relation Between Section Length and Average Slab Length—Age of 10 Years

length, a variation in the amount of steel is not accompanied by a corresponding variation in maximum cracking frequency.

Because of conservative design assumptions, the relation between amounts of reinforcing steel and the section lengths are such that the steel has, in all probability, never been stressed beyond its elastic limit. Therefore, if reinforcement which is adequate for a given section length, of say 400 ft., had been used for the entire range of section lengths, the relationships shown in Figure 21 would not

obtain and differences due to variations in the maximum steel stresses might have appeared. The longer sections would probably either subdivide due to breakage of the steel or contain fewer but wider cracks as a result of inelastic deformation of the steel.

Another analysis of the data that is of interest is shown in Figure 22 in which the average slab length, after 10 yr. of service, is plotted against section length as constructed. In this figure separate curves are given for each of the three types of reinforcement, and the maximum steel stresses as computed during the designing of the sections are indicated by the character of the symbols used Slab length is defined as the distance between transverse cracks or joints, all joints being considered as cracks Each point defining the curves is an average value of either 2, 4 or 6 sections.

Although the points defining the curves appear to be somewhat erratic for sections up to approximately 200 ft. long, it is believed that this is a statistical effect caused by the relatively small number of cracks in sections of these lesser lengths.

It is apparent that the three curves of Figure 22 follow the same general pattern; that is, the average slab length increases with an increase in section length until a peak value is reached, beyond which there is a rapid decrease in average slab length that becomes more gradual and finally approaches a constant value for the longer sections. In the case of the 1,310-ft section the present value of the average slab length is 4 2 ft. At the end of the first and fifth years this value was 7 0 and 5.1 ft respectively

The type of reinforcement has an obvious effect on the relation between section length and average slab length, especially in the case of the shorter sections. The greatest average slab length for the sections containing welded fabric is 109 ft. which was reached at an optimum section length of 135 ft. In the case of the billet steel bars the value is 97 ft. attained at an optimum section length of 115 ft ; but for the rail steel bars this value is only 48 ft the corresponding section length being 90 ft.

The reason or reasons for the differences in the peak slab-length values and corresponding optimum section lengths for the three types of reinforcement cannot be fully explained. There are, however, two conditions that may have had some influence on the data First, in that part of the curves which pertains to the shorter sections, there are fewer points defining the curve in the case of the rail steel bars than in the case of the other types of reinforcement. Second, in all sections reinforced with welded fabric and all sections, except one, with lengths of 270 ft or less reinforced with billet steel bars, the coarse aggregate used in the concrete consisted of a mixture of small-size gravel with large-size crushed limestone; whereas in sections reinforced with rail steel bars the coarse aggregate used in the concrete was entirely the crushed limestone. However, there is no other evidence that the difference in coarse aggregate mentioned affected in any way the behavior or present condition of the sections.

The data of Figure 22 mean that, under the conditions obtaining in this experimental pavement, the longest average slab lengths are found at the so-called "optimum" section lengths. Hence, if one were interested only in a minimum number of transverse cracks and joints these data suggest that in reinforced concrete pavements the transverse joints be spaced approximately 100 ft apart. However, as this investigation strikingly shows, a longer section with many transverse cracks can continue to be a strong, durable structural unit after many years of heavy traffic service if it contains an adequate amount of longitudinal reinforcement.

In the relations shown in Figure 22, the maximum computed steel stress values apparently have no influence on the amount of cracking in a given section length. This is concordant with the relations shown in Figure 21.

PART 4.—BEHAVIOR OF THE 500-FT. SPECIAL SECTIONS

The four special 500-ft. sections containing weakened-plane warping joints at 10-ft. intervals have been subjected to the same close study as have the regular sections.

It will be recalled that in each of the four special sections, relatively light welded-fabric reinforcement was placed continuously through all of the weakened-plane warping joints over the 500-ft section length. The bond between the steel and the concrete was

destroyed purposely for a distance of 18 in. on each side of each joint by omitting two transverse wires, one on either side of the joint and by greasing the longitudinal wires over the 36-in. length. In addition to the continuous reinforcement, shear bars consisting of ¾-in. diameter dowels, 18 in long and spaced 12 in. center to center were placed across the warping joints in one-half of each of the four sections.

The distinguishing features of the four 500-ft. special sections are as follows.

No 1 Weakened-plane joints are of the submerged type and the reinforcement weighs 91 lb. per square.

No. 2 Same as section No 1, except that the reinforcement weighs 45 lb. per square

No. 3. Weakened-plane joints are of the surface groove type and the reinforcement weighs 91 lb. per square.

No. 4. Same as section No. 3, except that the reinforcement weighs 45 lb. per square.

Through the design features of these special sections it was proposed to develop information on the practicability of a pavement design in which transverse crack control was obtained by means of relatively short slab units (10 ft.) with pavement continuity obtained by the use of continuous longitudinal reinforcement. Other information sought pertained to: (1) the amount of longitudinal steel necessary to resist the tensile forces created by subgrade resistance in a section of this length; (2) the value of the design feature in which bond was deliberately destroyed for 18 in. on either side of the joint; and (3) the necessity for protection of the longitudinal reinforcement against shear in the transverse joints by means of dowels used to develop shear resistance. It was thought that such protection probably would be necessary because of the relatively large joint opening expected from elastic elongation of the longitudinal steel over the 36 in. of unbonded length at the transverse joints

During a drop in pavement temperature, a continuously reinforced section naturally attempts to contract about the center of the section length. At the same time the individual segments or slab units of the section are attempting to contract about their individual centers. The amount that these individual segments contract should equal the elongation of the steel crossing the fractures that define their lengths. This elongation of the steel is dependent upon: (1) the magnitude of the stress induced in it by resistance as the segments tend to move over the subgrade; and (2) the length over which the bond between the steel and concrete is destroyed that is, the length over which this stress is effective.

Thus, by subdividing the 500-ft. special sections into 10-ft. slab lengths so that during a large temperature drop the contractive length change of an individual slab unit would be relatively small; and by breaking the bond for 36 in at each separation between slabs so that the elongation of the steel could be relatively great without exceeding the elastic limit, it seemed that a certain degree of control over the movements of a section should be gained without rupturing the reinforcing steel. For example, during a sudden drop in pavement temperature when subgrade resistance is relatively great, the continuous reinforcement would simulate a steel spring at each transverse joint, elongating and permitting the slab units to contract about their individual centers, and subsequently contracting and drawing the units together as the subgrade resistance decreased.

From the standpoint of design, all of the special sections behaved satisfactorily during the first 3 yr. of pavement service. Then the reinforcing steel began to fail at the joints During the condition survey at the end of 3 yr. two breaks in the reinforcing steel were discovered, both at joints in the sections containing the 45-lb wire fabric and both at joints without shear bars, one of the breaks being only 60 ft. from the end of a section After 5½ yr. the reinforcement was either found to be broken or suspected of being broken at seven of the joints All of these failures developed at joints without shear bars and in the sections reinforced with the 45-lb. wire fabric. At the end of 10 yr. the reinforcing steel was either broken or elongated beyond its elastic limit at 18 of the joints.

The distribution of these steel failures is given in Table 9

From Table 9 it will be noted that: (1) 16 were at joints without shear bars; (2) 15 developed in sections reinforced with the 45-lb. wire fabric, (3) 7 and 11, respectively, were found in sections constructed with the submerged and the surface-groove type of joints;

and (4) none occurred in the halves of the sections provided with shear bars at the joints and containing the 91-lb. wire fabric. The effect of such failures will be discussed in parts 5 and 6 of this report.

The fact that all of the earlier failures of the reinforcing steel and approximately 90 percent of those now present occurred at joints having no shear bars indicates that shearing forces caused by loads passing over the joints were primarily responsible for the steel failure. However, it is possible for a progressive separation to develop at a joint and eventually overstress the reinforcing steel. Infiltration of solid material would, in time, cause a permanent opening of the joint and dissipate all or part of the elastic elongation of the steel in the 36 in. of unbonded length across the joint. Subsequently, during contraction periods, the steel, if small in amount as in the 500-ft. special sections, might be subjected to direct tensile stresses sufficiently large to cause failures. Such action might account for the two cases of steel failure observed at the joints provided with shear bars. These developed after the pavement had been in service for 8 yr., both occurred at some distance from the ends of sections containing the 45-lb. wire fabric and both were at joints of the surface-groove type

TABLE 9

45-lb. Wire Fabric		91-lb. Wire Fabric	
Surface type joints:		Surface type joints:	
With dowels	0	With dowels	0
Without dowels	9	Without dowels	2
Submerged type joints:		Submerged type joints:	
With dowels	2	With dowels	0
Without dowels	4	Without dowels	1

Surface Condition of Weakened-Plane Joints Described—

Figure 23 shows photographs, taken after 10 yr., of two of the submerged type, weakened-plane joints at which the reinforcing steel was unduly inelastically elongated but not broken insofar as could be determined. These two joints were selected as extreme cases of straight and irregular cracking over the submerged parting strips used in creating this type of joint. The fractures that formed over these strips were, in general, meandering in character and were much wider than those that developed in the regular sections containing comparable percentages of continuously bonded reinforcement. Under the influence of traffic and exposure the edges of the fractures have raveled and spalled to a considerable width, creating a rather unsightly surface condition. This condition developed most rapidly during the first 2 or 3 yr. of service. Subsequently the deterioration in surface condition has been gradual.

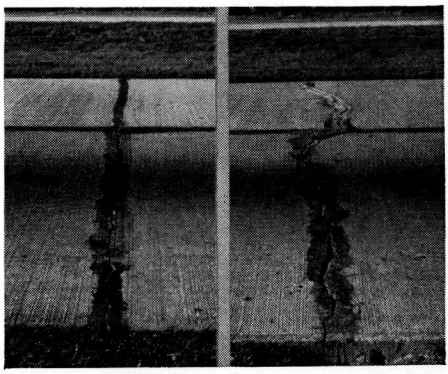


Figure 23. Two of the Submerged-Type, Weakened-Plane Joints After 10 Years of Service. Extreme Cases of Straight and Irregular Cracking over the Bottom Parting Strip

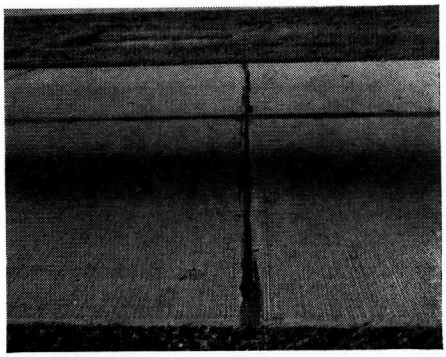


Figure 24. Present Condition of A Surface Type, Weakened-Plane Joint (Typical of Those at Which the Continuous Reinforcement is Structurally Sound)

The condition of the weakened-plane joints of the surface-groove type was excellent, initially, and continued to remain so except where there has been failure of the reinforcing steel. The present appearance of a typical joint is shown in Figure 24. Very little main-

tenance has been required at these joints so long as the steel remained structurally sound since the comparatively small length changes of the 10-ft. units are conducive to well-sealed conditions.

Daily, annual and progressive changes in the widths of the joints of the four special sections were measured during the same periods as those of the regular sections. In Figure 25 are shown the annual and progressive changes in the widths of the joints plotted with respect to base measurements taken during the first winter after construction. The width changes of the expansion joints are, in reality, length changes of the sections as determined by measurement to fixed reference points located at their ends. Attention is called to the differences in the vertical scales used for the width changes of the expansion and weakened-plane warping joints, this being necessary because of the relatively small magnitude of the width changes of the latter. The lengths of the stippled bars indicate changes in width that occurred at the joints during an annual cycle. These changes should be nearly maximum for such a cycle since an attempt was made to obtain the measurements during the hottest and coldest periods of the year. All measurements in a given section were discontinued as soon as the first failure in the reinforcing steel was noted

It is apparent in Figure 25 that, in spite of the continuity of the reinforcing steel throughout the length of a section, the expansion joints closed and the weakened-plane joints opened progressively with time. These progressive changes are in the same sense as those observed in plain concrete pavements built with expansion joints and closely spaced weakened-plane contraction joints It has been observed, also, that the progressive closure of expansion joints at the end of 3 yr. has been. (1) less in sections reinforced with the 91-lb. wire fabric than in those containing the 45-lb. wire fabric, and (2) less in sections with the surface-type joints than in those with the submerged type This latter observation implies that extraneous material infiltrates more readily into the submerged type joints, indicating that joints of this type are more difficult to seal. The behavior of the weakened-plane warping joints is somewhat

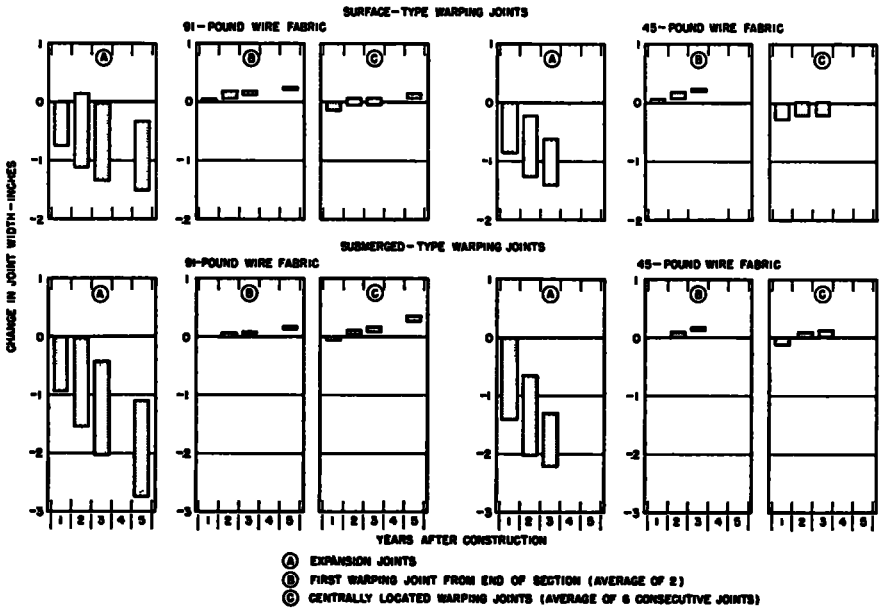


Figure 25. Annual and Progressive Joint Width Changes of the 500-foot Sections Containing Warping Joints at 10-foot Intervals—Values Above the Zero Line Denote Opening, Those Below Closing of the Joint With Respect to the Base Readings of December 1938

erratic and, other than the fact that a progressive opening has developed in all cases, clear-cut trends are not apparent

As remarked before, one of the purposes of the longitudinal steel in these 500-ft special sections was to hold the slab units of the sections together, as much as possible, during contraction periods. In this respect it is clearly shown in Figure 25 that the reinforcing steel, especially the heavier fabric, exercises considerable control over the behavior of the sections during such periods. Measurements of the overall section length changes at the ends of the two sections reinforced with the 91-lb. wire fabric showed 1.09 and 1.13 in., respectively, for a mean pavement temperature drop of 77 deg. F. that occurred between midsummer and midwinter of the second annual contraction period. This is shown in Figure 25 as the difference between the lower end of the second bar and the upper end of the third bar in each of the four graphs marked A. These values represent approximately 77 percent of the annual contractive change of a section of equal length, but containing heavier, continuously bonded reinforcement such as was used in the regular sections. For the same temperature change the length changes of the two 500-ft. special sections reinforced with the 45-lb. wire fabric were 0.67 and 0 71 in., respectively, or about 48 percent of the length changes of the comparable sections of the regular group. A similar comparison of the daily contractive length changes of the special sections with those of the section containing the continuously bonded steel results in percentage values of approximately 54 and 37, for the 91- and 45-lb. wire fabric respectively.

These comparisons show that the pattern of movement of the ends of the 500-ft. special sections, although of less amplitude, is similar to that observed in the regular sections containing continuously bonded steel It is apparent that during periods of contraction: (1) the heavier of the two weights of reinforcement in the special sections was more effective in holding together the individual slab units; and (2) both weights of reinforcement were more effective during annual periods than during daily periods.

It is of considerable interest that the small amount of longitudinal steel in the sections reinforced with the 45-lb. wire fabric was able to cause contraction of the entire 500-ft. section without steel failure By any reasonable assumptions this would indicate that, when such contraction occurred, the coefficient of subgrade resistance was very much lower than the value of 1 5 assumed when the regular sections were designed. It appears also that the coefficient of resistance is smaller for the slow annual changes in length than it is for the more rapid daily changes, this being in agreement with data obtained on the regular sections.

PART 5.—THE OCCURRENCE OF PUMPING

It is generally conceded that three factors are necessary for the development of pumping at joints or cracks in concrete pavements,

TABLE 10
SUBGRADE SOIL DATA

	Silt	Clay	Liquid Limit	Plasticity Index	Moisture Content		
					0–3 Inches Below Surface	3–12 Inches Below Surface	12–24 Inches Below Surface
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Maximum	65	26[a]	52	26	22 6	24 0	27.5
Minimum	20	7	19	4	6 1	8 9	8.1
Average	48	17	33	12	12 8	15.5	17 1

[a] This maximum percentage was exceeded in two instances, however, these cases were not considered as representative of the entire project

namely: (1) frequent repetition of heavy axle loads and accompanying large vertical movenents of slab edges; (2) fine-grained subgrade soils; and (3) free water under the pavement slab. Since the experimental sections were constructed as a part of a heavily traveled route, one of the factors, repetition of heavy axle loads, is always present. Moreover, the pavement was placed on a natural soil having characteristics given in Table 10. The soil analysis was based on samples taken from the finished subgrade at intervals of approximately 500 ft Referring to the average values of the table, it is observed that the combined amount of silt and clay was 65 percent of the total, indicating a fine-grained subgrade soil that would be considered conducive to pumping.

Early in the life of the pavement pumping

began to appear at some of the bridge-type joints which were used to create the wider separations between the longer and consequently more heavily reinforced sections. These joints had no medium for load transfer except the steel cover plate. After 10 yr. most joints of this type were pumping and, as shown in Figure 26, this action in some instances has resulted in serious faulting with transverse cracking of the forward and sometimes the approach slab. Of special interest is the fact that, in spite of the faulting, the heavy reinforcement has thus far held closed all cracks in the pavement areas adjacent to these joints. After 7 or 8 yr., pumping was observed at some of the conventional dowel joints which separate the shorter sections. However, to date, the action at these joints has been so slight that faulting is negligible and fracturing of the slabs has not occurred.

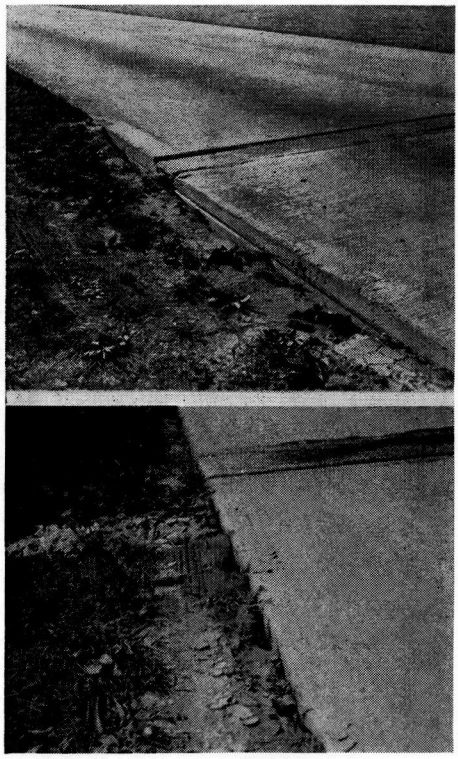


Figure 26. Pumping at Two of the Bridge-Type Joints

At the end of 10 yr. the performance survey showed that, with two exceptions, the only evidence of pumping in the entire pavement was in the vicinity of the transverse joints. One of the exceptions was the development of pumping at two of the cracks that formed in the sections containing the 32-lb. wire fabric. As stated before, the reinforcing steel ruptured at several cracks in these sections, allowing wide separations. Pumping appeared shortly after the reinforcement failed.

The other exception was the appearance of pumping at one point along the edge of the heavily traveled lane some distance from the end of one of the most heavily reinforced sections, which condition was observed during the survey at the age of 10 yr. Mud was not being ejected through any of the cracks but was appearing at the shoulder and the pavement edge. The cracks in the immediate vicinity were seemingly as tightly closed and as nearly watertight as any in the section. For this reason, it is believed that water reached the subgrade by some other channel, along the pavement edge or possibly through the longitudinal joint. At three consecutive cracks in the immediate vicinity of the pumping, where the crack interval is about 2.5 ft., the edges of the cracks on the pavement surface are raveled or chipped to a more pronounced extent than at the other cracks in the section, indicating that the segments of pavement have been deflected considerably by heavy loads in spite of the presence of heavy longitudinal steel. After a few more years of service this condition may reach a point where some form of maintenance is necessary.

The complete absence of pumping at the vast number of transverse cracks in these sections on a pumping type of soil is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections. Closed cracks not only minimize the leakage of free water to the subgrade but, by transferring load, minimize slab deflections as well.

Periodic observations of the weakened-plane warping joints of the special sections have shown that pumping developed at 10 of the 11 surface type joints at which the wire fabric reinforcement failed. In all ten cases the action of pumping began shortly after steel failures were noted. Conversely, pumping did not appear: (1) as long as the wire fabric crossing the warping joints remained

structurally sound, and (2) at any of the seven submerged type joints at which the reinforcing steel failed

The preceding observations indicate that the entrance of free water to the subgrade and large slab deflections have activated pumping Relatively wide separations (up to ⅜-in.) developed at all joints at which the reinforcement failed, thus reducing or destroying the effectiveness of aggregate interlock, and impairing the sealing of the surface type joints Of particular interest is the complete absence of pumping at the submerged type joints, even those at which the wire fabric failed When these joints were installed a copper seal which enveloped the bottom parting strip was incorporated in the design Apparently these seals are still functioning as planned despite the wide separation that has developed at the joints where the longitudinal steel has failed

The effect of repetition of heavy axle loads on the development of pumping is clearly revealed in this investigation As mentioned previously, the experimental pavement is one-half of a divided highway, with the result that the right-hand lane carries a greater number of heavy vehicles At the end of 10-yr the performance survey disclosed only one case of pumping in the left-hand or passing lane of the pavement, in contrast to the condition in the right-hand lane as just described

PART 6 —SMOOTHNESS OF THE PAVEMENT

The common goal of all pavement design is a continued smooth riding surface, economics being, of course, a limiting factor. To evaluate the riding quality of the experimental sections, an instrument for indicating the relative roughness of road surfaces was used With this device, which was developed some years ago by the Bureau of Public Roads[5], it is possible to compare the surface roughness of the various sections by means of a roughness index, expressed in inches per mile of pavement

The basic data indicating relative values of surface roughness of these experimental sections was obtained in August of 1940 or less than two years after construction At that time roughness indices were determined for only the sections in the heavily traveled or right-hand lane of the pavement, it being presumed that, early in the life of the pavement, the surface of both lanes would be equally smooth. A second set of data was obtained in August of 1949, these data including both the heavily traveled and the passing lanes so that the effect of traffic on surface roughness could be ascertained Eliminated from these latter data was the localized condition of roughness found at locations where a pair of bridge-type joints were spaced 10 ft apart

Average values of the surface roughness of short, intermediate-length and long sections are given in Table 11, the sections being grouped according to the range in lengths designated earlier in the report All values were obtained with the single wheel of the roughness vehicle traversing approximately a midlane path

TABLE 11
ROUGHNESS INDICES CLASSIFIED BY LENGTH OF SECTIONS

Range in Section Length	Units per Mile		
	1940	1949	
	Right Lane	Right Lane	Left Lane
ft			
0–120	89	131	129
120–470	86	130	124
470–1310	90	126	124

It is well to point out that, as a result of experience gained in using this equipment over many hundreds of miles of pavements of all types, it has been found that pavements with indices of the order of 80 to 120 have surfaces that would be classed as smooth riding

The data of Table 11 show that, initially, the pavement as a whole was very smooth indeed, indicating that the construction and, particularly, the finishing were unusually good The fact that little difference was observed in the roughness indices of the three groups of sections suggests that, with proper care during installation and finishing, the spacing of expansion joints need not affect

[5] J A Buchanan and A L Catudal, "Standardizable Equipment for Evaluating Road Surface Roughness," *Proceedings*, Highway Research Board, Vol. 24 (1940), also *Public Roads*, Vol 21, No 12, February 1941

the initial riding quality of concrete pavements

A comparison of the roughness indices, Table 11, determined in 1949 with those of 1940 indicates a marked increase in the surface roughness of all three groups of sections, the percentage increase in the units per mile being 47, 51 and 40, respectively, for the short, intermediate-length and long sections of the heavily traveled lane Even with this large percentage increase, the data indicate that the surface of the regular sections after 10 yr. of service is no rougher than some new pavements as constructed.

TABLE 12
ROUGHNESS INDICES CLASSIFIED BY TYPE AND PERCENTAGE OF LONGITUDINAL STEEL

Longitudinal Steel	Units per Mile		
	1940	1949	
	Right Lane	Right Lane	Left Lane
Rail Steel Bars (Deformed)			
percent			
1 82	99	124	127
1 02	85	123	121
45	85	127	124
.26	85	128	121
.11	84	130	122
Billet Steel Bars (Deformed)			
1 82	85	128	123
1 02	78	130	123
45	90	130	121
.26	88	125	128
.11	91	135	129
Wire Fabric (Cold Drawn Wires)			
42	90	137	129
28	90	133	124
24	89	131	127
17	89	137	139
.11	84	120	124
07	98	134	126
Average	88	129	125

Only a slight tendency is noted, however, for the surface of the heavily traveled lane to become rougher, with time, than that of the passing lane Also, in both lanes there is a slight but only a slight tendency for the pavement of the group of long sections to be smoother than that of the short sections, which contain relatively few cracks. It is apparent from these observations that: (1) to date, traffic has had little effect on the increase in midlane surface roughness, and (2) the many cracks that formed in the long sections have not affected the riding quality of the pavement.

In connection with the observed increase in pavement roughness between 1940 and 1949, it will be recalled that Figure 1 shows examples of observed changes in pavement elevation, changes that developed principally through heaving and settlement of the subgrade and that undoubtedly account for part, at least, of the increase in surface roughness

In Table 12 are given the roughness indices for sections containing the various percentages of longitudinal steel for each of the three types of reinforcement. These data, although showing no particular trends for the factors involved, do show that a narrow range in the roughness indices of the various sections existed initially and still exists This implies that all of the sections have remained structurally intact.

TABLE 13
ROUGHNESS INDICES OF THE FOUR 500-FOOT SPECIAL SECTIONS[1]

Weight of Reinforcement	Shear Bars	Units per Mile		
		1940	1949	
		Right Lane	Right Lane	Left Lane
Submerged-Type Warping Joints				
lb. per 100 sq ft				
91	Yes	100	134	137
91	No	90	134	127
45	Yes	90	141	148
45	No	90	134	148
Surface-Type Warping Joints				
91	Yes	79	141	116
91	No	90	120	169
45	Yes	79	148	127
45	No	95	218	137

[1] Each roughness index is based on 250 ft. of pavement

Roughness indices of the four 500-ft. special sections were obtained at the same time as those of the regular sections. These data, given in Table 13, are listed in accordance with the distinguishing features of the special sections.

At the time of the 1940 roughness survey the surfaces of the special sections were as smooth as those of the regular sections, the average index for the two types of pavement being 89 and 88 respectively. This indicates that the surface-type, weakened-plane joints were finished with great care and that the

rather wide and meandering cracks which formed above the parting strips of the submerged-type joints did not impair the riding quality of the pavement at that time.

In 1949 the surface of the special sections was, in general, somewhat rougher than that of the regular sections This greater increase in roughness is probably the result of the conditions at some of the joints that were described earlier, conditions that were not present at the time of the 1940 measurements An example is the large increase in the roughness index of the half of the section with surface-type joints and containing the 45-lb. fabric and no shear bars. It was in this half of the section that pumping developed at the joints where the reinforcing steel failed This action has resulted in some tilting of the 10-ft. slab units and a consequent faulting at the joints until, at present, this particular part of the pavement is quite rough.

CONCLUSIONS

In this report the performance of a wide range of continuously reinforced sections, constructed as a part of a U. S route carrying heavy rural traffic, has been traced through the first 10 yr. of pavement life. The following statements give what appear to be the most significant conclusions to be drawn from the results of this investigation.

(1) Changes in pavement elevation were generally small and nonuniform, the lack of uniformity becoming progressively more pronounced, especially during the first five years of service. The effect of these nonuniform elevation changes was not apparent in either the length changes or the crack patterns of the sections; but, as would be expected, was reflected in the riding quality of the pavement.

(2) Because of the wide range in section lengths an opportunity was afforded to study the effect of subgrade resistance as related to slab movement. The most important conclusions are: (a) excepting the very short sections, the daily and annual changes in section lengths are not directly proportional to length of section; (b) the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place; (c) for subgrade soil of the type on which the experimental pavement was constructed, it was estimated that, during the relatively rapid daily length change, the cenral region of sections greater than approximately 800 ft. will be in a state of complete restraint; whereas for the slowly developed annual length change, the central region of sections somewhat greater in length than the longest section (1,310 ft.) of this investigation will be completely restrained; and (d) for sections of lengths included in this investigation, the data suggest that tensile stresses induced by subgrade resistance are probably larger during the fall than at any other period during the year.

(3) Length changes of a progressive or permanent nature developed in sections of all lengths In the short sections containing comparatively few cracks, it appeared that repeated cycles of moisture and temperature were primarily responsible for such changes. In the longer sections, the tendency of the transverse cracks to open progressively a very small amount was an additional factor contributing to permanent increases.

(4) Transverse cracks in the experimental sections formed essentially at right angles to the axis of the pavement. The surface widths of these cracks because of slight raveling became, in time, much greater than their real widths For a given computed maximum steel stress both the surface and the real widths of the cracks increased approximately directly with a decrease in the percentage of longitudinal reinforcement. In the heavily traveled lane after ten years of service, the average values of the real width of the cracks (obtained in the fall of the year in the central region of the longest section for each percentage of reinforcing steel) ranged from 0.004 in. for the section with 1 82 percent steel to 0.011 in. for the section with 0.45 percent steel. Likewise, the average surface widths of the same cracks ranged from 0 05 to 0 10 in.

(5) The rate of crack development was most pronounced during the early life of the pavement, the greatest rate being between spring and fall of the first year after construction. On the basis of all transverse cracks that developed during the 10-yr. service period, 67 percent had appeared by the end of the first year. Very few cracks formed during the winter months indicating that nonuniform changes in pavement elevation caused by

frost penetration had little influence upon cracking, and, also, that tensile stresses originating from subgrade resistance were no greater during winter periods than during the fall periods

(6) The study of crack development indicated that the average interval between transverse cracks (average slab length) increased with an increase in section length until a peak value was reached, beyond which there was a rapid decrease in average slab length that became more gradual and finally approached a constant value for the longer sections If one were interested only in the minimum of transverse cracks and joints, the data suggest that, in reinforced concrete pavements, the joints be spaced at approximately 100-ft. intervals However, it must be kept in mind that this investigation most conclusively shows that the character and not the number of cracks is of the greater importance In the longer and consequently more heavily reinforced sections, many fine cracks have developed at frequent intervals, but these sections have continued to be strong, durable structural units after ten years of heavy traffic service

(7) The frequency of cracking increased from a minimum value at the end of a section to a maximum value in the central area For some distance, beginning at the ends of the longer sections, the frequency of cracking increased directly with increase in distance The maximum values of crack frequency, as found in the central area of the sections, increased progressively with increase in section length It seems reasonable to assume that such values would continue to increase until the sections are long enough to develop complete restraint to slab movement The 10-yr. data suggest that, for the conditions obtaining in this investigation, the crack interval in the region of complete restraint might be expected to be approximately 2 0 to 2 5 ft provided, of course, that the reinforcement was adequate.

(8) Repetition of traffic loads had only a slight influence on the development of transverse cracks. At the end of 10 yr , 53 percent of the transverse cracks present in both lanes of the pavement formed in the heavily traveled right-hand lane However, the greater volume of traffic using the right-hand lane produced more raveling and other superficial damage to the edges of the cracks than the lighter traffic on the passing lane, the average surface width of cracks in the heavily traveled lane being approximately three times that of the cracks in the passing lane. Traffic had some effect, also, on the real widths of the cracks, this being less pronounced than in the case of the surface widths.

(9) All sections were so conservatively designed that the limiting length of section for each percentage of longitudinal steel was not determined

(10) Longitudinal steel reinforcement, within the range of the computed maximum stress values of this investigation, held closed all cracks excepting those in the sections reinforced with the 32-lb wire fabric In several cases the steel crossing the cracks that formed in the sections containing this light fabric broke, probably from shearing forces It is indicated that wire fabric as light as 32 lb per square should be used with caution as reinforcement in concrete pavements

(11) The presence of the heavy longitudinal bar reinforcement was not in any way detrimental to the condition of the concrete in the pavement as attested by the complete absence of longitudinal cracking above such bars and by the continued durability of the concrete In fact, the manner in which the steel held closed all cracks, especially those in the heavily reinforced sections, is believed to be conducive to distributed interfacial pressure and should minimize damage of the concrete from concentrated pressure such as sometimes develops at cracks in plain concrete pavements

(12) The type of reinforcement had only a slight effect on the observed length changes of the sections or on the frequency of cracking in the central portion of long sections; but, for some unknown reason, seems to have had considerable influence on the average interval between cracks in sections of 300 ft. or less in length On the other hand, the working stresses within the range of the computed maximum values exercised no significant control over the length changes and crack patterns of the sections, probably because of the conservative design assumptions.

(13) Heavy reinforcement caused the length changes and crack patterns of the sections to

be quite symmetrical about the center of each section, thus indicating a structure of predictable behavior

(14) In the four special 500-ft sections, containing warping joints at 10-ft. intervals, certain inherent weaknesses developed during the 10-yr. of traffic service These weaknesses point out, first, the necessity of providing the warping joints with load transfer or shear units; and, second, the need of a heavier wire fabric than 45 lb per square In the halves of the sections provided with shear bars and containing the 91-lb. fabric, the steel did not break or inelastically elongate and the pavement remained structurally intact, the riding quality of these halves being nearly the same as that of sections reinforced with continuously bonded steel

(15) During periods of contraction, the continuous reinforcement in the 500-ft special sections exercised considerable control over the length changes of the section as a whole However, in spite of the continuity of the reinforcement, the warping joints opened progressively with time This behavior is definitely undesirable since a residual opening of the joints would dissipate all or part of the elastic elongation of the 36 in of unbonded steel and, in time, may cause failure of the relatively lightweight reinforcement. A corrective measure for the preceding condition would be to decrease the amount of available expansion space.

(16) Pumping developed at many expansion joints, but, with two exceptions, was completely absent at the vast number of transverse cracks This is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections, thus reducing slab deflections and minimizing the passage of free water to the subgrade soil The absence of pumping at the submerged-type warping joints of the special sections indicated that the copper seals which enveloped the bottom parting strips prevented the leakage of free water to the subgrade soil

(17) Relative roughness determinations of the regular sections showed that their surfaces were very smooth initially, and at present are no rougher than some concrete pavements as constructed The many fine cracks that formed in the long sections have not affected the riding quality of the pavement. The sections containing warping joints at 10-ft. intervals were as smooth initially as the regular sections indicating that, with proper care during installation and finishing, closely-spaced warping joints need not affect the initial riding quality of concrete pavements However, where certain weaknesses have developed such as faulting of the joints, these special sections have become much rougher than the regular sections.

ECONOMIC BENEFITS

The performance of the Indiana experimental sections has indicated certain economic benefits to be derived from long, continuously reinforced pavement of the type included in this investigation, namely. (1) the fine cracks, even though frequent, ravel only slightly with traffic and exposure, a condition that may be considered superficial and one that will require no maintenance, (2) except in localized areas of extremely poor subgrade, pumping will not develop thus minimizing the need for base courses or other expensive subgrade treatments, and (3) the riding quality of the pavement might be expected to remain excellent and the pavement itself would be protected from damaging impact forces such as frequently develop at faulted joints and cracks

More recently other researches relating to the use of continuously bonded longitudinal reinforcement have been inaugurated[6] The experimental sections in these pavements are all longer than the longest section of the Stilesville experimental pavement and are less heavily reinforced Sections of various thicknesses, reinforced with various percentages of longitudinal steel and constructed both with and without subbases are included. While the pavements have not been in service long enough to permit conclusions to be drawn

[6] W R Woolley, "Continuously Reinforced Concrete Pavements Without Joints," *Proceedings*, Highway Research Board, Vol. 27 (1947)

William Van Breemen, "Preliminary Report on Current Experiment with Continuous Reinforcement in New Jersey," *Proceedings*, Highway Research Board, Vol 27 (1947).

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

it seems probable that eventually they will provide considerable additional data on the relative economies of continuously reinforced pavements and those which are designed as a series of comparatively short independent slabs.

Thus, it is possible that, when all factors are considered, a concrete pavement without joints and reinforced with continuous bonded steel in sufficient amount to resist all stresses safely and to hold all cracks closed may, in many cases, cost no more than current designs of concrete pavement which include greater slab thickness, lighter reinforcement, transverse joints and subgrade treatment.

DISCUSSION

WAYNE R. WOOLLEY, *Bureau of Public Roads* —Although the experimental pavement has not furnished the data to allow a precise design of a continuously reinforced concrete pavement of indefinite length to be made, the reports of Messrs. Cashell and Benham have contributed valuable information toward this end.

Steel for this experimental pavement was designed by the subgrade friction formula in the same way that the steel is designed at the present time for most pavements. It is noted that no broken steel was found except at three cracks on pavement containing 32-pound mesh. Considering this project alone it would seem that the subgrade friction formula gives satisfactory results. However, observations made on other projects in the middle west cast considerable doubt on the adequacy of this formula.

Several years ago it was observed that open and faulted cracks were visible in reinforced pavements after they had been in service approximately eight or more years. Table A accompanying this discussion gives information on 37 reinforced concrete projects in Michigan, Indiana and Kentucky. For each of these projects, the theoretical stress based on a coefficient of subgrade friction of 1.5 was calculated and is shown in the table. The calculated stress ranged from 14,800 to 81,500 lb. per sq. in. of which 30 projects had a calculated stress less than 55,000 lb. per sq. in. Of these 30 projects, 26 contained some open cracks indicating overstressed and probably broken mesh reinforcement. The cause of this broken steel has not been definitely determined. Open and faulted cracks may be found whether or not the joints contain load transfer so that friction in the load transfer devices is not always responsible. (See lines 17, 18 and 12, 13 in Table A). Very little correlation was found between type of subgrade and the frequency of open cracks although serious pumping undoubtedly contributes to the formation of such cracks (See lines 14, 17, 18 which show open cracks on a sand subgrade.) The schematic diagram of Indiana Project NRM 69 D, Figure A (line 18) illustrates that an open and faulted crack was found over sand subgrade within 7 ft of an expansion joint which contained no load transfer Open and faulted cracks over clay, or pumping subgrades were found on a number of projects. Some evidence of overstressed steel was found in pavements containing mesh reinforcement with a calculated maximum stress as low as 18,700 lb. per sq in.

Perhaps it should be pointed out that with one exception all of the projects containing evidences of broken steel also contained expansion joints. The one exception (line 32 in Table A) contained four open cracks in 5 miles and it appeared that these four cracks may have been the result of fill settlement The presence of expansion joints probably increases the tensile stresses due to subgrade friction. Pavements without expansion joints may be presumed to be in compression a considerable portion of the time and to have reduced tensile stresses.

Although this table shows that in some cases, at least, certain things are not the cause of broken steel, it does not clearly indicate what does cause the steel to break. However, as a result of this study and from visual observation of the movement of pavements under heavy loads, it seems to me that the primary cause may be the result of repeated deflection under loads plus the effect of rust on the steel at the crack in the concrete. This rust is not believed to be sufficient to cause

TABLE A

Line No	State and Proj No.	X-Section	Joint Spacing E	Joint Spacing C	Type Load-Transfer	Type Subgrade	1941[a] Traffic	Age in Years	Theoret Stress in Steel	Condition of Cracks in pavement
		in	*ft*	*ft*						
1	Mich R 41	10 unif.	100	None	None	Non Pumping	9,800	16	81,500	Many open and faulted
2	Mich 91 & 139 E	9 "	100	"	"	" "	11,000	17	73,500	Av 18 open cracks per mi
3	Mich 139 G	9 "	100	"	"	" "	800	15	73,500	Several open cracks
4	Mich 148 C	9 "	100	"	"	" "	3,000	17	73,500	Many open and faulted
5	Mich 231 B	9-7-9	100	"	"	" "	2,500	17	59,000	About one open crack per mi
6	Mich 236 C	9-7-9	100	"	"	" "	4,000	17	59,000	3 to 15 open cracks per mi
7	Mich 232 D	9-7-9	100	"	"	" "	3,500	16	59,000	No open cracks observed
8	Mich 155 G	10-8-10	60	"	"	Slight Pumping	3,000	11	44,000	Many open and faulted
9	Ind 69 J	9-8-9	80	40	Translode at E Dowels at C	Sand	City Street	13	42,700	145 open cracks per mi
10	Ind HM69 H	9-7-9	80	40	Dowels	Part Sand " Pump	5,000	13	39,000	1 to 3 open cracks for each 40-ft slab
11	Ind 46 A	9-7-9	80	40	"	Pumping	6,000	12	39,000	Many open and faulted
12	Ind 263 B	9-7-9	80	40	Translode	Slight Pumping	6,000	13	37,600	115 open cracks in one mi
13	Ind GM 69 H	9-7-9	80	40	Dowels	Slight Pumping	5,000	12	37,600	Ave about one open crack in each 40-ft slab
14	Ind 31 F	9-7-9	80	40	"	Sand	15,000	13	37,600	1 to 3 open cracks in each 40-ft slab
15	Ind 499 B	9-7-9	80	40	?	Non Pumping	500	12	34,600	About one open crack per mi
16	Ind 728 C	9-6-9	120	40	?	"	800	7	33,200	No open cracks
17	Ind 367 A	9-8-9	105	35	None at E Dowels at C	Sand	15,000	14	33,000	5 open cracks in 0 8 mi
18	Ind 69 D	9-8-9	105	35	None at E Dowels at C	Sand	City Street	14	33,000	Averages about one open crack per 35-ft slab
19	Ind 285 A	9-6-9	100	33	Plate-dowel	Pumping	1,500	11	33,000	Numerous open cracks near Charlestown
20	Ind 74 E	9-7-9	40	None	Plate-dowel	"	1,900	11	30,000	Cracks open where pumping is serious
21	Ind 565 A	8-5-8	100	20	?	Gravel	400	10	30,000	Very few cracks and none are open
22	Ind 77 A(3)	9-7-9	120	40	Dowels	Slight Pumping	4,000	9	30,000	A number of open and faulted cracks
23	Ind 17 N(2)	9-7-9	120	40	Plate-Dowels	Pumping	4,000	9	30,000	Open cracks probably caused by pumping
24	Ind 48 A(2)	9-7-9	120	40	Dowels	Pumping	5,000	8	30,000	Many open cracks-pumping probably the cause
25	Ind 63 E	9-7-9	105	35	None at E Dowels at C	Gravel	City Street	14	29,600	Only one open crack in 1 4 mi
26	Ind 221 D	9-7-9	105	35	"	Non Pumping	City Street	14	29,600	About 15 open cracks per mi
27	Ind 40 A	9-7-9	80	40	[b]	Slight Pumping	6,000	13	29,500	Frequency of open cracks seems to depend on subgrade

TABLE A—(*Continued*)

Line No	State and Proj No	X-Section	Joint Spacing E	Joint Spacing C	Type Load-Transfer	Type Subgrade	1941[a] Traffic	Age in Years	Theoret. Stress in Steel	Condition of Cracks in Pavement
		in	*ft*	*ft*						
28	Ind 564 A	9-6-9	100	33	Translode	Pumping	800	10	27,200	Many open and faulted
29	Ind 564 B	9-6-9	100	33	Dowels	Pumping	800	10	27,200	Many open and faulted
30	Ind 401	9-6-9	80	40	Dowels at C Transl at E	Non Pumping	1,700	13	26,000	About 20 open cracks per mi Some frost heaves
31	Ind 6 A(2) (3)	9-7-9	100	33	?	Pumping	5,000	9	24,600	About 4 open cracks per mi
32	Ind 6 A(2) (3)	9-7-9	None	33	"	"	5,000	9	24,600	4 open cracks in 5 mi
33	Ky 194 E, F	9-7-9	90	30	?	Non Pumping	2,000	10	21,600	63 open cracks in 4 3 mi Some evidence of joint friction
34	Ind 221 A	9-7-9	90	30	None at E Dowels at C	Non Pumping	2,000	17	18,700	2 open cracks in 2,700 ft
35	Ind 221 A	9-7-9	100	20	"	"	2,000	17	17,000 to 20,300	No open cracks
36	Ind 40 A	9-7-9	None	20	Dowels	Non Pumping	6,000	13	14,800	Very few cracks—all of which are tight
37	Ind 650 A	9-7-9	120	40	T-G bars	Pumping		9	Approx 28,000	Expanded metal reinforcing numerous open cracks

[a] Traffic data partly estimated
[b] This is an experimental project and contains the following types of load transfer· dowels, translode, J-bars, Acme Open and faulted cracks occurred with all types of load transfer.

an appreciable reduction in cross sectional area, but may be enough to act as a stress raiser at that point. The fact that no broken steel was observed in pavements less than 8 yr. old may be because this length of time

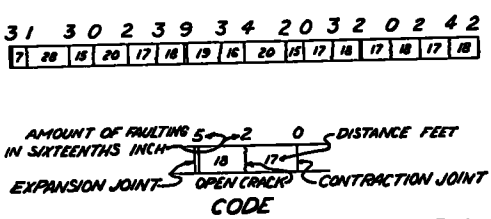


Figure A. Open and Faulted Cracks—Indiana Project NRM 69D Constructed 1934—Sand Subgrade—Estimated Traffic 6000 Daily—9-8-9-in. Cross Section, Reinforced—Theoretical Stress 33,000 psi.—¾-in. Dowel Bars at Contraction Joints—No Load Transfer at Expansion Joints

was required foi the effects of rust and repeated loads to become serious The repeated stresses caused by deflection under loads plus the effect of the rust as a stress raiser might be expected to be effective only after the pavement had been in service an appreciable number of years In this connection it is of interest to note that the authors state in conclusion No. 10 that "Longitudinal steel reinforcement . held closed all cracks excepting those in the sections reinforced with 32-lb. wire fabric" The calculated stresses in the 32-lb. fabric sections were the same as in the heavier sections, i e 25,000 to 55,000 lb. per sq in It also seems significant that open cracks were observed for the first time in the 32-lb fabric sections at the age of 10 yr Thus, the authors indicate that these stresses are not safe for a 32-lb fabric after 10 yr.

Data presented in Table A indicate that these stresses are not safe in all cases for considerably heavier weight fabrics. It may well be that in addition to designing pavement reinforcement to resist tensile stresses due to subgrade friction another factor should be added to take care of repeated live-load stresses This suggests that a certain minimum amount of reinforcement is necessary to resist repeated live loads and that to this minimum should be added an amount necessary to take care of tensile stresses due to

subgrade friction. The amount of steel necessary to resist live-load stresses is not known, but it seems safe to say that some of the lighter weight fabrics in use today are insufficient to resist permanently all the stresses to which they are subjected, even though they were designed by the subgrade friction formula to have stresses less than 55,000 lb. per sq in

WILLIAM E WILLEY, *Arizona Highway Department*—In the conclusions I would like to see a statement made to the effect that the tests so far show that the reinforced section is more economical or is not more economical than a plain unreinforced section with an adequate subgrade and base course Much is said in engineering and highway administrative circles about whether or not reinforcing steel in a concrete highway section is a necessity Some advocate the use of steel while others say the money could better be used in the base or for a thicker unreinforced concrete slab

From the number of cracks developed in the reinforced test section during the past 10 years it appears that its physical performance record has not been so good. In Arizona on Highway U S 66 in the northern part of the State is a concrete section of about the same age. With no reinforcing it now has fewer cracks and has required probably less maintenance than the test section reported upon I believe that after 10 years some definite conclusion along economic lines should be apparent and commented upon.

CASHELL AND BENHAM, *Closure*—The primary objective of the experimentally reinforced sections of the Indiana pavement was to study the effects of varying amounts of longitudinal steel in sections of various lengths Therefore, data from such a pavement with frequent changes in section length, type and amount of reinforcement and without comparable plain concrete pavement can warrant only the general conclusions given in the part of the report entitled "Economic Benefits".

The second paragraph of Mr Willey's discussion indicates a lack of appreciation of the structural significance of cracks in reinforced pavement as compared with those in plain concrete pavement

LIVE LOAD STRESS MEASUREMENTS ON THE FORT LOUDON BRIDGE, PENNSYLVANIA

NEIL VAN EENAM, *Highway Bridge Engineer, Bureau of Public Roads*

SYNOPSIS

In testing a low truss steel bridge, eight test vehicles were used ranging in weight from a two-axle 27,830-lb. truck to a five-axle 150,000-lb tank transporter The bridge had a span of 110 ft 8 in between centers of bearing Electro-magnetic gages were used, and during each run of a test vehicle, 24 simultaneous stresses were recorded. In this preliminary report, figures are presented for typical truss members, showing static stresses, impact stresses at various speeds and secondary stresses Data are presented on vertical and transverse horizontal truss deflections at the center of the span and stresses in the floor system are discussed

While these data apply only to the particular bridge tested, it is believed that the results will have some bearing on the design of highway bridges in general Tests of this kind should lead to a better understanding of the behavior of bridges under moving live loads

Under a joint project of the Bureau of Public Roads and the Pennsylvania Department of Highways, a low truss bridge at Fort Loudon, Pennsylvania, was tested during September and October, 1948 The Association of American Railroads conducted the tests, furnishing the necessary equipment and supplying the services of an experienced field party. The data have now been analyzed and the preparation of the final report is in prog-