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Experimental Continuously-Reinforced Concrete Pavements:

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Experimental Continuously-Reinforced Concrete Pavements: Progress Reports--1959

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Contents

Continuously-Reinforced Concrete Pavements
n Pennsylvania

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The Hamburg project in Berks County, Pa., has attained one year of service performance. The York project is now two years old, although it, too, has been opened to traffic only one year. This report describes the general condition of each project and presents the combined results of a comprehensive crack frequency and width survey on both pavements. A study is made of the different crack patterns evolved as a result of paving in opposite seasons of the year. Further analysis is made of the effect of various pavement thicknesses upon service performance of the Hamburg roadway and of the effect of various depths of subbase material.

A scattered number of detrimental transverse cracks appeared on the Hamburg project within several months after completion. The nature of these cracks and the investigation to determine their cause is described, together with an account of the subsequent repair of these damaged areas. The specifications for these repairs are outlined, emphasizing the necessary precautions required for this work. A detailed description is given of the methods employed in pavement removal, in restoring the continuity of the steel, and in replacing the damaged concrete.

THE TWO continuously-reinforced concrete pavements in Pennsylvania have been
n service more than a year now, and some very noteworthy characteristics and be-
aviors have been observed. A brief background history of each project will acquaint
he reader with the basic differences of these two projects (1, 2, 3).

Both pavements were constructed with reinforcing steel designed at 0.5 percent of
he cross-sectional area; there the similarity ends. The York County project (Fig.
) is 2.19 mi in length and is located on the Harrisburg-Baltimore Expressway, Traf-
ic Route 111, approximately 3 mi north of York, Pa. It was constructed in September
nd October 1956 and although the pavement itself is two years old, it has been opened
o traffic only since October 1957. The pavement is uniformly 9 in. thick and rests
n a 6-in. subbase material throughout its length. Steel reinforcement is of the bar
nat type.

The continuous section in Berks County (Fig. 2) is a portion of Traffic Route US 22,
ocated 2 mi east of Hamburg, Pa. This pavement was placed in May, June and July
957 and opened to traffic in October of the same year. Pavement thicknesses were
aried to include 7-, 8-, and 9-in. depths, and subbase material was designed for
lternate depths of 6- and 3-in. minimums; however, the final result varied somewhat
rom the design thickness. Here also, bar mats were used for reinforcement, with
he exception of 1,000 ft in the eastbound lanes, where wire mesh was used.

Both pavements presently carry large volumes of traffic, which will undoubtedly
ecome greater as the roadways are linked to the Interstate system. Latest figures
how the York pavement carrying an average of 7,500 vehicles daily and the Hamburg
avement carrying 8,500, a large percentage of which is heavy truck traffic. Both
rojects are in good condition. There has been no apparent deviation from the original
rade line, and the riding qualities are quite good. With the exception of a few un-
sual cracks at Hamburg, which are described later in this report, all cracks appear
o be structually sound.

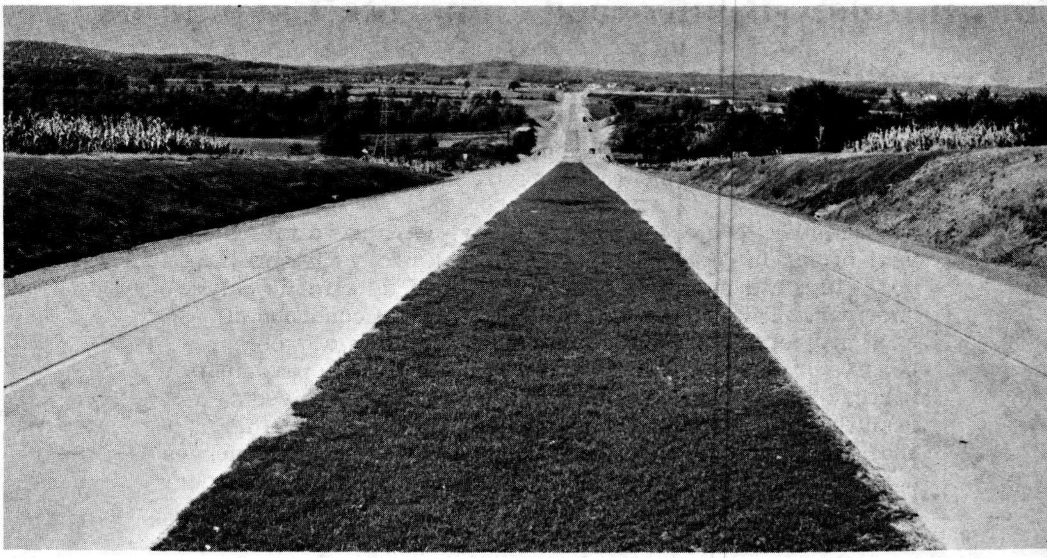


Figure 1. Traffic Route 111, York County.

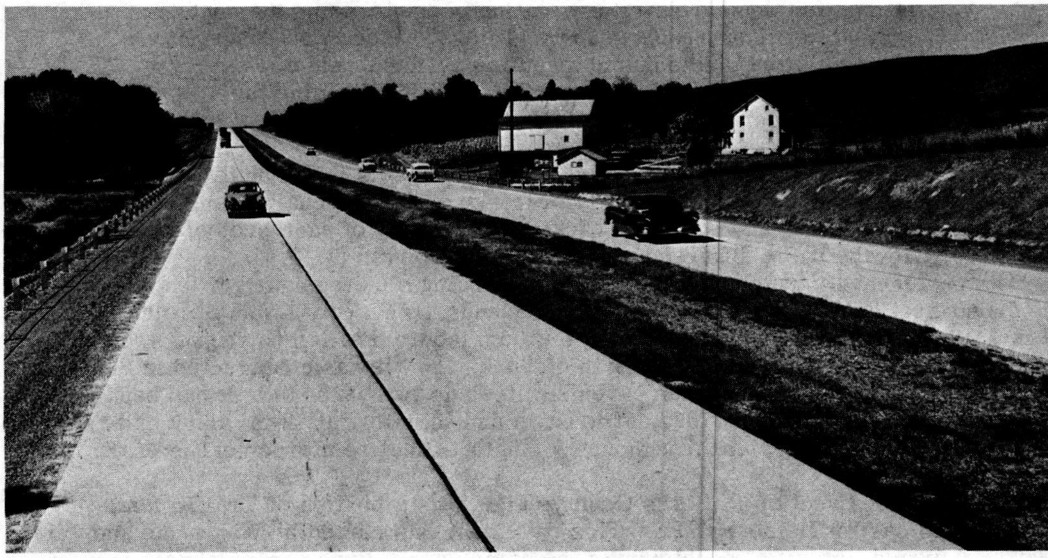


Figure 2. Traffic Route 22, Berks County.

The latest information obtained from the profilometer or roughness survey is shown in Table 1.

CRACK FREQUENCY

It is well known that continuous pavements placed in opposite seasons of the year react quite differently and produce various results. The two projects under study in Pennsylvania point up this characteristic quite vividly. The Hamburg pavement was placed under an average high temperature of 85 F and an average low of 67 F, whereas similarly, on the York project, the average high was 66 F and the average low was 44

TABLE 1

RESULTS OF PROFILOMETER SURVEYS

Date	Project	Lane	Length (ft)	Inches	
				Total	Per Mile
July, 1957	York	North-Outside	10,700	48	23.7
Nov. 1958	York	North-Outside	11,560	40	18.3
Nov. 1958	York	North-Outside	-	-	-
July, 1957	York	South-Outside	10,700	51	25.2
Nov. 1958	York	South-Outside	11,560	42	19.2
Nov. 1958	York	South-Outside	1,000[a]	8	42.2
Aug. 1957	Hamburg	East-Outside	6,300	30	25.2
Nov. 1958	Hamburg	East-Outside	10,800	40	19.5
Nov. 1958	Hamburg	East-Outside	1,000[a]	6	31.7
Aug. 1957	Hamburg	West-Outside	10,800	57	27.8
Nov. 1958	Hamburg	West-Outside	10,800	46	22.5
Nov. 1958	Hamburg	West-Outside	1,000[a]	4	21.1

[a] Conventional pavement.

The striking contrast in the crack pattern is effectively shown in Figures 3 and 4. The number of cracks on the Hamburg pavement is approximately 85 percent greater than the number found at York. The pattern seems to be fairly well established by six months and any cracking subsequent to that time does not materially effect the pattern. The graph for the Hamburg pavement, in particular, shows the effect of construction temperatures upon the number of cracks per 100 ft. The effect of early morning paving is also noticeable in the earlier surveys, but tends to level off after one year. It is interesting to note that the section of pavement at Hamburg between Stations 237 and 255 in the eastbound lanes, where paving was delayed until fall because of a slide condition, has a crack pattern quite similar to that at York. The temperatures and conditions under which both sections were placed, were practically identical.

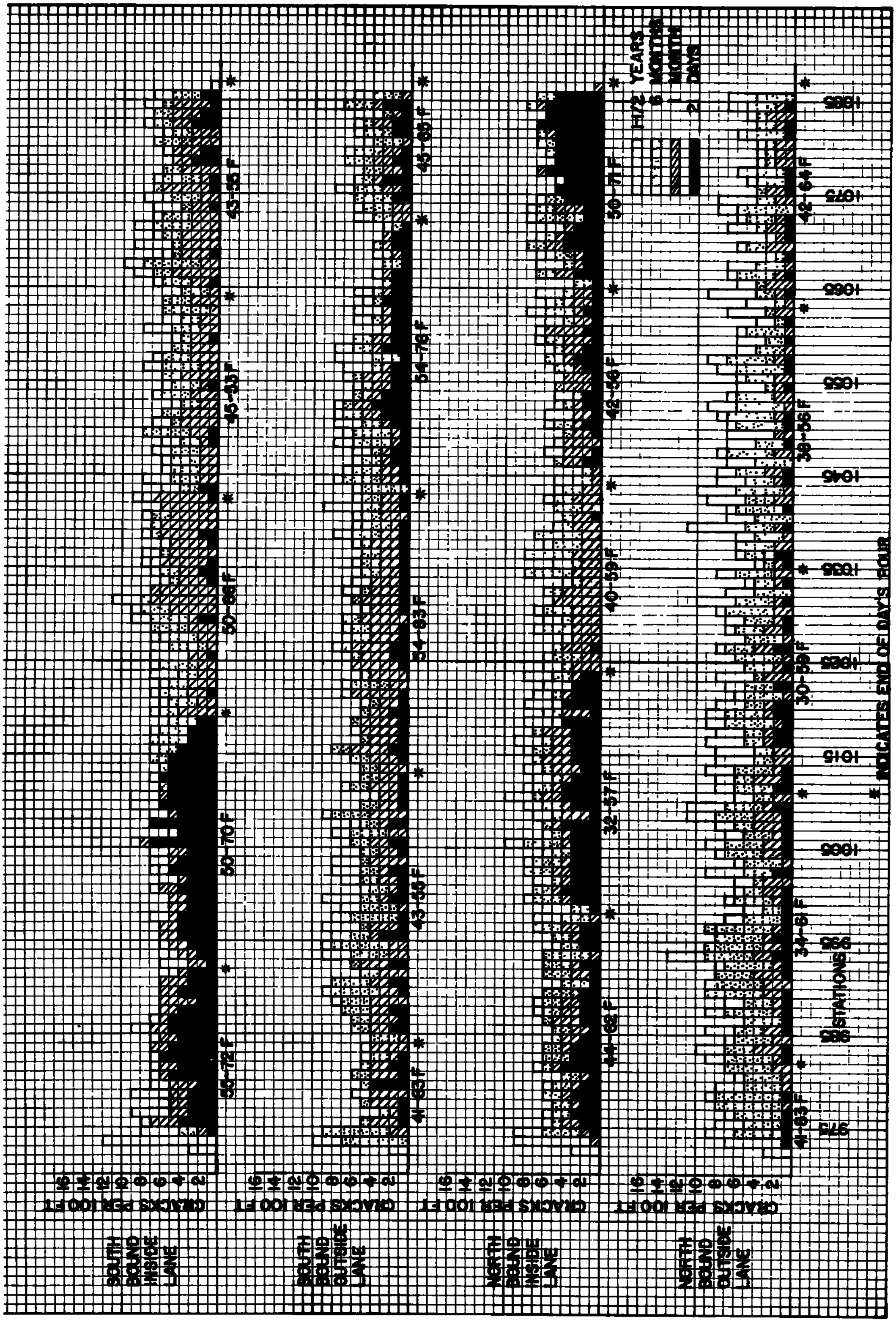
The crack spacing on both of these projects is somewhat greater than that experienced on other continuous pavements at the same age. Careful observations will be made to determine whether this condition continues.

The data from the graph have been summarized and condensed in Tables 2 and 3 to show the average number of cracks per 100 ft for all four lanes and each pavement depth. The average distance between adjacent cracks is also shown. Thus, in a 2-yr period all four lanes of the York project have leveled off at an average 15-ft interval between cracks. Comparatively few new cracks developed in the second year.

The Hamburg pavement after one year of service likewise has attained a fairly uniform frequency in all four lanes, with an average crack spacing of approximately 8 ft. An exception to this, of course, is in the eastbound lanes, where a portion of the 9-in. pavement was not integral with the continuous pavement until the section between Stations 237 and 255 was paved.

CRACK WIDTH

Two methods have been employed in recording crack widths on both continuous projects. The microscope was used at first as a matter of necessity before gage equipment was available. After an invar-type gage was obtained, both methods were used in order to obtain comparative results. Unless measuring plugs have been installed in the pavement prior to the development of any cracking, the invar gage cannot be used


SOUTH BOUND INSIDE LANE
SOUTH BOUND OUTSIDE LANE
NORTH BOUND INSIDE LANE
NORTH BOUND OUTSIDE LANE
CRACKS PER 100 FT
2 4 6 8 10 12 14 16
55-72 F
50-70 F
50-66 F
45-63 F
43-55 F
41-63 F
43-55 F
54-83 F
54-76 F
45-65 F
44-62 F
32-57 F
40-59 F
42-56 F
50-71 F
41-63 F
34-61 F
30-59 F
38-56 F
42-64 F
1 1/2 YEARS
6 MONTHS
1 MONTH
2 DAYS
975 985 STATIONS 995 1005 1015 1025 1035 1045 1055 1065 1075 1085
* INDICATES END OF DAY'S TOUR

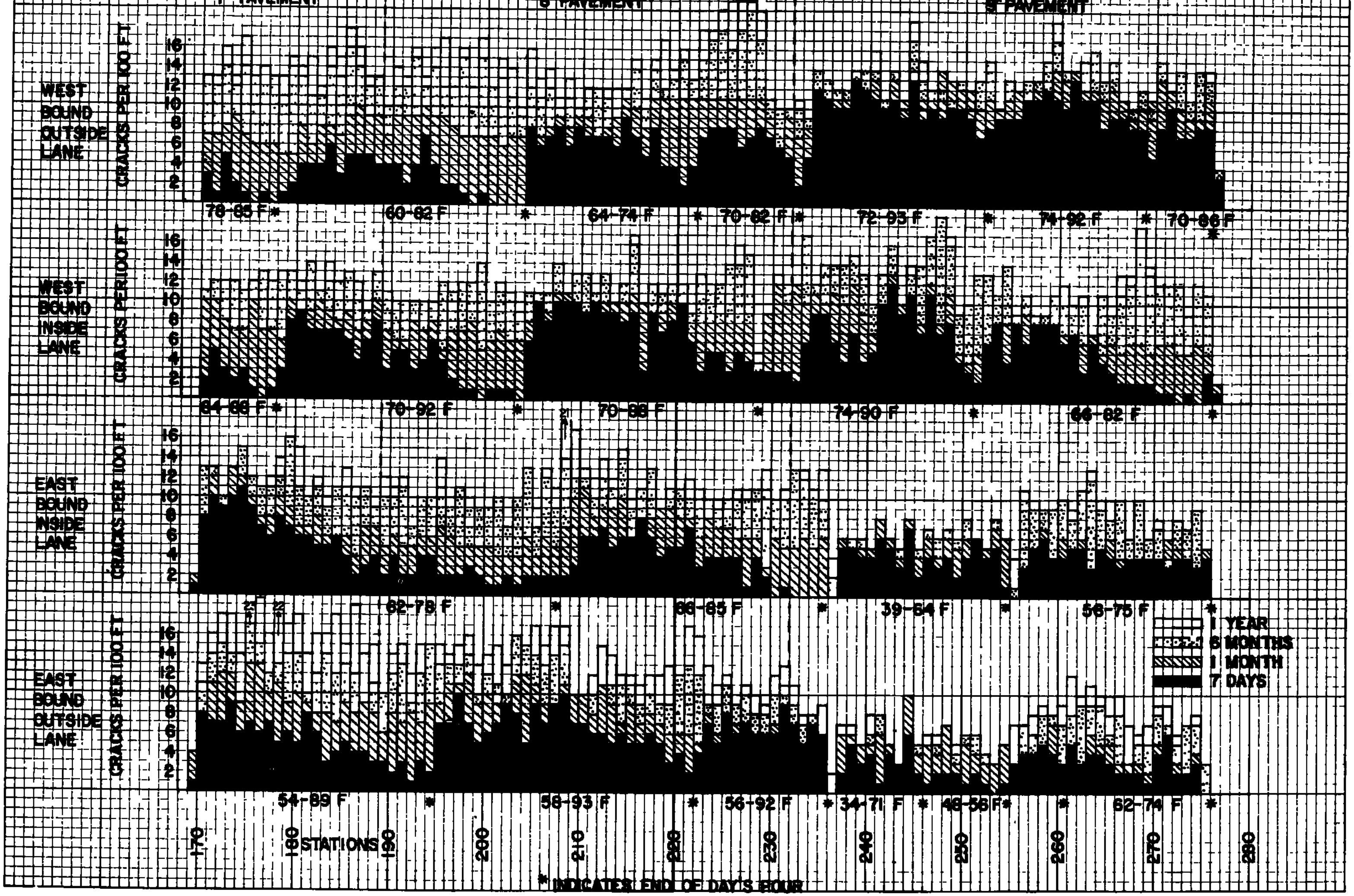


Figure 4. Crack frequency survey, Hamburg project.

to record an actual crack width, but it can be used to determine the extent to which any crack is opening or closing. Thus, it was necessary to correlate the first invar gage reading with a microscope reading taken at the same time.

The brass plugs installed on a number of cracks within certain areas on both projects were not set in the pavement until approximately three months after completion of paving at Hamburg and one year after completion at York. In the area between Stations 237 and 255 at Hamburg, which was paved at a later date, brass plugs were place in the fresh concrete every 10 in. in both eastbound lanes in a 100-ft section between Stations 240 and 241. In this way, the resulting cracks had to occur between some of the plugs, and since invar readings were taken twice before any cracking developed, a "zero" reading was obtained for a direct comparison with microscope readings. Dat obtained thus far from several readings in this area are given in Table 4. Generally, the comparative results obtained with these methods vary from good to fair and show quite good correlation of average widths. The variations in width range from 0.001 to 0.008 in. The authors believe that the microscope is as accurate an instrument as the invar gage, but that certain difficulties tend to keep it from being the preferred metho It is very difficult to set the microscope on the exact spot for successive readings, an if there is the slightest amount of spalling or chipping of the crack, the reading is affected accordingly. The angle at which the microscope is read makes a slight difference, thus care must be taken to read the instrument absolutely perpendicularly to the crack. The microscope is only capable of reading the surface width, which in reality may not be truly indicative of the actual width. Obviously, however, even though the invar gage is preferred, care must also be exercised in its use to insure accurate results.

TABLE 2

CRACK FREQUENCY AVERAGES; YORK PROJECT

Lane	Pavement Thickness (in.)	Cracks (no./100 ft)					Crack Spacing (ft)					Placing Temp. (°F)
		21 Days	1 Mo	6 Mo	1½ Yr	2 Yr	21 Days	1 Mo	6 Mo	1½ Yr	2 Yr	
North outside	9	0.70	1.93	5.15	7.08	7.16	142.9	51.8	19.4	14.1	14.0	51
North inside	9	2.01	3.60	5.27	6.17	6.32	49.8	27.8	19.0	16.2	15.8	51
South outside	9	1.21	3.14	5.14	6.16	6.32	82.6	31.8	19.5	16.2	15.8	60
South inside	9	1.81	4.03	5.51	6.83	6.98	55.2	24.8	18.1	14.6	14.3	56

TABLE 3

CRACK FREQUENCY AVERAGES; HAMBURG PROJECT

Lane	Pavement Thickness (in.)	Cracks (no./100 ft.)				Crack Spacing (ft)				Placing Temp. (°F)
		7 Days	1 Mo	6 Mo	1 Yr	7 Days	1 Mo	6 Mo	1 Yr	
East outside	7	5.6	9.4	12.3	16.3	17.9	10.6	8.1	6.1	72
	8	5.9	8.5	11.4	13.3	16.9	11.8	8.8	7.5	74
	9[a]	3.1	3.7	6.4	7.6	32.3	27.1	15.6	13.2	64
East inside	7	6.4	8.0	11.1	11.7	15.6	12.5	9.0	8.5	70
	8	3.3	6.6	10.4	11.5	30.3	15.2	9.6	8.6	73
	9[a]	2.9	4.1	7.8	7.8	34.5	24.4	12.8	12.8	64
West outside	7	2.8	7.0	12.4	13.6	35.7	14.3	8.1	7.4	76
	8	4.9	8.4	12.7	14.2	20.4	11.9	7.9	7.0	72
	9	9.3	10.8	12.7	13.1	10.8	9.3	7.9	7.6	80
West inside	7	4.5	8.1	11.4	11.6	22.2	12.3	8.8	8.6	84
	8	5.2	8.3	11.5	11.6	19.2	12.0	8.7	8.6	81
	9	5.3	7.6	12.2	12.3	18.9	13.2	8.2	8.1	78

[a] Area between Sta. 237 and 255 excluded.

(a) Eastbound Outside Lane

TABLE 4

COMPARATIVE RESULTS OF MICROSCOPE AND INVAR WIDTH READINGS; HAMBURG PROJECT

	1 Month			2 Months			3 Months			6 Months			1 Year		
Stations	Air Temp. (°F)	Micro (in.)	Invar (in.)	Air Temp. (°F)	Micro (in.)	Invar (in.)	Air Temp. (°F)	Micro (in.)	Invar (in.)	Air Temp. (°F)	Micro (in.)	Invar (in.)	Air Temp. (°F)	Micro (in.)	Invar (in.)
240+02.9	54	0.032	0.037	40	0.048	0.049	42	0.040	0.061	71	0.008	0.019	78	0.016	0.016
240+19.7	54	0.004	0.006	40	0.004	0.008	42	0.008	0.006	71	0.008	0.006	78	0.020	0.009
240+30.4	54	0.012	0.012	40	0.012	0.015	42	0.012	0.011	71	0.014	0.012	78	0.008	0.016
240+39.6	54	-	0.001	40	0.008	0.011	42	0.006	0.009	71	0.004	-	78	0.008	0.011
240+50.5	54	0.004	0.007	40	0.010	0.008	42	0.008	0.011	71	0.008	0.006	78	0.008	0.008
240+63.0	54	0.004	0.009	40	0.008	0.012	42	0.004	0 008	71	0.016	0.007	78	0.010	0.005
240+69.5	54	-	0.002	-	-	-	42	0.008	0.008	71	0.006	0.007	78	0.008	0.009
240+86.4	54	0.012	0.015	40	0.012	0.021	42	0.020	0.019	71	0.010	0.016	78	0.012	0.021
(b) Eastbound Inside Lane															
240+18.7	46	-	0.013	34	0.012	0.045	-	-	-	73	0.012	0.013	78	0.008	0.023
240+33.9	46	-	0.015	34	0.016	0.021	-	-	-	73	0.006	0.012	78	0.010	0.019
240+47.9	46	-	0.012	34	0.012	0.016	-	-	-	73	0.006	0.011	78	0.012	0.018
240+70.4	46	-	0.000	34	0.010	0.018	-	-	-	73	0.008	0.012	78	0.012	0.018
240+86.2	46	-	0.010	34	0.016	-	-	-	-	73	0.008	0.011	78	0.012	0.018

The core shown in Figure 5 effectively points out the variation in crack width throughout the depth of the pavement. Maximum width is observed at the pavement surface (top). As the crack progresses through the core, it decreases in width, being practically non-existent at the bottom of the core. Only by allowing water to penetrate through the core does the crack become visible at the bottom.

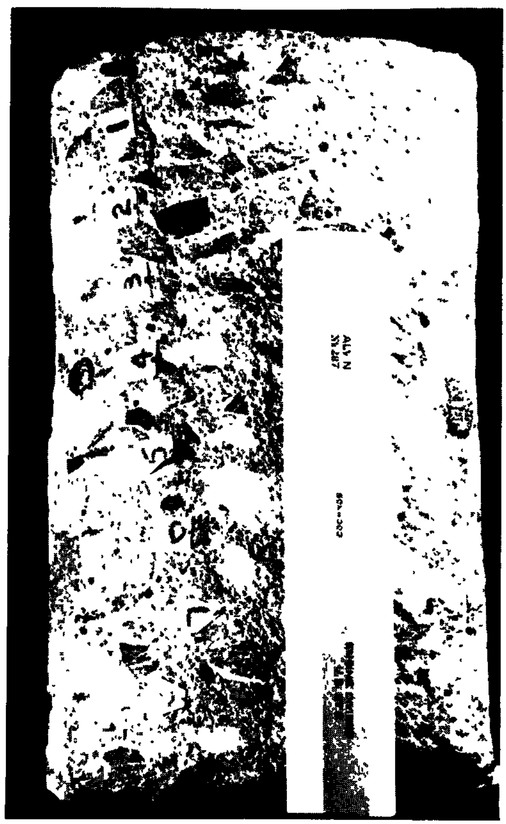


Figure 5. Pavement core from typical crack on York project.

Tables 5 and 6 show the average crack readings obtained by microscope and gage for the Hamburg project. These have been averaged from the first 500 ft, the middle 400 ft and the end 500 ft of each pavement depth in all four lanes, comprising a total of 36 test areas. Invar gage readings have been obtained from a 100-ft section within each of these 36 areas. Average widths for each pavement thickness can be summarized as follows:

Pavement Thickness (in.)	Crack Width (in.) Microscope	Invar Gage
7	0.011	0.015
8	0.010	0.014
9	0.012	0.017

Corresponding crack width data for the York pavement are given in Table 7. Average width readings range from 0.015 to 0.22 in. and, for the most part, are slightly greater than those obtained at Hamburg. This is to be expected, inasmuch as the York pavement has the greater crack spacing.

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MICROSCOPE WIDTH READ

Pav't. Age	7-In. Pavement						8-	
	Beg.		Middle		End		Beg.	
	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)
(a) Eastbou								
2 mo.	74	0.008	67	0.014	67	0.009	70	0.009
3 mo.	86	0.007	86	0.009	86	0.007	86	0.009
6 mo.	58	0.009	58	0.009	54	0.007	64	0.006
1 yr	65	0.010	65	0.008	65	0.007	64	0.008
1½ yr	58	0.010	58	0.008	56	0.009	56	0.009
(b) Eastbou								
2 mo.	88	0.007	88	0.007	88	0.012	88	0.008
3 mo.	74	0.017	74	0.012	74	0.009	78	0.008
6 mo.	36	0.017	36	0.015	36	0.016	40	0.015
1 yr	70	0.006	78	0.006	78	0.008	78	0.009
1½ yr	50	0.008	50	0.012	50	0.012	60	0.008
(c) Westbou								
2 mo.	82	0.008	85	0.009	88	0.007	87	0.006
3 mo.	80	0.007	78	0.007	78	0.007	78	0.008
6 mo.	42	0.011	56	0.008	70	0.009	70	0.009
1 yr	88	0.009	88	0.007	81	0.008	80	0.007
1½ yr	76	0.008	76	0.014	60	0.011	65	0.010
(d) Westbou								
2 mo.	88	0.007	88	0.009	74	0.008	84	0.007
3 mo.	78	0.007	78	0.008	78	0.007	82	0.008
6 mo.	58	0.008	58	0.008	58	0.012	61	0.009
1 yr	87	0.006	87	0.008	87	0.011	87	0.011
1½ yr	68	0.012	82	0.010	82	0.011	82	0.010

End movement on both projects has not been extensive. Measurements have been obtained from gage plugs originally installed 10.0 in. apart on each of the finger-type joints in each lane. At Hamburg, the original measurement was taken at 82 F; one year later, at the same temperature, a ⅝-in. average decrease in length was noted at each end. After 1½ yr, at 37 F, an increase of ¼ in. at each end was recorded. Two subsequent readings from the York project at fairly equal temperatures indicated an average increase in length of 7/16 and ⅜ in., respectively. The readings were taken at 15 and 16 months after installation of the plugs.

EFFECT OF PAVEMENT THICKNESS

The latest survey indicates a definite relationship between the depth of pavement and the resulting crack pattern. The difference was less noticeable between the 7-in. and 8-in. pavements than between the 8-in. and 9-in. sections. Table 8 gives the

HAMBURG PROJECT

vement			9-In. Pavement					
dle	End		Beg.		Middle		End	
Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)
side Lane								
0.011	70	0.008	-	-	80	0.005	80	0.007
0.007	88	0.008	-	-	86	0.006	86	0.010
0.010	64	0.010	-	-	47	0.009	42	0.015
0.007	64	0.006	-	-	72	0.006	72	0.007
0.009	66	0.010	-	-	66	0.011	58	0.014
de Lane								
0.007	90	0.008	-	-	84	0.007	84	0.009
0.011	78	0.008	-	-	88	0.009	-	-
0.016	40	0.015	-	-	35	0.017	39	0.022
0.009	81	0.008	-	-	79	0.012	80	0.011
0.011	60	0.009	-	-	54	0.010	55	0.016
ide Lane								
-	86	0.008	86	0.009	87	0.008	80	0.007
0.008	72	0.008	78	0.011	80	0.009	80	0.008
0.013	64	0.014	63	0.013	63	0.010	58	0.009
0.009	80	0.013	80	0.013	82	0.010	82	0.008
0.010	73	0.011	73	0.013	73	0.011	76	0.008
de Lane								
-	74	0.012	78	0.008	72	0.008	80	0.007
0.008	86	0.013	78	0.010	68	0.012	74	0.008
0.009	70	0.008	70	0.009	68	0.012	68	0.009
0.007	87	0.007	91	0.008	91	0.011	91	0.008
0.010	80	0.009	72	0.011	72	0.011	72	0.013

average number of cracks found in each pavement thickness comprising all four traffic lanes. Further reference is made to Table 3, where a more complete breakdown can be found.

EFFECT OF SUBBASE DEPTH

At this time, the Hamburg project indicates that an increase in the thickness of subbase does not tend to reduce substantially the number of cracks in any particular pavement thickness. In fact, on this project, where various depths of subbase material were placed, there tend to be fewer cracks in the pavement over the thinner depths of subbase. During the placing of the subbase, depth measurements were taken every 50 ft throughout the entire project. Thirty-four areas of 400- to 500-ft lengths, and of contrasting depths, were selected for study. These results are given in Table 9.

Generally, where subbase is less than 5 in. thick the pattern averages 11.4 cracks per 100 ft; where between 5 and 7 in., 12.5 cracks per 100 ft; and where greater than 7 in., 12.7 per 100 ft.

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INVAR WIDTH READI

Pav't. Age	7-In. Pavement						8	
	Beg.		Middle		End		Beg.	
	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)
(a) Eastbou								
6 mo.	50	0.013	50	0.014	50	0.018	50	0.014
1 yr	65	0.009	65	0.012	65	0.016	64	0.012
1½ yr	58	0.013	58	0.015	58	0.020	56	0.016
(b) Eastbou								
6 mo.	36	0.018	36	0.018	36	0.016	40	0.018
1 yr	84	0.014	84	0.015	84	0.014	84	0.016
1½ yr	50	0.015	50	0.021	50	0.017	60	0.018
(c) Westbou								
6 mo.	42	0.013	70	0.008	70	0.011	72	0.009
1 yr	92	0.012	92	0.008	92	0.014	92	0.012
1½ yr	76	0.012	76	0.009	76	0.015	65	0.013
(d) Westbo								
6 mo.	58	0.007	58	0.012	58	0.011	64	0.010
1 yr	92	0.009	92	0.013	92	0.011	92	0.011
1½ yr	68	0.012	68	0.014	68	0.012	68	0.012

TABLE 7

MICROSCOPE WIDTH READINGS, YORK PROJECT

Lane	Pavement Age	Beg.		Middle		End	
		Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)
North outside	1⅔ yr	78	0.018	76	0.017	80	0.023
	2 yr	55	0.017	55	0.016	59	0.020
North inside	1⅔ yr	80	0.018	82	0.020	81	0.024
	2 yr	57	0.023	56	0.026	59	0.019
South outside	1⅔ yr	76	0.024	80	0.013	87	0.012
	2 yr	60	0.027	59	0.018	60	0.015
South inside	1⅔ yr	78	0.015	84	0.011	85	0.014
	2 yr	60	0.021	59	0.012	60	0.017

IAMBURG PROJECT

Pavement			9-In. Pavement					
ddle	End		Beg.		Middle		End	
Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)	Air Temp. (°F)	Crack Width (in.)
tside Lane								
0.017	54	0.020	-	-	52	0.013	52	0.018
0.016	64	0.018	-	-	72	0.011	72	0.012
0.017	56	0.021	-	-	66	0.014	58	0.023
ide Lane								
0.016	40	0.015	-	-	35	0 018	42	0.022
0.012	84	0.011	-	-	91	0.010	91	0.018
0.014	60	0.015	-	-	54	0.020	54	0.026
side Lane								
0.012	64	0.011	63	0.011	63	0.012	54	0.008
0.013	92	0.012	92	0.012	92	0.013	92	0.011
0.014	65	0.013	73	0.013	73	0.013	76	0.012
ide Lane								
0.008	64	0.008	70	0.010	68	0.012	68	0.008
0.006	87	0.009	91	0.009	91	0.013	91	0.012
0.010	68	0.010	72	0.008	72	0.014	72	0.015

WIDTH-TEMPERATURE CURVES

TABLE 8

EFFECT OF PAVEMENT THICKNESS

Pavement Thickness (in.)	Length of Section (ft)	Total No. Cracks	Avg No. Cracks/ 100 ft	Avg Spacing Between Cracks (ft)
7	1,999	1,062	13.3	7.7
8	4,366	2,209	12.7	8.0
9	4,441	1,772	10.0	10.6

Measurements were taken over a 24-hr period in April 1958 to observe the relationship of crack width to temperature. The internal temperature of the pavement ranged from 51 F to 79 F; the corresponding air temperature for the same period was 38 F to 75 F. The resulting curves are shown in Figure 6. Readings were obtained from eleven cracks from each of the westbound lanes in the 70 in. pavement. Although the data are not of sufficient scope to make any broad conclusions, it nevertheless is interesting to note the resulting pattern. The maximum crack opening occurs around 6:00 a.m. and for the 12-hr period between 9:00 a.m. and 9:00 p.m. there is little change in crack opening. This was characteristic of a sunny spring day and it is entirely possible that other factors and conditions would produce a different curve. The greater degree of variation observed in the inside lane may have resulted from an average 10-deg higher placing temperature over the outside lane. An extension of these observations over a 1-yr period covering all types of weather conditions would no doubt provide much interesting information.

PAVEMENT REPAIR, HAMBURG PROJECT

Shortly following the completion of the Hamburg project, generally within three months, it became evident that a number of cracks were becoming unusually wide and were serious enough in nature to warrant careful observation. With the approach of colder weather the cracks assumed much wider proportions, some of them approaching ¼ to ½ in., measured at the surface (Fig. 7). A number of these cracks developed in scattered locations throughout the project, and in the fall of 1957 a small-scale investigation was conducted, consisting of the removal of several cores at these particular cracks. The results of this survey indicated that in some locations the bar mat reinforcement had not been lapped sufficiently (2).

In the spring of 1958 a full-scale study was made of 15 such areas, involving the removal of a section of concrete approximately 6 in. in width and 4 ft in length at the edge of the pavement at each crack location. The pavement was removed only to the depth of the reinforcing steel. The study revealed that at three locations there was a definite lack of reinforcing overlap. The bar mats were required to be overlapped 12 in., but failed to meet by ⅜ in. at Sta. 228 + 85, by 9¾ in. at Sta. 254 + 46 (Fig. 8), and up to 12 in. at Sta. 269 + 93. Of the remaining twelve cracks, two were found to occur somewhere in the midsection of the bar mat. Fortunately, these cracks were far less serious in nature and will probably offer no further difficulty. The average lap of steel in the remaining ten areas was 7½ in., ranging from 3 to 11 in. A typical situation is shown in Figure 9, where the lap was found to be 8 in.

TABLE 9

EFFECT OF SUBBASE THICKNESS

Pavement Thickness (in.)	Lane	Subbase Thickness (in.)	Number of Cracks	Cracks per 100 ft
7	Eastbound outside	8.5	69	13.80
		8.0	73	18.25
		6.5	82	16.40
	Eastbound inside	7.5	51	10.20
		8.0	57	14.25
		7.5	55	11.00
	Westbound outside	7.5	55	11.00
		8.0	55	13.75
		7.9	76	15.20
	Westbound inside	5.0	47	9.40
		6.0	50	12.50
		7.5	60	12.00
8	Eastbound outside	6.5	66	13.20
		8.5	49	12.25
		4.0	56	11.20
	Eastbound inside	6.5	55	11.00
		8.0	48	12.00
		3.0	58	11.60
	Westbound outside	9.0	64	12.80
		4.0	62	15.50
		7.5	66	13.20
	Westbound inside	7.5	51	10.20
		4.0	47	11.75
		7.0	54	10.80
9	Eastbound outside	4.5	39	9.75
		8.0	23	4.60[a]
	Eastbound inside	3.5	44	11.00
		9.0	28	5.60[a]
	Westbound outside	8.0	59	11.80
		8.0	59	14.75
		4.0	56	11.20
	Westbound inside	7.0	68	13.60
		7.5	41	10.25
		4.5	45	9.00

[a] Paved in October.

It is perhaps significant, that 67 percent of the serious cracking occurred in the westbound lanes, and a total of 73 percent of the cracking was in the 9-in. pavement. The crack locations are shown in Figure 10. Although it may be possible to attribute the excessive cracking in the westbound 9-in. pavement to the extremely high temperatures during which it was placed, it is generally believed that the cracking in this area was induced by the same factor which caused the cracking throughout the project—namely, improper overlap. Some thought was given to the belief that strike-off and finishing equipment might have dragged these bar mats forward, but several investigations 16 ft beyond a crack in the direction of paving clearly indicated no double length of overlap, as might have been expected. The specifications and proposal covering this project did not require the bar mats to be tied. It is now apparent that this requirement should have been specified, and certainly should be written into future contracts involving continuously-reinforced pavements.

After determining the cause of the cracking, arrangements were made to repair these areas. It was essential that this work be performed carefully; to insure optimum results, specifications were written covering every detail of the operation. These concrete pavement repairs, within the areas designated, consisted of removal of the existing pavement, restoration of the continuity of the longitudinal reinforcing steel, and replacement of the pavement with high early strength cement concrete. All work was done in accordance with the requirements of the paving section of the Department's Specifications, Form 408, with additional special requirements.

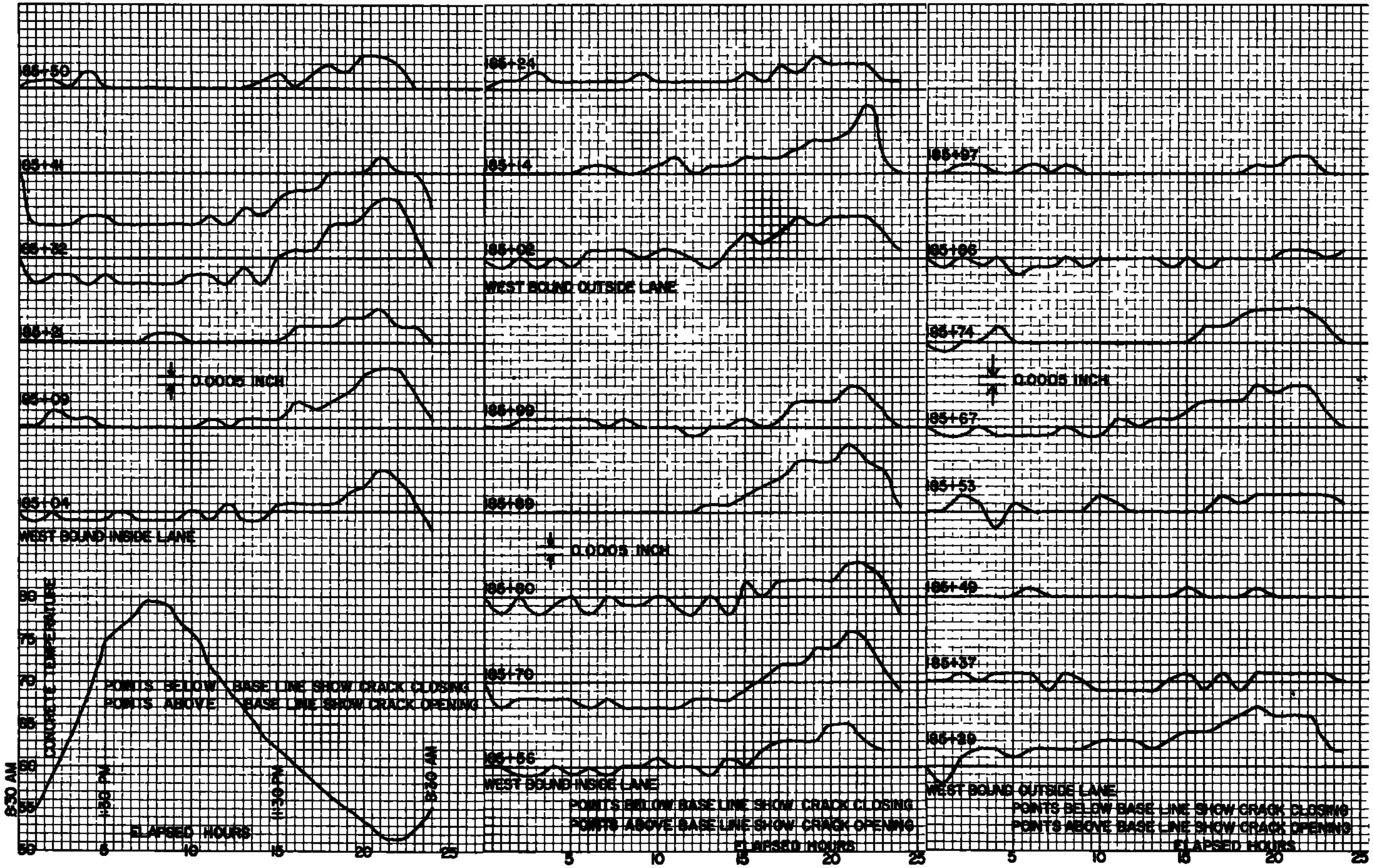

Figure 6. Hourly changes in crack widths.

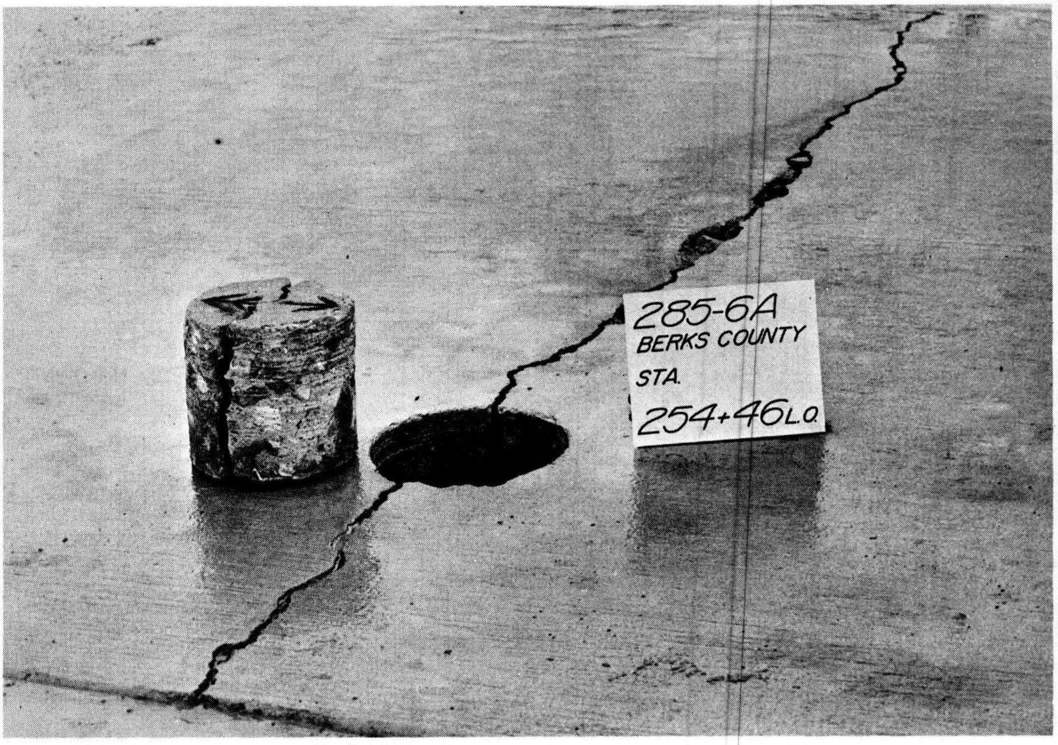

Figure 7. Typical wide crack.

Figure 8. Gap in reinforcing bars.

Thirteen such areas to be repaired were located as follows: eastbound outside lane, Sta. 222 + 46 and 245 + 56; eastbound inside lane, Sta. 201 + 79 and 255 + 09; westbound outside lane, Sta. 186 + 94, 233 + 51, 247 + 92, 254 + 46, 267 + 96, and 269 + 93; westbound inside lane, Sta. 228 + 85, 259 + 46, and 264 + 49.

Materials

Concrete was designed, proportioned and mixed in accordance with Department Specifications, and Type D, high early strength, air-entraining portland cement was required. Aggregates were from the same source and of the same quality as those used in the original pavement. Steel reinforcement identical with that in the existing pavement was required.

Repair Procedure

Throughout the patching operation traffic was diverted from both lanes of one side, so that all repairs on one side could be completed at the same time. Traffic flow was then reversed, allowing repairs to be made on the opposite side.

The pavement repairs were performed by the contractor between September 8 and September 29, 1958. The weather was quite favorable for the replacement of the nine areas on the westbound lanes, and there were wide fluctuations in temperature while four patches in the eastbound lanes were being made.

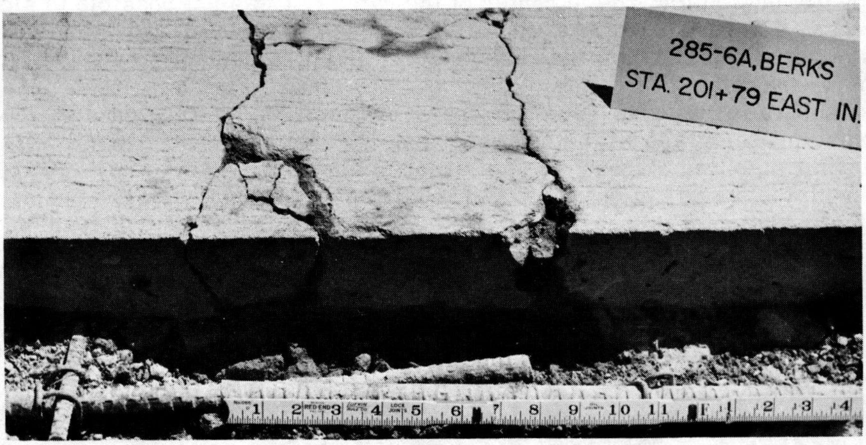

Figure 9. Steel bars lapped 8 in.

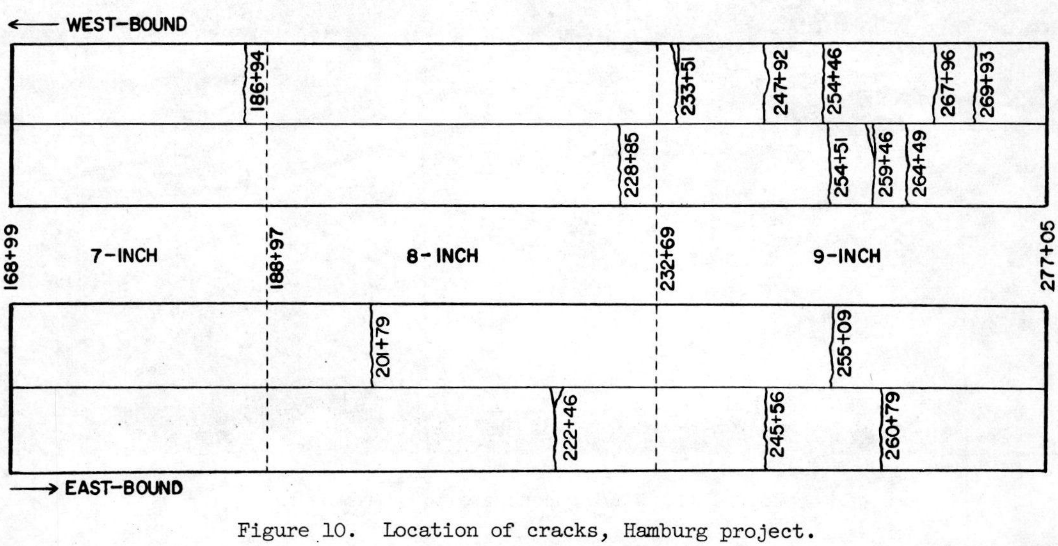

Figure 10. Location of cracks, Hamburg project.

The areas of concrete to be removed in making the pavement repairs were a minimum of 6 ft in length for a full lane width (12 ft). Generally, concrete had to be removed for a distance of at least 3 ft on each side of the abnormal transverse cracks. In no instance could the patch terminate within 4 ft of any existing normal crack. The patches could be shifted slightly to avoid adjacent cracks or, if this were not possible, they were lengthened to include adjacent cracks.

The pavement was cut with a saw to a depth of 2 in. at both ends of each patch to form a neat joint at the junction of old and new concrete. A saw cut to a depth sufficient to cut through the steel reinforcing bars was required at a distance of 21 in. from each end of the patches (see Fig. 11). The portion of concrete and steel in the center of the patch could then be removed without disturbing the existing concrete and steel at the ends of the patches. The remainder of the concrete was then broken away from the steel bars with extreme care. All chipping and breaking of concrete was accomplished with pneumatic air hammers. This was a slow process, requiring care to prevent damaging the steel. This was difficult to do, and in several patches in the westbound lanes an occasional bar was broken. It was not believed however, that this damage would measurably affect the performance of the reinforcing steel. All broken pieces were butt-welded in place.

In forming the joints at the ends of the patches, the vertical face of the pavement below each sawed edge was carefully broken to prevent undercutting the joint and was sloped slightly inward toward the center of the patch. It was not possible to slope the vertical edges toward the center of the patch as much as desired, but the resulting joint was quite irregular below the 2-in. saw cut and provided good aggregate interlock with the new concrete.

Special subgrade material (subbase) wherever disturbed was restored to its original condition and proper grade with stone screenings.

Reinforcing steel was brushed free of excessive rust and placed and welded in accordance with the specifications (Fig. 12). Extensive laboratory tests were conducted prior to the selection of this particular method to insure that the steel would not be

Figure 11. Sawing concrete to be removed.

greatly weakened by welding. The ultimate stress was above the yield point of the steel in all of the specimens tested; however, the results obtained with this particular method were superior to all others. In repairing the steel, No. 5 deformed steel bars of the same quality as the existing steel were required for splicing and of sufficient length to provide an 18-in. overlap at both ends. The splice bars were placed against the existing protruding steel and positioned longitudinally to provide a 3-in. clearance between the ends of the bars and the joints at the ends of the patch. The steel was welded at the center of the lap with a $\frac{1}{2}$-in. bead, on one side only, for a length of 5 in., using General Electric W-616-A $\frac{5}{32}$-in. diameter electrodes. No. 3 transverse bars were placed and tied to the longitudinal bars in accordance with the pattern of the original bar mats.

It was specified that aggregates be stocked on or near the job site and stored and handled in accordance with methods required by the Department Specifications. Use of approved portable scales for batching aggregates was permitted and bagged cement was specified. Mixing was accomplished at the job site using a mixer of at least 14-S capacity. Use of ready-mixed concrete was not approved.

Prior to placing the concrete, both edges of each patch were painted with an epoxy resin adhesive. The concrete mix was designed at $1\frac{1}{4}$-in. slump and 3.0 percent air. Proportioning was accomplished with portable scales on the job site using aggregates stocked at each patch area. A 14-S paver was used to mix the concrete, which was wheeled to the patch in hand buggies. The average slump obtained was $1\frac{1}{2}$ in. and air entrainment averaged 4.2 percent. A vibrator was used to insure good placement of concrete around the steel, and the usual finishing practices were followed. A completed patch is shown in Figure 13.

A coat of an epoxy resin sealer was sprayed on the surface of the patches to delay the loss of moisture, and curing was effected with a blanket of straw between double layers of saturated burlap. Flexural beams, cast at the time the concrete was placed, were cured in the same manner as the concrete and tested in accordance with Department procedure. The concrete was cured for five days, at which time flexural strength

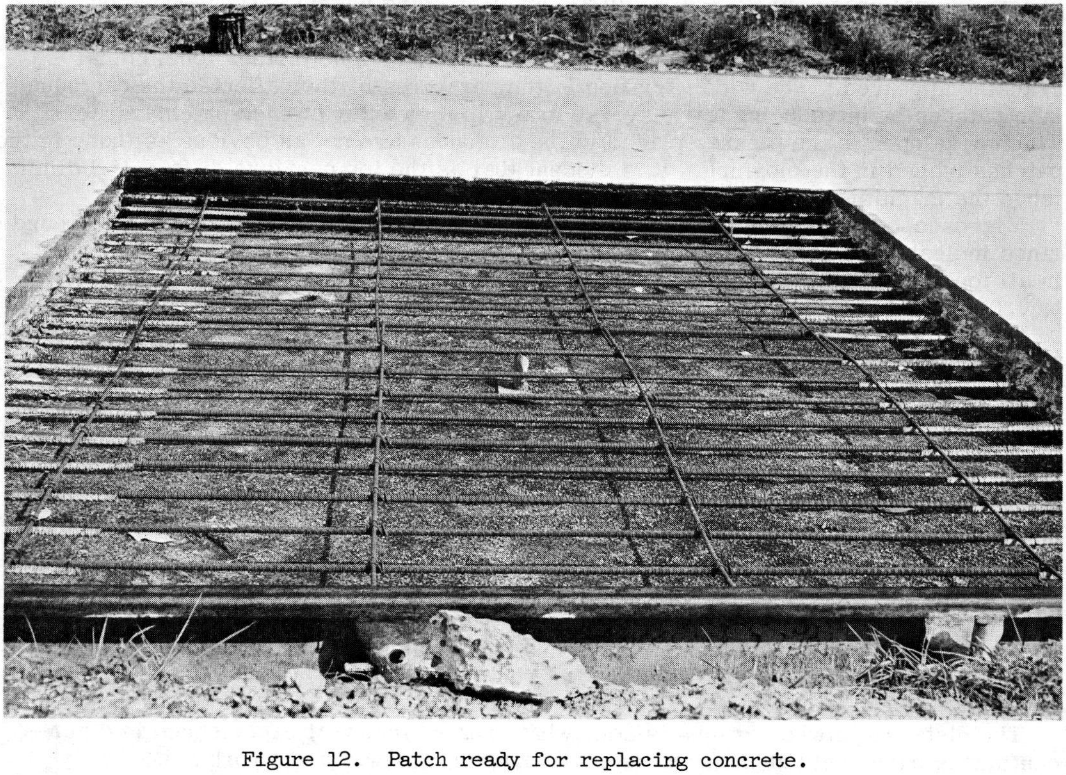

Figure 12. Patch ready for replacing concrete.

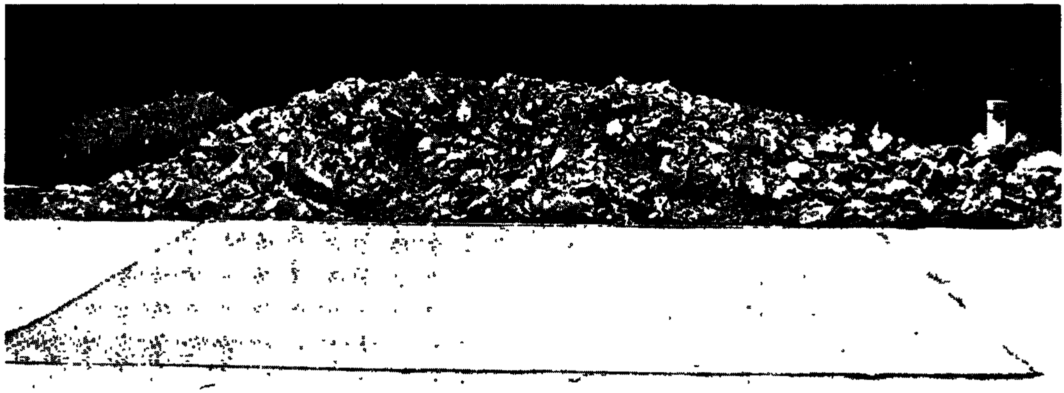

Figure 13. Completed patch.

developed to 615 psi for patches placed in the westbound lanes and to 700 psi for patches in the eastbound lanes. All traffic was excluded from the patches for the full curing period.

A wide contrast existed between the separate lanes with respect to variations in air temperatures. While the westbound lanes were being placed and cured, temperatures were mild and fluctuated within a range of 10 to 15 deg. As a result, an excellent repair was made and after two months of observation no cracks had developed. There were, however, microscopic, superficial cracks at the joints where the mortar was brushed onto the old pavement. During the placing of the concrete in the eastbound lanes greater variations in temperatures occurred, the maximum approaching 35 deg. This had a definite influence on the patches being placed, as evidenced by the formation of longitudinal cracks at Sta. 201 + 79 2½ hr after being placed. These were on the outside edge and were immediately above the steel bars. This portion of the patch was removed down to the steel and replaced again, after which no further longitudinal cracking appeared. The wide variation in temperatures further affected the eastbound lane patches by introducing a transverse crack in the center of each patch. Cracks which developed in the patches placed in the afternoon are not as obvious as those in the patches placed in the morning. It is evident that as the temperature differential diminished the magnitude of the crack also lessened.

Microscope readings of the tiny cracks at the joints of the patches on the westbound lanes indicated a range of from 0.002 to 0.012 in. The transverse cracks appearing in all four patches in the eastbound lanes ranged from 0.020 in. at Sta. 201 + 79 down to 0.008 in. at Sta. 255 + 09.

Careful workmanship and close observance of the specifications written for this work resulted in neat patches blending into the existing pavement with a minimum of contrast. Use of the epoxy resin adhesive helped to some extent in bonding the concrete. It was encouraging to note the absence of any cracks in patches placed under favorable weather conditions. The cracks that did occur were expected, and are certainly a tremendous improvement over the previous ones. They are within the range of the other normal cracking on the project, and should offer no further trouble. It appears that areas of failure on a continuously-reinforced pavement have been repaired successfully.

SUMMARY

The two continuous pavements placed in Pennsylvania under different weather conditions in opposite seasons of the year reacted quite differently with respect to crack patterns.

The distance between cracks is somewhat greater than that experienced in other continuous pavements at the same age. Average crack widths at York are slightly greater than those obtained at Hamburg; correspondingly, the York pavement has the greater crack spacing.

The crack pattern is formed early in the pavement life by concrete shrinkage and stresses caused by the steel. Traffic has apparently had little effect in causing additional cracks to form on these two projects.

Data from the Hamburg project indicate that when the percentage of steel is kept constant there is a slight decrease in cracking as the pavement thickness varies from 7 to 9 in. An increase in the depth of subbase material did not actually reduce the number of cracks in the Hamburg pavement.

REFERENCES

1. Witkoski, F.C., and Mattson, W.E., "Preliminary Report on Continuously-Reinforced Concrete Pavement." Report No. 1-56-38, Pennsylvania Department of Highways.
2. Witkoski, F.D., and Shaffer, R.K., "Continuously-Reinforced Concrete Pavement in Pennsylvania." HRB Bull. 214, p. 80.
3. Taylor, I.J., and Eney, W.J., "First-Year Performance Report on Continuously-Reinforced Concrete Pavements in Pennsylvania." HRB Bull. 214, p. 98.

Discussion

BENGT F. FRIBERG, Consulting Engineer, St. Louis, Missouri — The four experimental continuously-reinforced 24-ft pavements in Pennsylvania (two on the York project each 11, 559 ft long, paved September-October 1956, and two on the Hamburg project each 10, 806 ft long paved during May and June 1957, except for one gap of 1, 795 ft which was paved in October 1957) are an interesting and informative addition to earlier experimental projects of the type. The substantial mileage and varied seasonal construction conditions, from May to October, give fairly representative seasonal fluctuation to the data for 0. 5 percent longitudinal steel. The close relation between construction air temperature and transverse cracking during the first year is clearly illustrated. The frequency at one year varied from a low of 5 cracks per 100 ft for pavement placed between 48 and 56 F, to more than 15 cracks per 100 ft for construction above 85 F, air temperature. Considering both the York and Hamburg projects, the cracking per 100 ft at one year age averaged 5 to 6 cracks in pavement placed below 60 F, about 10 cracks for 70 F, and about 14 cracks for over 80 F construction air temperature.

The authors should be complimented on their careful comparison of crack width measurements with microscope and with invar gage. The microscope apparently included a glass with etched scale showing 1-mm (0. 04-in.) divisions, read to $\frac{1}{10}$ division; the invar gage was a direct-reading dial-equipped gage, read in $\frac{1}{1000}$ in. The average difference in reading 0. 004 in., or $\frac{1}{10}$ division on the microscope, shows good agreement under the circumstances, with the invar gage reading by far the more accurate measurement. Given a sufficiently close reading microscope, such instruments should give usable measurements, as stated by the authors.

The particular stretch on the Hamburg project (Table 4) with invar gage plugs placed at time of construction was built in October 1957 at an approximate air temperature of 50 to 60 F. After the concrete cracks, some warping should normally develop at the cracks, especially at early age in drying concrete. The invar gage measured changes of a 10-in. length $\frac{1}{4}$ in. below the concrete surface. The measurements include an increment of width due to slopes of the warped concrete. Referring to measurements of crack widths (Table 4) for the eastbound outside lane, in which all cracks from Sta. 241 to 242 were apparently measured, the average crack width at 1 month of 0. 011 in. at 54 F air temperature is probably to a substantial extent warping, with the crack width at the bottom much smaller. In late fall warping would normally decrease, apparent as an incremental decrease in crack widths measured at the surface, but crack widening is probably not obscured by compression restraint across the cracks. During that season between 1- and 3-month age the total crack width per 100 ft (Table 4) changed from 0.089 in. at 54 F to 0.133 in. at 42 F. Lacking concrete temperatures, no close comparison can be made between observed and theoretical crack width; however, if the air temperatures are considered equal to concrete temperatures, the observed changes in crack width are about 75 percent of the theoretical contraction. Most of the observed contraction took place at one or two of eight cracks observed. The measurements in Table 4 at 6 months 71 F, and 1 year 78 F, undoubtedly were made when the pavement was in compressive restraint. The data show that cracks do widen during the cold season.

The foregoing observations are in accord with theoretical studies *, which show (for 0.5 percent reinforcement, n = 7) that a crack width 75 percent of theoretical would correspond to about 0.09 ratio between active bond length and crack spacing, or about 13 in. on each side of the 8 cracks at 3 months, which (Fig. 11 of the reference) would correspond to a steel stress increment of about 1,300 psi per deg temperature drop, and a concrete stress of 6 psi per deg some 16,000 and 70 psi, respectively, for the temperature drop from 54 to 42 F.

Exceptionally predominant changes in width of some cracks are possibly more indicative of actual tension restraints than average crack widths, given in Tables 5, 6, and 7. Frictional subgrade drag between cracks is very small; forces at adjacent cracks therefore must balance each other substantially. A wide crack indicates a low rate of bond development with slip, and possibly bond failure, at that crack; adjacent effectively tied cracks could then not be expected to change appreciably in width because of the relatively low restraint force necessary at those cracks. That large variations have occurred is indicated in the text, and by the reports of failed cracks. The average crack widths in Table 7 for the York project, placed in September-October, have been measured at or above construction temperatures, with the pavement undoubtedly in compressive restraint except near the ends. Under compressive restraint average crack widths would not be expected to show substantial width variation, except as connected with changes in warping.

The authors' gage measurements of crack widths during a 24-hr period of sunny April weather on the 7-in. westbound lanes (Fig. 6) throw interesting light on pavement restraint conditions. The station covered is far from any end, but an open crack had been discovered in the outside lane 94 ft east of the end of the outside lane station. In the inside lane 11 cracks were measured out of 13 to 15 cracks; in the outside lane, 11 of 12 cracks indicated in Figure 4. In Figure 14 the measured changes in crack width totaled for the 100 ft of each lane are plotted in relation to time, as is the concrete temperature. The cracks show no significant change in surface width from 9:30 a.m. at 55 F internal (assumed to be average) temperature to 10 p.m. at 65 F. From that time till 6:00 a.m. the surface width increases at a fairly even rate, less in the outside than inside lane. Figure 15 shows the measured fluctuation in width of individual cracks. Near the west end measured changes are nearly alike in both lanes; the difference between the lanes is the result of decreasing change in width of cracks toward the east end of the outside lane, about 100 ft from the crack at which steel bond apparently had failed.

For sunny April weather normal temperature gradients could be expected to reach 3 F per in. depth during the forenoon, maximum near noon, and reversed 1 F per in. during the evening hours. Approximate values of top and bottom temperatures accordingly are estimated to have been at 9:30 a.m. about 62 and 48 F, at 10 p.m. 62 and 70 F, respectively. The period during which no change in crack surface width was observed accordingly corresponded to above 62 F approximate surface temperature, both at 9:30 a.m. and 10 p.m. Above about 62 F surface temperature apparently the surface of the pavement was in compressive restraint across the cracks, and obviously without measurable change in crack width. The concrete surface probably reached a temperature near 80 F, and the concrete restraint compression at the surface may

* Friberg, B. F., "Frictional Resistance under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs." HRB Proc., 33:167 (1954).

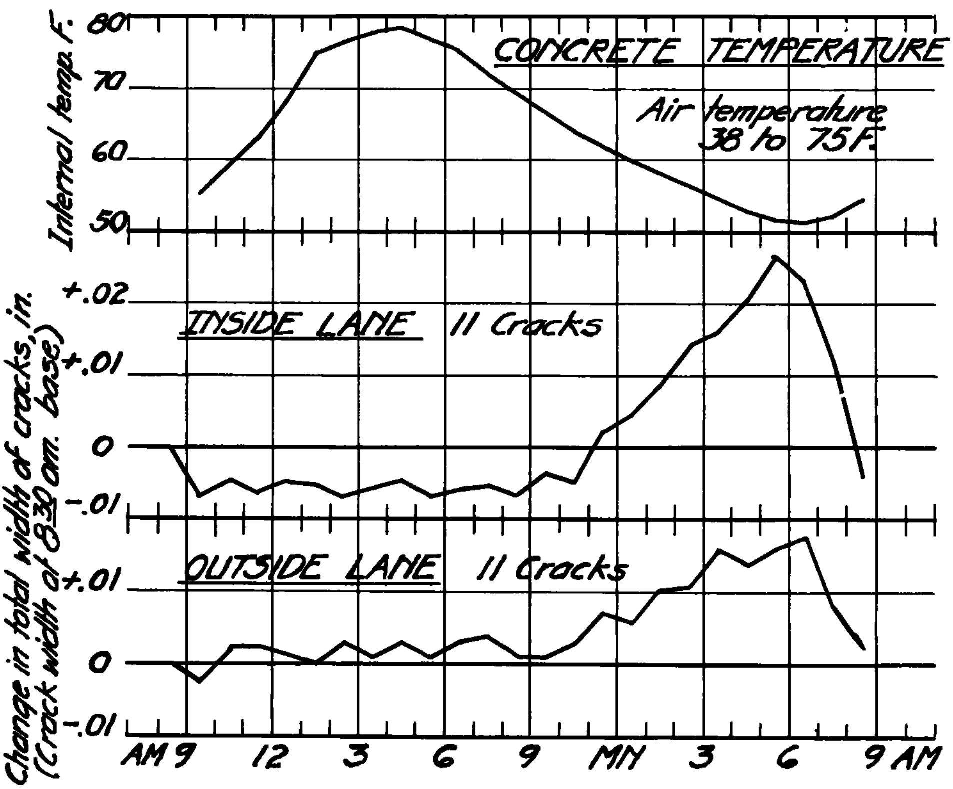

Figure 14. Concrete temperature and surface-width fluctuation of cracks in 100 ft of 7-in. pavement 9 months old during 24 hr of April weather.

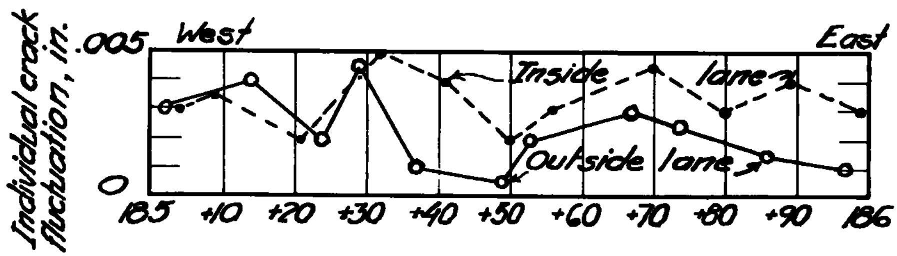

Figure 15. Total fluctuation in surface width of each crack between Stas. 185 and 186 during the 24-hr period. A wide crack existed in the outside lane at Sta. 186+94.

well have exceeded 300 psi. During the hours of restraint the compression stress strain compensated for most expansion due to temperature gradient through the slab, probably without noticeable curling.

During the night, from 10 p.m. with 62 F at the surface and nearly 70 F at the bottom, to 6 a.m. with slightly under 50 F at the top and about 55 F at the bottom, crack widening obviously could occur at the surface; however, near the bottom compressive restraint could have persisted much later than 10 p.m. The surface widening measured after 10 p.m., during the early part of the night period at least, could be due to slope change incident to curling, rather than to uniform crack widening. The crack widening measured between 3 and 6 a.m. seems to have occurred at a greater rate of change in the inside lane, not much less than that corresponding to unrestrained contraction, or about 0.005 in. per degree. The explanation could be that after 3 a.m. there was no compressive restraint left anywhere across the cracks (concrete temperature at bottom 60 F or less); crack widening in the fully restrained inside lane followed normal development, with steel in tension strain at cracks elongating nearly equal to the temperature contraction.

In the outside lane, on the other hand, relatively free movement at a wide crack 100 ft away prevented full restraint because tension was limited to that caused by frictional subgrade drag under slab portions to the wide crack, and as a result slab displacement could have occurred near the east end in that lane as contraction toward the west rather than as crack widening. Near the west end, more than 170 ft from the wide crack, tension restraint conditions were apparently nearly alike in both lanes, as is indicated by equal changes in individual crack widths in the two lanes. The integrated differences in movement between the two lanes is only about 0.015 in. (Fig. 15). The friction forces, accordingly, could not be very large, just as the tension restraint forces in the steel at cracks probably would not be large for the modest temperature drop.

The measurements between 6 and 9 a.m. show fairly well that curling had a major influence on width measurements. The temperatures probably changed from about 50 F top and 55 F bottom at 6 a.m. to about 60 F top and 50 F bottom at 9 a.m., with little change in average temperature, yet the crack measurements changed from maximum width to nearly the constant value indicating restraint at the surface in that short interval. Although the foregoing explanation is hypothetical on many points, it is not contradicted by the measurements, and provides explanation for observations which might not otherwise be explained.

This discussion is intended to show deductions which can be suggested from the careful research done by the authors. The authors should be complimented highly for their enterprising and objective pursuit of observations, which can be expected ultimately to explain adequately the behavior of continuously-reinforced pavements. It is hoped that the representative daily cycles of crack measurements may be extended to lower temperatures, and to include measurements of temperature gradients and curling slope changes at the cracks.

Observations on the Behavior of Continuously-Reinforced Concrete Pavements in Pennsylvania

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This paper summarizes the results of three years of sponsored research on continuously-reinforced concrete pavements as conducted by the Fritz Engineering Laboratory of Lehigh University.

Test data and conclusions based on instrumentation, physical measurements, and observations of two current pavement projects are reviewed, and a general pattern of behavior for continuous pavements is established.

Some of the weaknesses found in existing pavements are described and given consideration in suggestions for design and construction improvements in continuous pavements.

● THE FIRST continuously-reinforced concrete pavement in Pennsylvania was constructed in October 1956, and a second pavement was completed in July 1957. The construction details, instrument installations, and early behavior of these test pavements have been described in earlier reports (1, 2).

Through the joint efforts of the Testing Laboratory of the Pennsylvania Department of Highways and the Fritz Engineering Laboratory of Lehigh University, a considerable amount of information has been obtained from these projects. Much of this is in the form of data collected periodically by physical measurements at the pavement surface, and electronic instrument readings taken from transducers within the concrete, the granular base course, and the supporting soil.

At regular 30-day intervals during the 26 months from October 1956 to November 1958, gage data have been collected from a single instrumented test section in the pavement constructed in 1956. The same schedule of data collection has been maintained for 16 months on the second pavement, where four instrumented test sections are under observation. With each test section providing 40 gage readings, approximately 3,600 separate measurements have been obtained from these five gaged sections in the two test pavements.

In addition to the gage readings at the instrumented sections, a series of other measurements has been made in order to provide a record of the behavior of both pavements throughout their entire 2-mi length. Crack surveys, end movement measurements, surface roughness recordings and other informative data have been collected.

Several tests on continuously-reinforced pavements and similar concrete structures had been conducted prior to the test program in Pennsylvania. The information available from these tests was studied in order to anticipate pavement behavior and to plan a practical measurement scheme which would provide the greatest amount of information with a reasonable number of installed gages and limited field testing personnel.

It is believed that a complete history of the phenonema associated with the formation and behavior of transverse cracks would allow a much better understanding of reinforced pavements and possibly provide specific design information. For this reason, a concentration of instrumentation was installed at a structurally weakened section in the pavement where a transverse crack would occur.

Shortly after the first pavement was constructed it was apparent that the measurement scheme selected was satisfactory and could add materially to the existing knowledge on continuous pavements. Some of the first results of the investigation seemed to be contrary to the generally accepted theories, but it was found that these results were not unreasonable and, instead, established an excellent quantitative relationship between reactions within the pavement, surface measurements, and general pavement behavior.

The results from the second pavement project and a series of closely controlled laboratory tests have added sufficient evidence to support these earlier findings.

Rather than presenting detailed test data from individual gage measurements, this report deals primarily with the general behavior of the pavements under test and the more conclusive information that is required for the establishment of design criteria for all continuous pavements.

In presenting this information, the authors do not wish to imply that additional investigation of such pavements is unnecessary. The refinements required for optimum design can be developed only by continued field testing and laboratory research.

PAVEMENT BEHAVIOR

There are five major force-producing and potentially damaging influences which act on all concrete pavements regardless of their design. The ultimate success of a pavement constructed to a particular design may be judged by its ability to resist or respond to these controlling influences with a minimum of distress and deterioration.

A better understanding of the mechanics, magnitude, and relationship of these influences in a continuously-reinforced concrete pavement will result if, at first, each of the five is considered as an individual occurrence.

Shrinkage

During the process of curing the concrete decreases in volume and increases in strength while bond develops between the concrete and the reinforcement. What is believed to be the result of this combined action is shown in Figure 1, where a single longitudinal reinforcing bar and the concrete within its effective influence area are considered.

Some shrinkage occurs in all of the concrete, but the volume reduction is not uniform throughout the entire mass. In shrinking, the concrete flows along the path of least resistance from areas of low tensile strength toward those of higher tensile strength. Some of the weaker areas may result from poorly mixed concrete, but more often they originate where minimum bond with the reinforcement offers the least resistance to shrinkage flow.

Shrinkage reduces the transverse cross-sectional area of the beam without developing any significant tensile stresses in the concrete. Because of this, a major portion of the total shrinkage occurs in this plane.

Longitudinal shrinkage of the beam is opposed by the base friction, end anchorage, tensile strength of the concrete, and any strain resistance of the reinforcement that may be transferred to the concrete through the developing bond.

Bond strength is not constant along the reinforcement but develops in a fairly regular pattern, reaching maximum and minimum strength at space intervals dictated by the

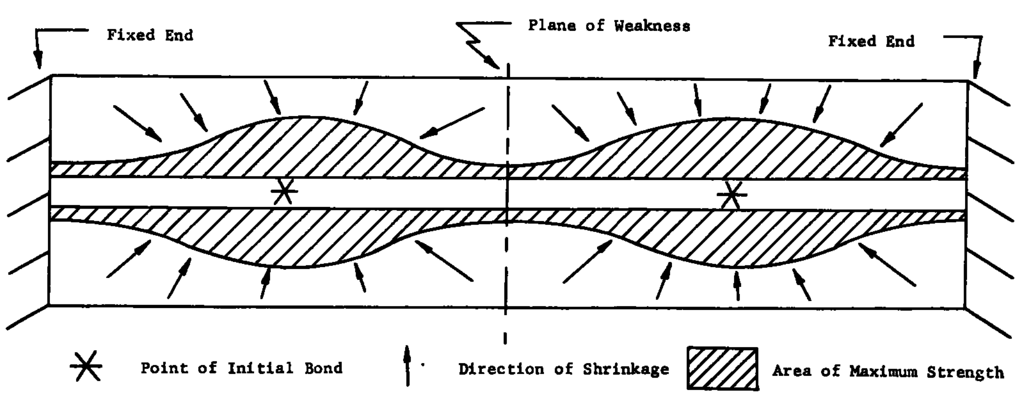

Figure 1.

combined influence of the concrete mixture, reinforcement perimeter, rate of curing, and base friction.

In areas where bonding with the reinforcement occurs, the tensile strength of the cross-section is increased in proportion to the bond strength. As the concrete continues to cure, shrinkage flow is toward these bonded areas and away from areas where the bond is less effective. This subjects the concrete between the better bonded areas to a pulling force which may exceed the tensile strength of the concrete and cause it to rupture.

Even if the tensile stresses do not develop sufficiently to cause rupture, the cured concrete within each affected area will contain a residual stress concentration maintained by the final bond with the reinforcement.

If the full width of a pavement is considered, the developed stress pattern becomes more complex. When several reinforcements are placed parallel in a pavement, a stress pattern similar to that shown in Figure 2 will result.

The spacing and magnitude of the residual stress concentrations along each individual reinforcement will vary, due to normally encountered minor inconsistencies in materials and construction procedures. Because each stress concentration originates independently, it is only when the increasing magnitude of the stress enlarges the area of influence that the stress pattern in an adjacent parallel reinforcement is affected. A shrinkage crack will occur where the random coincidence of longitudinal stress concentrations develops a transverse plane of weakness in the pavement and the combined tension forces exceed the tensile strength of the concrete at this plane.

Many of the numerous shrinkage-induced stress concentrations developed in a pavement lack the required force and/or the longitudinal coincidence of occurrence to cause transverse cracking. They remain as areas of constantly changing force potential, where cracks may occur if the pavement is subjected to the additional tension necessary to exceed the balance relationship between the concrete and the reinforcement.

Temperature

Temperature is the most formidable force to which continuously-reinforced pavements are subjected. It is responsible for constantly changing the conditions of stress and strain during construction and throughout the useful life of the pavement.

Because the pavement develops anchorage near each end, the center portion has no

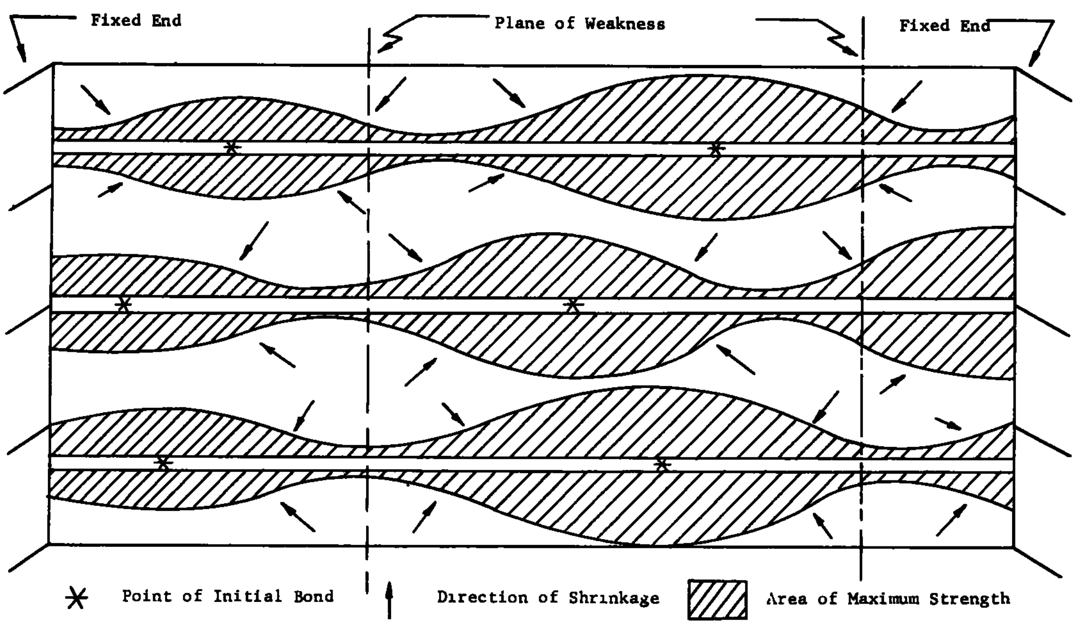

Figure 2.

opportunity for over-all longitudinal movement. A sudden decrease in temperature produces tensile strains which may exceed the elastic range and creep response rate of the concrete. When this occurs, a crack will develop along a transverse plane of weakness in the pavement. This crack will increase in width until the tensile force developed in the reinforcement at the crack exceeds the tensile strength of the concrete at another plane, and a new crack is formed.

As a crack pattern develops in the pavement, each individual section between existing cracks will respond as a single isolated unit with fixed ends. New transverse cracks will open in highly stressed areas and the number of separated units will be increased. This process will continue until the force of contraction is insufficient to cause further rupture of the concrete.

Under sustained stress, such as that resulting from the mean seasonal temperature, the concrete will creep. This reduces the possibility of new cracks and tends to transfer some of the stress from the concrete to the reinforcement.

When rising temperature causes the pavement to expand, compressive forces developed between the end anchorages may become very high. Most of this expansion is absorbed by creep and elastic straining of the reinforced concrete, but under extreme conditions the ends of the pavement may lose effective anchorage and be pushed outward. The extent of this pavement elongation, or "growth," is dependent upon several factors, and probably may be controlled by alterations in future pavement design.

Moisture

The initial water-cement ratio of the concrete affects the strength and durability of the pavement. Proper proportioning of ingredients can regulate and control these effects.

Free water or excessive capillary moisture beneath the pavement may result in heaving during very cold weather, causing extensive damage, but proper drainage and subgrade design can minimize this danger.

In addition to these more obvious and controllable effects of moisture, there are other subtle influences which are environmental in nature, significant in effect, and practically impossible to control.

Concrete is pervious to water, and will increase in volume when moisture is absorbed. Inversely, when moisture is removed, the volume of the concrete will decrease. This phenomenon will continue to influence the behavior of the concrete throughout the life of a pavement, and is independent of the initial shrinkage associated with curing.

In addition to its general influence upon the total longitudinal stress development in a continuous pavement, this moisture-induced volume change may, under some conditions, produce additional localized influences.

The bottom of the pavement, in direct contact with the subbase, is subjected to the constant moisture of the earth, whereas the top surface is intermittently subjected to rain and high humidity, or the drying effects of the sun and wind. Surface dryness causes an uneven distribution of moisture in the concrete, and results in a tendency for the pavement sections to warp and lift up at the cracked ends. Increased tension in the reinforcement and traffic wheel loads oppose the upward movement and tend to restrain the pavement in its original position. Because the pavement lacks base support at the warped ends, this increased tension may cause additional cracks to occur.

Warping may account for the occasional occurrence of a second crack near the end of an older cracked section. This second crack will occur near a point where bond has been sufficiently retained to resist further straining of the unbonded reinforcement across the original crack.

The uneven distribution of moisture in a pavement is evident in the gradual increase in the width of cracks as they extend upward from the subbase to the surface. Examination of cores cut from the pavement indicates that many cracks which have a considerable surface width may be entirely closed below the level of the reinforcement. When this condition exists, infiltration of small particles of foreign matter may completely fill the crack and assist in its eventual stabilization.

Serious damage to the pavement does not result when water freezes in transverse cracks if these cracks do not exceed 1/32 in. in width. The small expansion incurred in the freezing of such a thin layer of water is elastically absorbed by the adjacent concrete with very little increase of strain in the reinforcement.

Foundation

A highway foundation should furnish adequate support for the static weight of the pavement structure and the normal dynamic loads imposed by traffic. This support should remain sound under all weather conditions and for the life of the pavement.

Much has been accomplished toward improving highway foundations, but for many practical and economic reasons, pavements are constructed on foundations which are certainly not ideal.

Although poor foundations are not recommended, continuous pavements are inherently less susceptible than other concrete pavements to many of the damaging influences associated with foundation weakness. The shorter spaced cracked sections permit greater longitudinal flexibility of the total pavement, and the continuous reinforcement allows slight pavement settlement without resulting in pumping or vertical offset between the individual cracked sections.

Friction opposes any movement between the pavement and its foundation (Fig. 3). Near the free ends of a continuous pavement this friction develops to the extent that an effective anchorage is established. The pavement extending from the point of anchorage to the free end will continue to change length with changing environmental conditions, but the pavement inside the anchorage must maintain a fixed length unless the anchorage is destroyed. Over a long period of time, sustained high compression within the pavement may cause a gradual increase in its total length, but this slight movement has a relatively insignificant effect on the general strain pattern.

When complete anchorage establishes fixed points near each end of a pavement, most stresses and strains which develop between these points are controlled in the vicinity of their origin.

The stress in the steel and the concrete at comparable points is normally about equal in magnitude throughout the fixed length of the pavement, and is dependent on localized straining to maintain this equality. In effect, each individual cracked section has fixed ends maintained by the reinforcement, and this section expands and contracts about its own geometric center. This enables the pavement to respond to changing conditions of stress and strain as a localized function. It also reduces the absolute movement of cracked sections and resultant base friction to practical insignificance.

Wheel Loads

All pavements are subjected to the forces of dynamic loading imposed upon them by moving traffic. Flexible pavements are designed to respond to these forces and rigid pavements to resist them, but under heavy wheel loads and adverse environmental conditions both pavement types may suffer some degree of damage. When a local failure occurs in a pavement surface the constant pounding of traffic usually brings about rapid deterioration and repairs are necessary.

Continuous pavements, although falling in the general classification of rigid pavements, incorporate some of the more desirable design features of both types. The

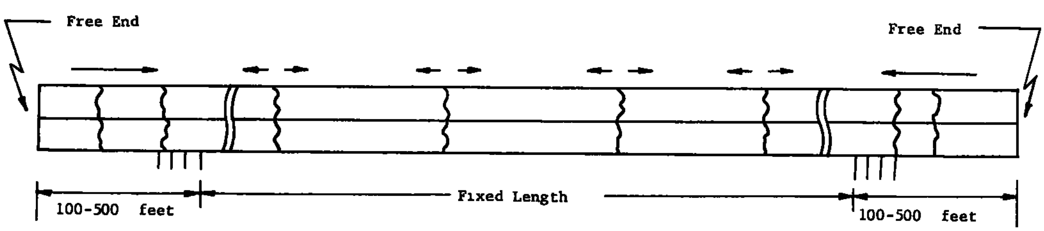

Figure 3. Pavement response to contraction forces. (Expansion forces will reverse the direction of strain.)

short segmented concrete sections allow some deflection of the pavement from surface loading, but the longitudinal reinforcement maintains continuity between these segments and, within the elastic recovery range of the steel, limits the deflection. Residual stresses throughout the pavement provide a reserve of potential strain which may be released to prevent excessive tension in the reinforcement under changing vertical load conditions.

Continuous pavements show a natural tendency for self-preservation by actively resisting damage from wheel loads. When properly designed, they should be practically immune from structural distress caused by normal traffic loads, and have a high overload safety factor.

PAVEMENT DESIGN

A thorough understanding of the major influences controlling the behavior of a continuous pavement is necessary in order to establish optimum design specifications.

In theory, if these several influences could be correctly correlated in all possible combinations of their magnitude, duration, and coincidence of occurrence, it would be possible to predict their effects upon a pavement and produce an ideal design. However, a lack of precise values for the formidable array of encountered variables makes this approach extremely difficult and of doubtful practical value.

Design based on experience alone requires years of observations of experimental pavements which eventually may prove to be over designed or structurally unsound. Some useful design limitations have been established by this method, but an unwillingness to risk weak pavements on public highways has linited large-scale field testing.

Laboratory testing with small specimens, or even full-scale pavement sections, can be very difficult to conduct, and may result in misleading information if environmental influences are not properly simulated.

During the period 1956-1958, several sponsored projects at Lehigh University have permitted a wide approach to the problems of continuous pavement design. Useful information has been obtained from many sources. Theories have been carefully checked with field and laboratory test results. By combining theory, field testing, and laboratory research with engineering experience it has been possible to develop the following basic design principle.

In transverse cross-section, the yield strength of the total longitudinal steel reinforcement must be greater than the tensile strength of the total concrete. For design purposes, the tensile strength of concrete may be taken as $\frac{1}{10}$ its compressive strength, unless tests with the concrete to be used indicate a need to alter this very conservative ratio.

The logic of this rather simple principle may be shown by explaining the process of reasoning which has led to its development.

Failures in continuous pavements usually occur at a transverse crack which has opened to an excessive width and lost the aggregate interlock of the concrete. Although a $\frac{1}{16}$ in. wide crack may close in warm weather without apparent adverse effect, greater pavement integrity is assured if the maximum cold weather crack width is held to $\frac{1}{32}$ in. or less.

A crack will occur where the tensile strains in a pavement exceed the elastic and creep response range of the concrete at a structurally weak transverse plane. After a crack develops the reinforcement becomes the only connection between two separate sections of pavement and will respond to subsequent straining without the influence of continuous concrete.

When the reinforcement is extremely weak, as compared to the tensile strength of the concrete, any additional strain in the pavement will tend to mobilize at the crack, causing the reinforcement to yield and thereby increasing the width of the crack. The crack will continue to increase in width with increasing strain, and new cracks will occur only at widely separated intervals regulated primarily by subbase friction.

When the tensile yield strength of the total reinforcement and the ultimate tensile strength of the concrete cross-section are relatively equal, strains across each crack remains well within the elastic range of the reinforcement. Under these conditions, when a pavement is subjected to additional straining, the resulting total tension is only slightly relieved by the elastic straining of the reinforcement at cracks. This additional strain in the pavement upsets the force balance at stress concentrations, and new cracks develop across these weakened planes. This process of transverse cracking will continue to relieve the strain as it develops and prevent earlier formed cracks from opening to an excessive width. Therefore, the control of cracking becomes a function of the concrete and reinforcement and is dependent upon their relative strength.

The transverse crack pattern, or cracks per unit length, is not a dependable criterion in evaluating the soundness or potential life of a continuous pavement. Two pavements of identical design, but constructed and cured under different conditions of temperature or humidity, may develop very different crack patterns. They may be equally sound and the crack widths in both pavements may be approximately the same. On the other hand, two pavements, differing only in percentage or reinforcement, may develop very similar crack patterns, yet the average maximum cold weather crack width in one may be 0.01 in. and as much as 0.08 in. in the other.

The residual stress condition of the new concrete, at a time when the first high tensile stresses of temperature contraction become operative, has a predominate influence on the number of cracks that occur in a unit length of continuously reinforced pavement. Although there is a relationship between the width of formed cracks and the number of cracks per unit length, this relationship is neither direct nor constant. Unpredictable variations in the sequence, duration, and magnitude of the early tensile stresses must be considered as controlling factors in the relationship.

The creep potential of concrete is such that under favorable conditions, most of the straining in a continuous pavement could occur without causing cracks. Although the creep rate of the concrete is often exceeded by the strain rate of the total pavement, both are mutually influential in their effect upon a developing crack pattern and upon crack widths.

Since crack width is ultimately the principal criterion for soundness, and is a function of the relative strength of the concrete and its reinforcement, pavements should be designed accordingly.

Based upon present knowledge and available materials, the following specifications are recommended as a very close approach to an optimum design for continuous pavements:

1. Foundation: 3-in. thick granular base course upon stable, well drained and compacted native soil.
2. Pavement dimensions: 8-in. thick, 12-ft wide and any length desired.
3. Concrete: air entrained, with a 28-day compressive strength of 4,000 psi.
4. Longitudinal reinforcement: reinforcing steel with a minimum yield of 60,000 psi placed at mid-depth, and comprising 0.7 percent of the pavement cross-section. Until research proves otherwise, each individual reinforcement member should not exceed ½ sq in. in cross-section area, and longitudinal continuity of reinforcement should be maintained by a 30-diameter overlap at the ends
5. Curing: the completed pavement may be cured by any conventional method which assures good quality concrete.

A pavement constructed to this conservative design should respond satisfactorily to the most adverse combinations of force-producing influences, and should remain sound for a prolonged period of time without requiring major repairs.

Continuous pavements with 0.5 percent reinforcement have been constructed in Pennsylvania, and although their performance has been satisfactory, it is believed that they represent the lowest limit to which reinforcement may be safely reduced. Because each 0.1 percent reduction in reinforcing steel reduces the paving cost approximately $0.35 per sq yd, the possibility of an occasional repair must be justified by the saving in initial construction costs.

End movement of these pavements with 0.5 percent reinforcement has been small. The finger type expansion joints on the Route 111 project have opened and closed within a $\frac{1}{2}$-in. travel range during the annual temperature cycles and have indicated a gradual pavement elongation of $\frac{1}{4}$ in. during $2\frac{1}{2}$ yr of service. The seasonal motion will continue at about the present rate, but it is believed that the pavement "growth" will decrease each year until the total pavement becomes stabilized.

Although research at Lehigh University has primarily involved pavements which were reinforced with deformed bars, preliminary tests have indicated that most other high yield steel reinforcements with reasonable bond-developing qualities may be used with equal success. Specifications may be altered to suit the characteristics of the steel if the recommended ratio of relative strength of the steel and concrete is maintained.

OBSERVATIONS AND CONCLUSIONS

Research which involves the investigation of a product for the purpose of development and improvement must necessarily be concerned with its weaker and less desirable qualities. Consequently, in making recommendations for improvements, reports on such an investigation will emphasize these weaknesses. Any superior qualities, which require no improvement, may be generally ignored.

To some extent, this has been the case with research on continuously reinforced concrete pavements. Several recent reports, including those from Lehigh University, have presented information and photographs which deal mainly with the evidence of pavement distress. Often it is not clearly indicated whether this distress came about because of a weak design, poor workmanship, or other conditions that could be equally destructive to other types of highway pavements.

Continuous pavements, when compared with other types, offer several distinct advantages. Although their simplicity of construction and initial cost are very similar to those of conventional concrete pavements, their low maintenance costs, long life, and smooth riding surface give them a definite advantage.

The 2-mi length of 4-lane continuous pavement on Route 111 year York, Pa., has provided an opportunity for comparison with the conventional jointed design used in the remaining portion of this heavily traveled highway. Two years after construction, both types of pavement are in good condition, but the smoother riding qualities of the continuous pavement are definitely noticeable. There has been no evidence of distress in the continuous pavement and it has not required any maintenance. Practically no new cracks have developed during the past year, and many of the existing cracks are gradually stabilizing.

Figure 4 (first presented at the 1958 HRB Annual Meeting and herein extended to December 1958) shows the relationship between air temperature, crack width, and average strain in the 6 gaged reinforcing bars at an instigated transverse crack as recorded every 30 days throughout the past two years.

During this period of time only two of the original 36 wire resistance strain gages installed on the reinforcement have become inoperative, and the electrical resistance to ground has remained high under all environmental conditions.

Figure 5 shows the minimum and maximum individual strain values used to obtain the average reinforcement strain shown in Figure 4. Although this pavement contains the minimum reinforcement believed to be practical, it should continue to demonstrate the several advantages of concrete pavements constructed with continuous steel reinforcement.

In the past, pavements have been constructed as inert slabs with adequate thickness to resist local damage from direct wheel loads. They have been at the mercy of their environment and almost totally dependent upon the foundation to maintain their original vertical alignment and general structural integrity.

It is believed that future pavements will be designed with the inherent potential ability to oppose successfully all normally encountered forces which will constantly act to destroy their usefulness.

The authors believe that continuously reinforced concrete pavements, even in their present state of development, meet the requirements for future highway construction and that they are definitely superior to the more conventional designs now in general

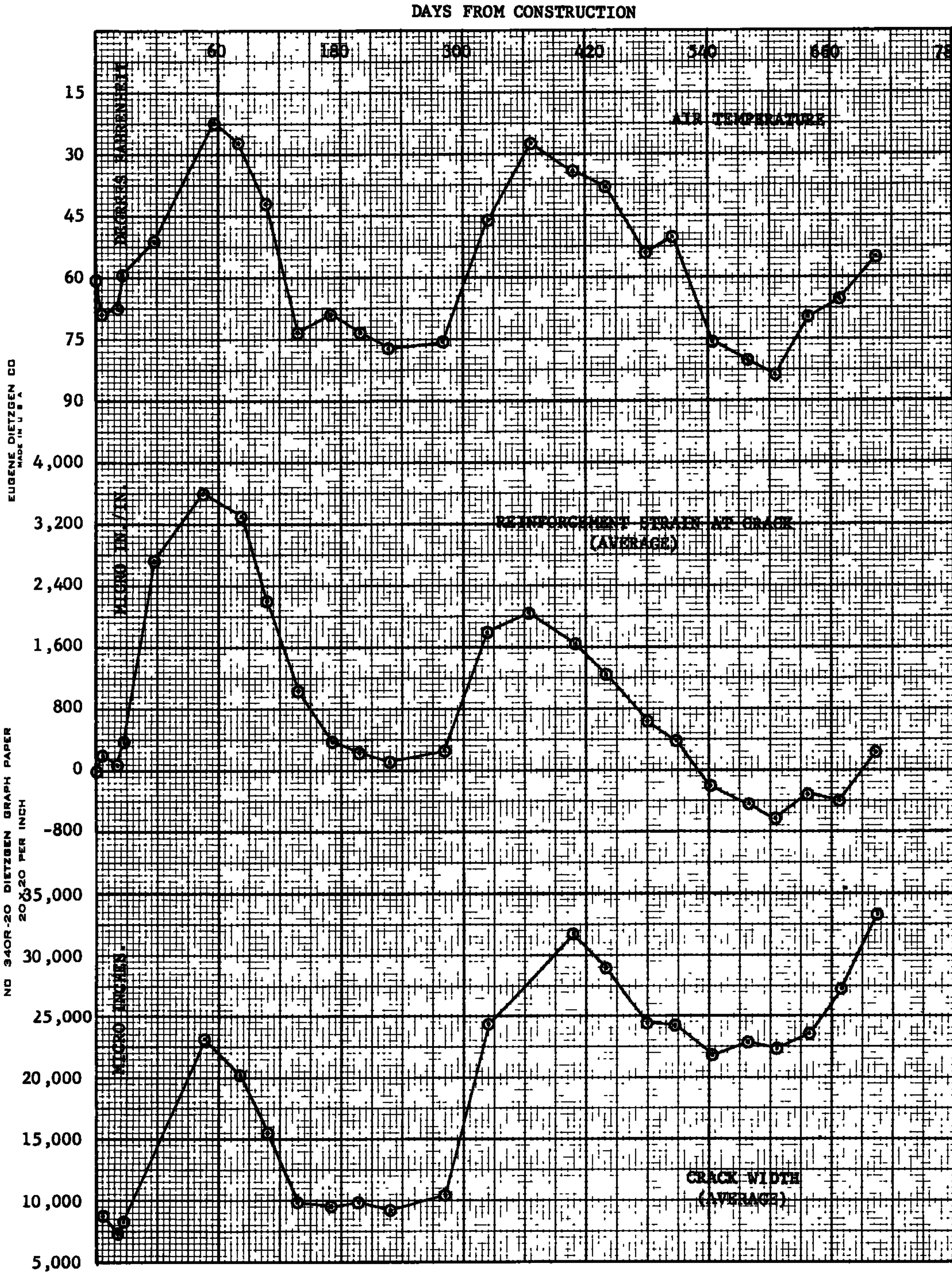
DAYS FROM CONSTRUCTION
60
180
300
420
540
660
780
DEGREES FAHRENHEIT
15
30
45
60
75
90
AIR TEMPERATURE
MICRO IN./IN.
4,000
3,200
2,400
1,600
800
0
-800
REINFORCEMENT STRAIN AT CRACK
(AVERAGE)
MICRO INCHES.
35,000
30,000
25,000
20,000
15,000
10,000
5,000
CRACK WIDTH
(AVERAGE)
EUGENE DIETZGEN CO
MADE IN U.S.A.
NO 340R-20 DIETZGEN GRAPH PAPER
20 X 20 PER INCH

Figure 4.

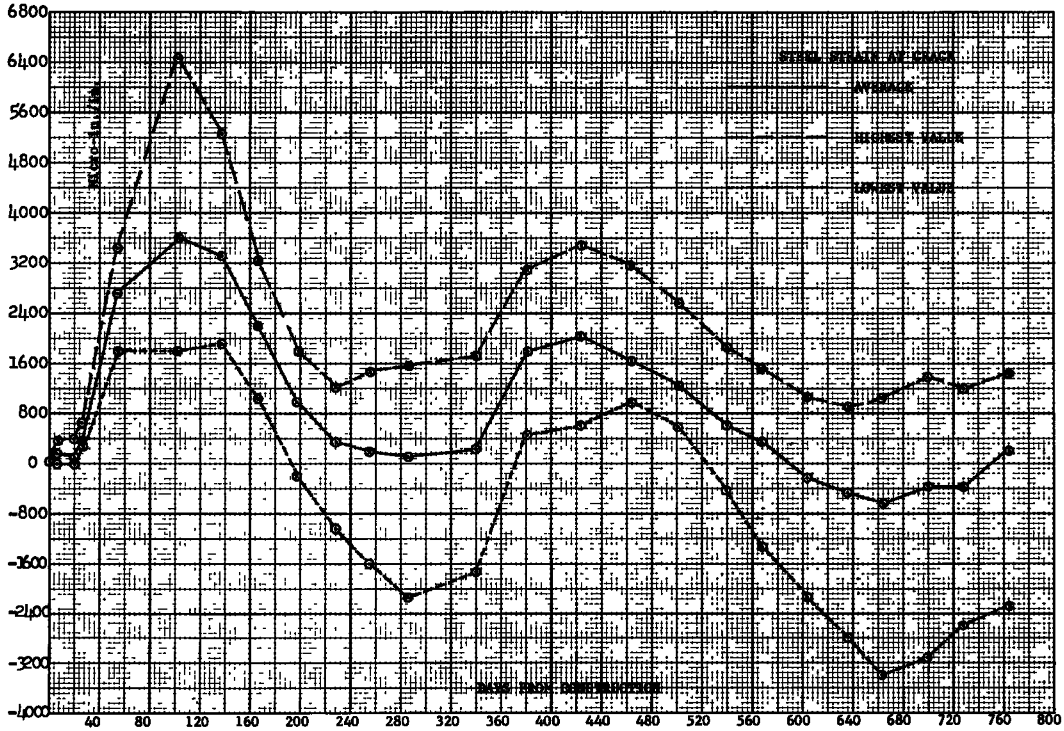

Figure 5.

use. It seems that highway engineers are beginning to accept the randomly spaced narrow transverse cracks as evidence of proper pavement behavior and to realize that they are not indicative of pavement distress which will soon require repairs. Although these cracks, with their slightly worn edges, are visible by close inspection, the motorist is unable to detect them as he drives over the surface of the highway.

The first continuous concrete pavements were constructed about twenty years ago, and their unexpected durability has been largely responsible for the current renewal of interest in their use. It is hoped that the research now being conducted will result in a wide acceptance of this type of pavement as a means of providing superior highways.

ACKNOWLEDGMENT

The authors wish to express their appreciation to the U.S. Bureau of Public Roads, the Pennsylvania Department of Highways, and the American Iron and Steel Institute for sponsoring the continuous pavement projects at Lehigh University. Many of the men within these groups have contributed valuable information and ideas to the projects.

The assistance of the entire technical staff of the Fritz Engineering Laboratory in conducting field tests and laboratory research is gratefully acknowledged.

REFERENCES

1. Schiffman, R.L., Taylor, I.J., and Eney, W.J., "Preliminary Report on Continuously-Reinforced Concrete Pavement Research in Pennsylvania." HRB Bull. 181, p. 5 (1958).
2. Taylor, I.J., and Eney, W.J., "First-Year Performance Report on Continuously-Reinforced Concrete Pavements in Pennsylvania." HRB Bull. 214, p. 98 (1959).

Discussion

BENGT F. FRIBERG, Consulting Engineer, St. Louis, Missouri — Factual research information on the paper, summarizing three years of research on continuously reinforced concrete pavements, is limited to Figures 4 and 5 which graph the progressive development of air temperatures, steel strains, and width of an artifically induced crack in one instrumented test section to an age of 700 days. The test section is about midway in the northbound outside lane of the continuously reinforced highway project between Harrisburg and York, Pa. Other research reports[1] indicate the following approximate crack frequency near the test section (including the artificial crack):

Age	Cracks per 100 ft
21 days	1
1 month	2
6 month	7
1½ yr	8

The pavement was built during early October in cold weather with seasonal temperatures decreasing further between 1- and 4-months.

Figure 6 gives the varying observed crack widths in relation to air temperature up to 700 days, as obtained from Figure 4 (for early age from HRB Bulletin 214, p. 102). The numerals show the age of the pavement at the different observations. The sloping dashed lines in Figure 6 indicate, for equal crack spacing, the theoretical rates of

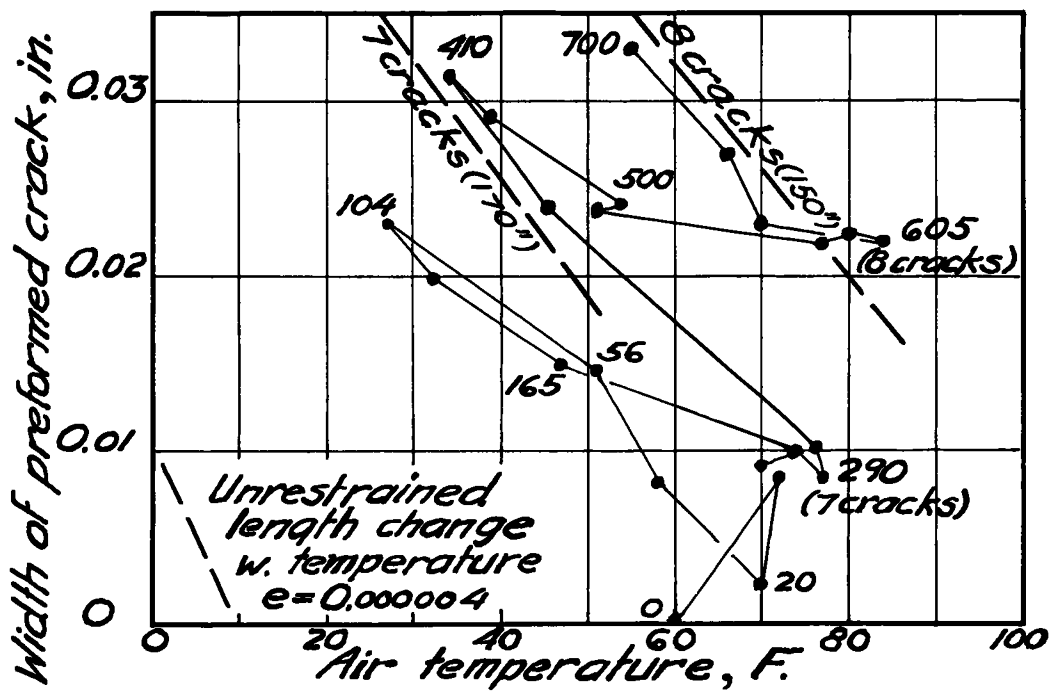

Figure 6. Progressive change in observed crack width at ages shown, from 0 to 700 days. Comparative unrestrained length change for 7 and 8 cracks per 100 ft is indicated for equal concrete temperatures.

[1] Progress Report on Continuously Reinforced Concrete Pavements in Pennsylvania, F.C. Witkoski, and R.K. Shaffer, HRB Proceedings 1959.

unrestrained changes in crack width for equal concrete temperatures, based on 0.000004 thermal coefficient, which can be considered representative when adjustment is made for seasonal variation in pavement moisture content. Concrete and air temperatures show a measure of agreement, the differences between them varying with time of day and season, 5 F to 15 F above or below. It is regrettable that temperatures of the air, rather than those of the concrete, have been plotted for comparison with steel strains and crack widths because the latter would have permitted closer study. Nevertheless, the following developments are clearly indicated at the observed crack:

1. The crack has not stabilized at two years, but shows a progressive widening which, for construction temperature of 60 F, has reached 0.03 in. without diminishing tendency for continued widening.
2. The crack widened appreciably with decreasing temperatures below 60 F during the second and third seasonal temperature drop. The widening is substantially equal to that which would occur at untied and unrestrained cracks of the same spacing as that observed.
3. No crack narrowing occurred above about 65 F air temperature, either the second or third summer, indicating compression restraint across the crack at higher temperatures.

Crack widening during the first seasonal temperature drop was only slightly less than what would be the unrestrained crack widening for 7 cracks per 100 ft. Most, if not all, of the 7 cracks per 100 ft observed at 6 months probably did occur during the early part of the first winter.

Before appreciable cracking occurred temperature contraction was all restrained by concrete stress change. The observed change in steel strains at the crack converted to concrete stress then give a measure of the modulus of elasticity in the concrete. As indicated by slab temperatures and steel strains shown in Figure 11 on page 15 of HRB Bulletin 181, between the 3- and 8-day cracking, the concrete modulus of elasticity varied between 1 and 1½ million psi, at 28 days the modulus of elasticity had increased to over 3 million psi (assuming 0.010 in. per 100 ft length variation at cracks), and at 40 days to about 4 million psi (assuming 0.020 in. per 100 ft at cracks.) Obviously, the modulus of elasticity of the concrete increased slowly at early age in the low temperature concrete, and daily temperature fluctuations could be accommodated without high tension stresses in the concrete. Concrete modulus of elasticity and tensile strength, in combined direct tension and flexure, dominate the development of initial cracking in continuous pavements.

Figure 4 shows unit strains at yielding of the steel (about 0.0025) at about 50-day age; however, temperature records are incomplete. Prior to effective relief of tension strains through extensive cracking, the yield stress in the 0.45 percent of steel at the pre-formed crack could have been exceeded for temperature drops greater than 40 F for 1.5, 20 F for 3.0, and 15 F for 4.0 million-psi concrete modulus, corresponding to 300-psi average concrete tension on the cross-section, which undoubtedly was less than the concrete strength from 10 days on. Shrinkage after removal of the curing paper apparently induced a tension strain corresponding to at least a 10 F temperature drop. Accordingly, yield strains at the crack probably were reached considerably before the 50-day age, indicated in Figure 4 at an air temperature 10 F lower than construction temperature, with true yield at somewhat higher temperature.

Unit tension strains went considerably higher than the 0.0036 shown in Figure 4. Bond failure apparently progressed away from the crack under repeated stresses near yield stress during the first winter, reaching 24 in. back from the crack at 140 days (HRB Bulletin 214, p. 102). Thereafter the observed changes in steel strains 24 in. from the crack were about 80 percent of those at the crack, showing virtually no bond resistance in the 24-in. distance. Shortly after 140 days zero stress in the steel at the crack would correspond to about 0.003 unit elongation, clearly indicated by steel strains 24 in. away, which then during some indefinite short period appear to give a true indication of steel stresses, until compression yield strains were exceeded.

Without detailed information on steel strains during the intervals between observations, the strains in Figures 4 and 5 obviously permit only extremely vague deductions concerning steel stresses at cracks. After the first winter, Figure 6 indicates no restraining value of the steel in holding the cracks from widening with decreasing temperatures, below the temperature at which the cracks obviously were under compression restraint.

The paper contains no data on the instrumented test panels in the 7-, 8-, and 9-in. bar reinforced and the 9-in. mesh reinforced pavements of the project on US 22 near Hamburg, Pa., which were part of the same research. Those test panels, placed at different temperatures, could possibly furnish valuable information on early restraint stresses and relations between construction temperature and compression restraint across cracks. Omission of the data is regretted, especially as the paper gives no indication that information can be expected. To quote from the paper: "Rather than presenting detailed test data from individual gage measurements, this report will deal primarily with the general behavior of the pavements under tests and the more conclusive information that is required for the establishment of design criteria for all continuous pavements."

On the basis of the factual research information discussed above, and published theoretical information on restraint stresses in continuous reinforcement, the general "conclusive" information and specific recommendations for continuous pavements are not amply supported. General statements on pavement behavior without direct bearing on the specific research and design recommendations, will not be discussed.

The intended characteristics of continuously reinforced pavements is obviously closely spaced cracks; maximum cold weather crack width of ⅓₂ in. is stated as desirable. Based on the observed crack and average crack spacing data there is no evidence that the continuous reinforcement would be able to hold the crack widths to that maximum. It is stated that the yield strength of the total longitudinal steel must be greater than the total concrete strength to induce desired results. Although the criterion has been stated repeatedly prior to this research, the statement is insufficient on the evidence of the research. Construction temperature obviously exerted much more noticeable influence on crack spacing than steel percentage. For summer construction temperatures on the Hamburg project crack spacings less than 10 ft at one month were most generally observed with 0.5 percent steel. The concrete tension strength varies, is influenced by flexural restraints, and its relation to cracking does not appear to be governed by the simple rule suggested for the steel. The shortest crack spacing has been observed in the concrete with highest strength at early age.

Progressive destruction of bond near cracks appears to be a factor of equal or greater concern than yield strength in determining restraint stresses which can be induced by the steel. The total steel tension can be very greatly diminished by elastic straining over some distance on both sides of cracks. There is evidence on the Hamburg project that bond failure at some cracks can destroy completely the effective continuity of the steel.

Contrary to suggestion in the paper, average crack spacing and crack width appear to be closely related. Measurements on an Illinois continuously reinforced project show such relation at high temperature, Figure 6 indicates even closer relationship for temperatures below those at which cracks are in compressive restraint. Differences in average crack width of 0.01 and 0.08 in. cannot be dismissed without explanation. The statement that pavements "may develop very different crack patterns" and yet "be equally sound and the crack widths in both pavements may be approximately the same," has not been supported by evidence. The evidence indicates that crack widths in mature pavements depend upon temperature of observation, below that at which compression restraint prevents further closing of cracks, and that crack widths are not appreciably decreased by continuous reinforcement. The measurements apparently would be about the same for the same crack spacing in a similar unreinforced pavement. Crack width measurements at high temperature are a measure of variable infiltration that may have occurred earlier, rather than pertinent crack width for low temperature service.

Design recommendations in the paper disregard the important influence of bond upon the steel stresses which alone are stated to induce close crack spacing, and assure narrow cracks. Even if true, it is doubtful that 60,000-psi stress can be utilized in ¾-in. bars, and perhaps not in smaller bars under repeated bond stresses occurring in continuous reinforcement for 0.03- to 0.06-in. crack widths. The more pertinent important influence of construction temperature is disregarded. Curing methods which do not protect the pavement against high temperatures may well be favored in continuous pavements, although their use may well be questioned for conventional slabs where they are commonly used in preference to covering paper or burlap.

The continuing value of continuous reinforcement in mature pavements has not been demonstrated beyond the apparent beneficial effect of longitudinal compressive restraint across cracks which is not exclusively reserved for continuously reinforced pavements. More than soundness at early age, or even continued durability of a 20-yr continuously reinforced pavement (a not exceptionally impressive age for concrete pavements) is required to justify the expenditure for longitudinal steel of $2.50 per sq yd of pavement.

I. J. TAYLOR and W. J. ENEY, Closure — It was not believed feasible to provide the complete data from three years of pavement research in a paper of the type presented. More than 6,000 separate readings from 200 gage devices would have been involved in presenting measurements alone. Since earlier reports on the subject of continuous pavements have contained substantiating data covering many of the statements made in this paper, it was not considered necessary to repeat test-proven and generally accepted facts on pavement behavior.

Each reader, depending upon his special field of interest, considers certain specific data to be pertinent to the conclusions or recommendations offered by the authors. Since the authors could not anticipate the contentions of each reader, only limited general data supporting the claims of the paper were presented.

The graph, showing the reinforcement strain in Figure 4, plots the average value of 12 strain gages located on 6 of the 20 longitudinal reinforcement bars which comprise the pavement cross-section at a preformed transverse crack. (HRB Bulletin 181, p. 11). To assure an early crack, the cross-section of the concrete was reduced 50 percent by the insertion of a thin metal strip across the lower half of the pavement. A 20 percent reduction of each gaged bar cross-section resulted when the bars were machined to provide a smooth surface for attachment of the gages. Also, the bond potential for each gaged bar was destroyed along a 2-in. length by the process of waterproofing the attached gages.

Figure 5 shows the extreme range from which the average strain values for Figure 4 were obtained.

Since the induced crack occurred approximately 36 hr after pavement construction, bonding between the reinforcement and the new concrete was relatively weak. This brought about a bond and strain condition that was different from that found in the remainder of the pavement, where cracks developed considerably later in the stronger and longer cured concrete. (HRB Bulletin 214, p. 103).

Although the relationship between strain in the reinforcement, crack width, and air temperature may provide very useful information, it is unlikely that the average strains from 30 percent of the total reinforcement or the strain from any individual bar can be converted to obtain reliable concrete stresses or moduli of elasticity. The authors have never contended that the resistance gages used in continuous pavement research gave more than the accurate strain conditions of a steel reinforcing bar directly beneath the applied gage.

Since factual stress information is not available from Figures 4 and 5, or even Figure 6, it is considered diverse from the stated intent of the discussion, and further conjectural comment would be unwarranted.

The solid line in Figure 6 plots the strain-temperature history of the induced crack.

The broken lines represent the theoretical strain-temperature response of a crack in a similar concrete pavement without reinforcement, but with a uniform crack spacing and crack width corresponding to the average of the reinforced pavement. In order to accomplish this, it was necessary to assume a crack pattern that could develop only through the use of continuous reinforcement and assign an adjusted thermal coefficient of 0.000004 to the 100-ft section under consideration.

The conclusions, (a) that the crack on the York pavement will continue to widen, and (b) that the reinforcement offers no restraint to free thermal contraction of the pavement, were found incompatible when the information available from Figure 6 was carefully analyzed.

With the adjusted thermal coefficient of 0.000004, the total contraction of 100 ft of unrestrained concrete pavement would be 0.288 in. as the result of a decrease from a construction temperature of 60 F to a low of 0 F. With the occurrence of eight evenly-spaced cracks of uniform width, each crack would be 0.036 in. wide. The calculated maximum crack width for an unrestrained pavement (0.036 in.) is essentially that of the observed induced crack at 700 days (0.033 in.), and further widening of the observed crack should not occur at pavement temperatures above 0 F.

Since it is known that the observed pavement crack will actually widen as the temperature decreases toward 0 F, this method of analyzing the strain-temperature history is obviously unsound. By reducing the thermal coefficient of concrete from the generally accepted value of 0.000006, it is possible to provide a basis for conclusion (b), but by doing so the logic of conclusion (a) is destroyed.

If the pavement contracts under the influence of temperature, without restraint by the reinforcement at cracks, then temperature alone would regulate the crack width; furthermore, progressive widening of the crack would require a progressive decrease in the reinforcement restraint. These two actions—contraction due to temperature drop, and progressive widening of the crack—are interdependent and simultaneous under the conditions involved in the discussion of Figure 6. Therefore, progressive widening is possible only if the reinforcement offers tensile restraint which diminishes because of progressive yielding or bond loss on either one or both sides of the crack.

Although a simple analogy based solely upon a temperature-strain relationship provides some useful information, the influence of many other variables must be considered if the behavior of an actual pavement is to be satisfactorily explained. Furthermore, any direct comparison between a continuously reinforced concrete pavement and one that has no reinforcement can be misleading. The latter, depending upon the construction temperature and the tensile strength of the concrete at the instant of the initial cracking, would develop a crack spacing of from 30 to 80 ft, with correspondingly wide cracks.

The induced crack in the York pavement represents the most adverse crack condition expected to be found in a continuously reinforced pavement. The very early occurrence of the crack in a pavement with only 0.5 percent reinforcement allowed the crack to open wider than 1/32 in., and caused slight yielding of the reinforcement. While cracks that developed later have remained narrower, it is likely that the reinforcement has yielded at these cracks also.

The authors have recommended a practical pavement design that will limit the width to which transverse cracks may open before any increased strain resulting from thermal contraction will instigate new cracks along the continuous pavement structure. When crack width is controlled by the use of reinforcement, crack frequency only indicates the maximum temperature-induced tensile strain to which the pavement has been subjected.

Several pavements have been constructed based upon the principles set forth in the authors' recommendations. Direct comparisons will be made with sections of conventional jointed pavements constructed as controls. It should not be necessary to wait until these pavements are destroyed, or even so long as 20 yr before their condition, behavior, or life expectancy can be evaluated.

The cost of the additional reinforcement used in continuous pavements is offset by the savings in materials and labor required for the installation of load-transfer and expansion joints needed in conventional pavements. The unit cost of the 8-in. thick continuous pavement on the Hamburg project was 9.5 percent lower than that of the 10-in. thick jointed pavement constructed under the same contract. (HRB Bulletin 214, p. 90).

The authors agree with Mr. Friberg that statements published without the accompaniment of substantiating data or factual information may appear to be only conjecture; but to counter those statements with others of a similar nature merely indicates a difference of opinion. They are enthusiastic concerning the possibilities of continuous pavements, and are anxious to convince others of the advantages.

Laboratory Research on Pavements Continuously Reinforced With Welded Wire Fabric

M.J. GUTZWILLER and J.L. WALING, respectively, Associate Professor and Professor of Structural Engineering, Purdue University

Since February 1955, research has been conducted at the Structural Engineering Laboratory of the School of Civil Engineering, Purdue University, on reinforced concrete pavements using welded wire fabric as the principal reinforcement. The final techniques as used in the laboratory are presented.

The specimens chosen were 28 ft long by 3 ft wide by 8 in. thick. The reinforcement consisted of either 6 x 12, $\frac{0}{3}$; 6 x 12, 00000/0; or 4 x 12, 00000/0 welded wire fabric. The specimens were cast in a portable form in which the amount of steel, the location of the steel, and the depth of slab could be varied. Each of the specimens was fabricated with weakened planes such that the slabs would crack at definite locations in the testing region of the slab. This permitted measurement of strains in the fabric at these predetermined cracks.

The slabs were tested on an elastic subgrade having a subgrade modulus of approximately 160 pci, although the subgrade modulus could be varied within reasonable limits. The slab specimens were loaded with vertical static loads to simulate traffic loads and horizontal loads to simulate stresses induced by temperature changes. Electric SR-4 strain gages were placed at various locations on the fabric to determine the stresses in the fabric. Vertical deflections of the slabs were obtained by use of Federal dial indicators, and crack widths or surface strains in the concrete were obtained by use of a Whittemore strain gage.

● RECENT years have brought an increased amount of interest in trying to determine engineering facts about concrete pavements in order that these facts might be incorporated in design criteria for such slabs. Reinforced concrete pavements are an answer to the search for a more efficient highway pavement; but the behavior of these pavements, with all of the variables involved, is still not fully understood. At the 1958 Highway Research Board Annual Meeting, various papers pertaining to continuously reinforced pavements were presented (1, 2, 3, 4). In previous sessions the condition of the experimental section of continuously reinforced highway in Indiana has been reported (5). Also at the 1958 meeting, a paper was presented on a theoretical "Analysis of Special Problems in Continuously Reinforced Concrete Pavements" (6). It can readily be seen that there is indeed a great amount of interest shown in the use of continuous reinforcement in concrete pavements. In each case, the experimental work reported involved field tests on sections of pavements continuously reinforced with either welded wire fabric or deformed bars.

Various state highway departments have conducted research on pavement slabs. Other organizations such as the Bureau of Public Roads, Highway Research Board, Portland Cement Association, American Concrete Institute, and many individuals have conducted research hoping to improve present design concepts.

In recent years the American Iron and Steel Institute organized a Committee on Welded Wire Fabric Reinforcement Research. The committee consisted of individuals who had varying interests, and it soon became apparent to certain members of the committee that there was a definite need for some basic research on concrete pavements in which some of the variables could be isolated, controlled, and studied. C. A. Willson, Research Engineer for the American Iron and Steel Institute, the late Wayne Woolley, Highway Engineer for Republic Steel, K. B. Woods, Head of the School of Civil Engineering, Purdue University, and several members of the committee were interested in a laboratory research program on pavement slabs and arranged for a meeting at Purdue to discuss the possibilities of starting such a research program. As a consequence of this initial meeting, the Committee on Welded Wire Fabric Reinforcement Research and the Purdue University School of Civil Engineering initiated a research program in 1955.

This laboratory research program has been continued since that time and a number of the variables involved in the design of concrete pavement slabs continuously reinforced with welded wire fabric have been studied. This investigation has been supplemented by a theoretical analysis (7) of continuously reinforced pavement slabs sponsored by the Indiana Joint Highway Research Project. Also, another committee of the Institute has sponsored a laboratory program to study the use of deformed bars as a continuous reinforcement in concrete pavements (8).

PURPOSE AND SCOPE OF RESEARCH

The objectives of this research were to determine by laboratory means the following:

1. The relationship between the percentage and position of continuous steel reinforcement and the formation, spacing, and widths of cracks in reinforced concrete pavement slabs of determined thickness, caused by stresses induced by temperature, curing shrinkage, and live loads.
2. The relationships of stresses and deflections to percentage and position of steel in concrete pavements continuously reinforced with welded wire fabric, resting on subgrades of various stiffnesses, and subjected to various combinations of live loads and simulated temperature changes.

Thus, the controlled variables included in the experiments were percentage of longitudinal reinforcement, position of reinforcement, subgrade modulus, range of simulated temperature drop after casting of the concrete (longitudinal forces) and magnitude and position of simulated wheel loads. The dependent variables which were measured during the experiments were strains in the welded wire fabric steel at and adjacent to preformed cracks, concrete surface strains and crack widths, and vertical deflections of the slabs.

EXPERIMENTAL SPECIMENS AND APPARATUS

In designing the experiments, several items concerning the test specimens and the test apparatus received consideration.

Size of Specimen

The specimens used in each phase of this research work reported to date have been of the same size. The concrete slabs were 28 ft long, 3 ft wide, and 8 in. thick. Since it was not intended to simulate any of the effects of lateral stresses such as warping and curling of the slab, specimens 3 ft in width were chosen. This width provided enough contact surface for application of the vertical loads and permitted lateral distribution of the loads. The length of slab was limited to 28 ft to permit use of the facilities which were already available in the Structural Laboratory. This provided adequate length for a reasonable portion of each slab (middle 10 ft) to simulate, under the test conditions described later, the fully restrained area of a continuously reinforced pavement and to furnish sufficient measured data. Furthermore, in this length of continuously reinforced pavement slab, a definite crack pattern could form to predict the patterns which would occur in the field. An initial depth of 8 in. was chosen to conform with the actual depth used by some states in their present pavement design standards.

Concrete Specifications

The specifications for the concrete were established to conform to average present-day highway requirements. Due to the size of the specimens, it was appropriate to obtain the concrete from a local ready-mix plant. The concrete furnished was to meet the following minimum requirements:

1.	Twenty-eight day ultimate compressive strength	4,000 psi
2.	Maximum size aggregate	1½ in.
3.	Slump	2 to 4 in.
4.	Air entrainment	3 to 6 percent

Upon delivery of the concrete to the laboratory, slump and air entrainment tests were immediately made to see if the specifications were satisfied. Three 6-in. diameter cylinders 12 in. long were cast to determine the compressive strength, and three 6- by 6- by 16-in. were poured to determine the flexural strength of the concrete.

Steel Reinforcement

The specifications for the reinforcement steel used in the laboratory specimens were as follows:

1. The steel was to meet the standard specification for cold drawn steel wire for concrete reinforcement as specified in ASTM Specification A82-34.
2. The welded steel wire fabric was to meet the specifications for welded steel wire fabric as specified in ASTM Specification A185-54T.
3. The welded wire fabric was to be chosen from sizes that are available commercially.

The welded wire fabrics used were 6 x 12 - ⁰/₃, 6x12 - 00000/0, and 4x12 - 00000/0. Standard tension tests were performed on individual wires both between and across the welds to determine the ultimate strength and the modulus of elasticity of the fabric used in the experiments. The ultimate strength of the welds was determined by means of the Wire Reinforcement Institute Weld Tester.

Subgrade

The slabs were tested on an elastic subgrade. Tests were to be made on several slabs resting on subgrades having the same modulus. It was anticipated that the use of a soil subgrade would introduce a variable subgrade modulus due to changing moisture content and progressive compaction during the series of tests. A subgrade was needed that would have a constant modulus during the period of the test (2 yr or more), and which would be reproducible, or very nearly so, at any future date. The Firestone Industrial Products Co. fabricated a small test slab of rubber 41 in. wide, 66 in. long, and 5 in. thick for preliminary testing. This subgrade specimen was made up of ten pads 41 in. by 33 in. by 1 in. stacked and glued brick fashion to alternate the joints. This specimen of rubber was subjected to a plate loading test and was found to have an average subgrade modulus of 175 pci. This subgrade modulus would represent a soil having medium plasticity, soft plastic clays having a modulus of approximately 50 pci while densely graded non-plastic sandy gravels may have a modulus of 500 or more (9). As a result of this preliminary test on the rubber subgrade, sixty pads 41 in. by 33 in. by 1 in. were purchased with the request that the rubber be made slightly more flexible than that of the trial rubber slab, to be nearly the middle range of soils of medium plasticity. The pads were stacked on the floor brick fashion in five layers, left unglued, and the resulting subgrade modulus was 155 pci. Leaving the pads unglued has the advantage that it provides greater flexibility for the laying out of future subgrades of different dimensions, patterns and moduli. As an example, such a laid-up subgrade 3 in. thick has a modulus of 440 pci.

Preformed Cracks

A survey of the literature pertaining to continuously reinforced pavements showed that the spacing of cracks developed in service varied, from approximately 2 to 15 ft depending on the amount of reinforcement, thickness of slab, distance from a formed joint at the end of a test strip, and other factors. A crack spacing of 5 ft was thought to be fairly representative of the average condition in the central portion of a continuously reinforced pavement; hence, three transverse cracks were performed — one at the the center of the slab and one 5 ft from the center on each side. The primary purpose of preforming these cracks was to insure that strain gages would be installed on the steel reinforcement at cracked sections.

The cracks were formed using 18-gage sheet metal strips 3 in. wide and 4 ft 6 in. long placed through vertical slots in the side boards of the form to span the form just under the longitudinal reinforcement. Two small pieces of plywood nailed to the bottom of the form on each side of the metal were used to keep it from deflecting to any great extent when the concrete was poured (Fig. 1).

Form for Test Slab

The form consisted of three plywood carts each 8 ft long and one cart 4 ft long resting on truck casters. The four carts assembled end-to-end with side and end boards in place provided a clear space 28 ft by 3 ft by 8 in. Holes were drilled in the end and side boards of the forms to receive the longitudinal reinforcement and the transverse bars. Slots were cut in the sides of the form to hold the sheet metal used to preform the partial cracks in the slabs (Fig. 2).

Preparation for Pouring

The steel reinforcement was installed in a single layer with the sheets lap-spliced at the two outermost preformed cracks, except in slab 1 where one lap was formed just beyond one of the cracks. An extra short length of fabric of the same size as the main reinforcement was installed at each end of the slab to provide adequate strength in fittings for applying longitudinal forces to the reinforcement. These extra sheets were offset laterally a distance of a half space and were turned over so that all of the longitudinal steel at the slab end was in the same plane.

Twelve No. 4 transverse bars were placed in the slab on 2-ft 6-in. centers. These bars served three purposes:

1. By means of threads and nuts on the ends of the bars, they tied the side boards of the form together, thus restraining the sides against spreading.

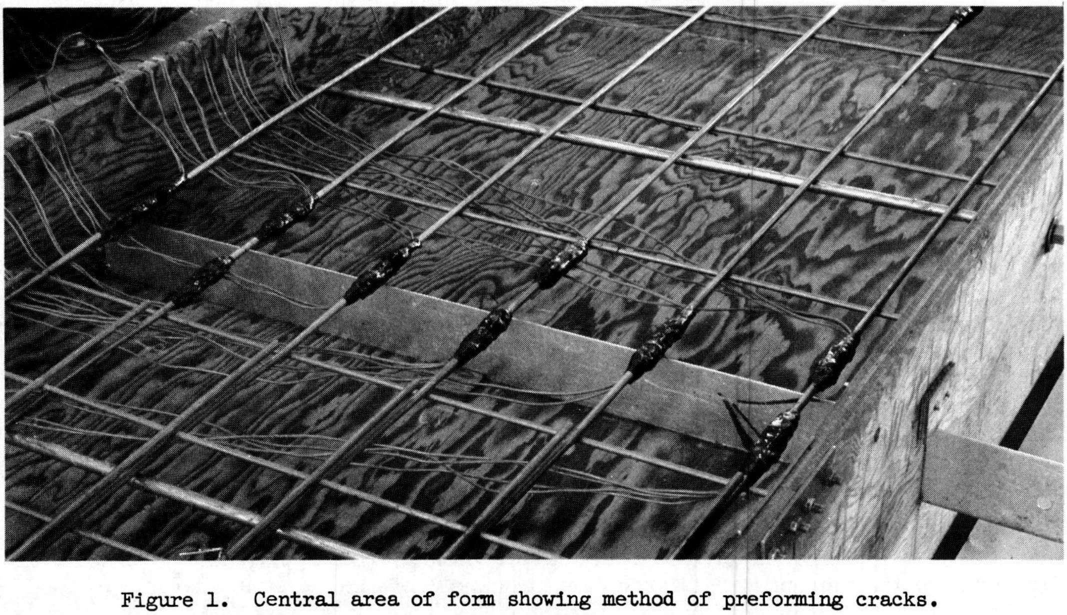

Figure 1. Central area of form showing method of preforming cracks.

2. They supported the longitudinal reinforcement in its correct position.
3. They served as lugs for the hangers used in lifting the slab from the form and lowering it onto the rubber.

The strain gage stations on the welded wire fabric were prepared by removing all rust and scale with emery cloth. The cleaned surface was roughened to insure a good base for the glue and then thoroughly cleaned with acetone. The gages were preformed to fit the fabric and were placed on the wire fabric at the preformed cracks and at several other gage locations between the cracks. Either two type A-7 or two type A-18 SR-4 electric strain gages were applied diametrically opposite one another, wired, and waterproofed at each of the predetermined gage stations. The fabric reinforcement was then placed in position and each sheet of fabric was tied to the transverse bars with soft wire. Two temperature compensation gages mounted on short pieces of reinforcement steel were placed in the form. The strain gage lead wires were carefully laid out to emerge from the form at the top edge near the middle of the test specimen.

The three pieces of oiled 18-gage sheet metal were placed through the sides of the forms at the location of the preformed cracks.

Pouring of Concrete

As mentioned before, the test slabs were poured using ready-mix concrete. The concrete was vibrated carefully to insure the filling of all spaces around reinforcement, strain gages, and lead wires. After striking off and before initial set had taken place, plugs for Whittemore strain gage readings were embedded in the top surface of the slab.

Curing of Concrete

Approximately 6 hr after completion of the pouring, two layers of wet burlap were placed on the slab and were covered with a heavy canvas tarpaulin. The burlap was kept continuously wet for three weeks after which the cover, the burlap bags, and the side and end boards of the form were removed (Fig. 3). The test cylinders and beams were cured in the same manner as the slabs and were tested at 28 days after casting.

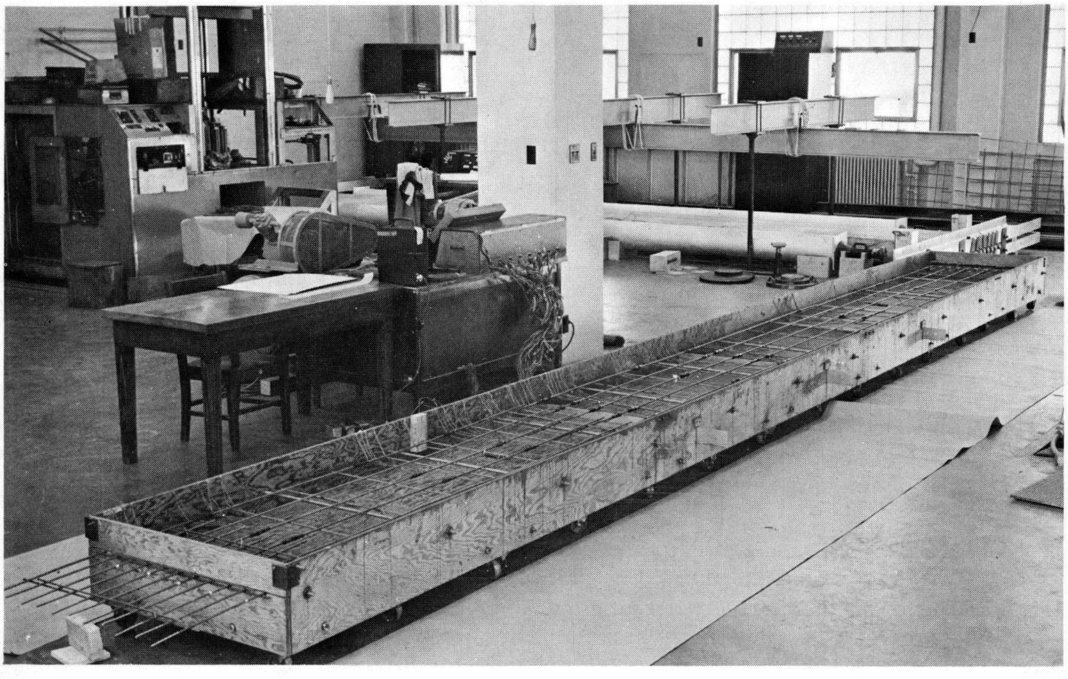

Figure 2. Full-length view of form.

Positioning Slabs for Test

A system (Fig. 4) was devised whereby a test slab could be carefully transferred from the form to the rubber base. Eye-bolt hangers were hooked onto the projecting ends of the transverse bars in the test slab. The threaded upper ends of the hangers on each side of the slab were engaged by nuts and washers to a longitudinal timber beam which was supported in position parallel to the side of the slab by a steel framework over the slab. By tightening all of the nuts at the upper ends of the hangers, the slab was raised free of the form and suspended while the form was pulled out and the rubber subgrade was laid into position. The slab was then lowered into position by loosening all of the nuts evenly.

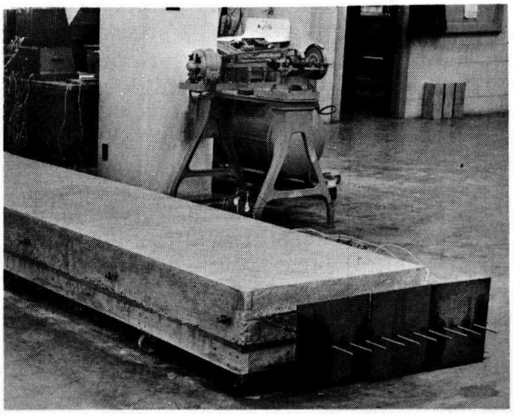

Figure 3. Slab at end of curing period.

EXPERIMENTAL PROCEDURES

Measurement of Vertical Deflection

On a line 5 in. in from one edge of the slab, nine 0.001-in. Federal dial indicators were suspended from a framework with the stems of each engaging a piece of sheet tin bonded to the surface of the concrete. These indicators were spaced at 20 in. on centers in the test area, with an indicator at each of the preformed cracks. The dials were used to measure the vertical deflections of the surface of the slab at these points, when the slab was subjected to the various combinations of loading. This arrangement of dial indicators is shown in Figure 5.

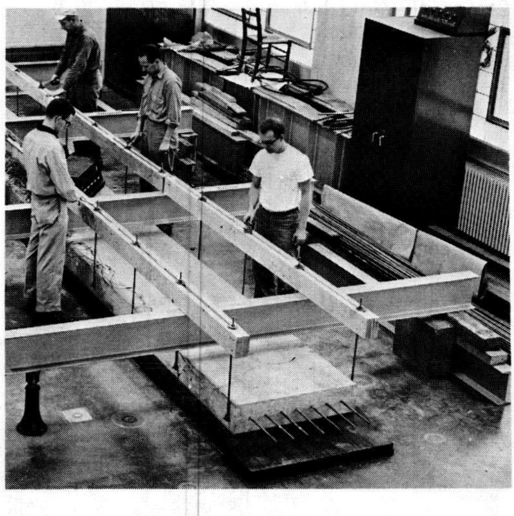

Figure 4. Lowering of slab onto rubber base.

Measurement of Surface Strains and Crack Widths

In the test area, plugs were embedded in the surface of the concrete at 10-in. intervals on a line 8 in. in from one edge of the slab. Gage holes were drilled in these plugs and a 10-in. Whittemore strain gage was used (Fig. 6) to make surface strain and crack width measurements.

For an uncracked section, the unit strain was taken as the change in length between adjacent plugs divided by ten. When a cracked section occurred between two plugs, it was assumed that the total change in distance between plugs was due to the change in crack width.

Determination of Stresses in Welded Wire Fabric

As mentioned previously, Electric SR-4 strain gages were used to determine the stresses set up in the welded wire fabric. Since the stresses in the longitudinal steel could be assumed to be primarily uniaxial, the strain readings obtained with the gages, when multiplied by the modulus of elasticity of the steel in the fabric gave the stresses in the wire fabric. Type A-7 and type A-18 paper backed constant wire gages having a resistance of 120 ohms and a gage factor of $1.9^{\pm}$ were used.

Two gages were used at each station. They were placed longitudinally on opposite sides of a wire, with the center of their gage filaments at the same cross-section but with their leads facing in opposite directions. The two gages were used to observe any bending or eccentric load effect.

Strain readings were taken with a Baldwin SR-4, type L, strain indicator.

Vertical Loading

The framing for application of the vertical loads to the slab consisted of a longitudinal beam supported above the longitudinal center line of the slab by 4 transverse bents spaced 6 ft on centers. Each bent was made up of a transverse beam clamped at its ends to vertical pipe posts which were threaded into floor inserts on each side of the slab. The longitudinal beam transmitted the reaction of the vertical load to the bents which in turn transmitted the vertical uplift to the floor inserts (Fig. 7).

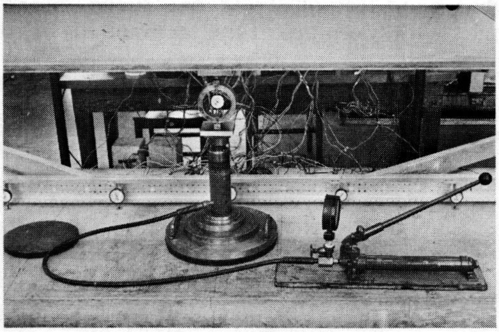

Figure 5. Vertical deflection and vertical loading apparatus.

The vertical load was applied by means of a hydraulic jack. Its magnitude was determined with a proving ring placed between the the jack and the longitudinal beam. Three loading magnitudes of 5,000, 10,000, and 15,000 lb were applied. The largest of these loads compares closely to a wheel loading for the H20-S16-44 loading which is the present standard for design for bridges and larger than the wheel load for highway pavements. It was estimated that a 15,000-lb load on the 3-ft width of the test slab would produce maximum bending moments in the longitudinal direction of about the same magnitude as the maximum moments produced by a pair of 16,000-lb wheel loads on a 12-ft pavement slab. The 5,000-lb load was transmitted to the slab through a stack of circular steel plates of increasing size, of which the largest was 12 in. in diameter, and a piece of rubber of the same diameter located on the surface of the slab at the test position. The 10,000- and 15,000-lb loads were applied through a stack of plates, of which the largest was 18 in. in diameter, and a piece of rubber of the same diameter. The purpose of the rubber was to seat the plate on the slab surface, thus obtaining a more uniform application of load to more nearly simulate an actual tire load. These sizes conform sufficiently close to the equivalent contact areas of 12-, 16-, and 19-in. diameters, respectively for 5,000-, 10,000-, and 15,000-lb dual wheel loads as given in Concrete Pavement Design (9).

Figure 6. Measurements of surface strains and crack widths.

Longitudinal Loading

To simulate temperature stresses, tensile stresses were induced in the reinforced slab by applying longitudinal loads to the longitudinal wires which protruded 18 in. from each end of the slab. Each wire was threaded approximately 4 in. for loading purposes. Tension tests on similar threaded wires indicated an ultimate strength of threaded wires approximately $^5/_7$ of the ultimate strength of unthreaded wires. Thus, in the case of 6 x 12 fabric, five longitudinal wires were added to the original six wires at the ends of the specimen.

A set of two beams was placed transverse to the slab at each end with the projecting wires passing between them. Plates and nuts were fitted on the threaded ends of the wires with the plates flush against the beams.

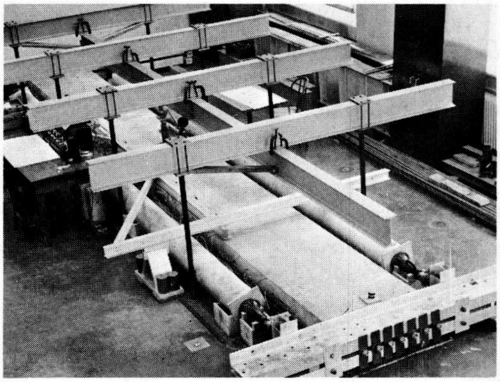

Figure 7. Framework for vertical loading.

Figure 8. Longitudinal load jack and cell.

Thus, when the beams were forced away from the slab in the longitudinal direction, a tensile load was applied to the wires which in turn transferred a portion of it to the concrete of the slab. The nuts were adjusted so that the longitudinal load was distributed as evenly as possible among the longitudinal wires. This distribution among the wires was facilitated by SR-4 strain gages mounted on the projecting wires between the threaded portion of the wires and the slab end (Fig. 7).

The longitudinal load was applied to the transverse beams by either screw jacks or hydraulic jacks placed on each side of the slab near each end. The jacks on each side of the slab worked against each other through pipe columns (Fig. 7). The applied load was measured by means of cylindrical load cells which were placed between the jacks and the transverse beams (Fig. 8). Small bearing beams placed between the load cells and the transverse beams served to distribute the load on the flanges of the transverse beams.

For the large longitudinal loads, four solid cylindrical steel load cells each having a cross-sectional area of 3.14 sq in. and a 4-in. length, were placed two at each end. For the small and medium loads, to obtain greater sensitivity, two of the solid load cells were replaced by hollow circular stainless steel load cells having a cross-sectional area of 1 sq in. and a 4-in. length.

Each load cell was constructed by placing four SR-4 strain gages on longitudinal elements at the quarter points of the circular cylinder. Two diametrically opposite gages were connected in series; the two pairs were then connected in parallel. The series connections eliminated the effect of any bending of the cylinder and the parallel connection of the pairs in series resulted in a resistance change of the connected four gages equal to that of a single gage on a perfect axially loaded cell. The load cells were calibrated in a 120,000-lb capacity Baldwin testing machine.

Many additional details concerning the design of the experiments as well as the design and operation of the test apparatus are given in reports written by Witzell (10) and Houmard (11). Results of the first phases of this research program are included in other papers (12, 13).

ACKNOWLEDGMENTS

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The authors are indebted to the members of the sponsoring research committee for many helpful suggestions and to William E. Witzell, George Fait, and James E. Houmard, graduate students in the department, for their assistance in planning and carrying out this research investigation.

REFERENCES

1. Lindsay, J.D., "A Ten-Year Report on the Illinois Continuously Reinforced Pavement." HRB Bull. 214 (1959).
2. Van Breemen, W., "Ten-Year Report on Continuously Reinforced Concrete Pavements in New Jersey." HRB Bull. 214, (1959).
3. Tremper, B., "Continuously Reinforced Concrete Pavement in California after Eight Years Service." HRB Bull. 214, (1959).
4. Taylor, I.J., and Eney, W.J., "First Year Performance Report on Continuously Reinforced Concrete Pavement in Pennsylvania." HRB Bull. 214, (1959).
5. Cashell, H.D., and Teske, W.E., "Continuous Reinforcement in Concrete Pavements." HRB Proceedings, Vol. 34, (1955).
6. Zuk, William, "Analysis of Special Problems in Continuously Reinforced Concrete Pavements." HRB Bull. 214, (1959).
7. Miller, M.M., and Gutzwiller, M.J., "The Mechanics of Continuously Reinforced Pavements." HRB Bull. 238, (1960).
8. Gutzwiller, M.J., and Waling, J.L., "Laboratory Research on Pavements Continuously Reinforced with Deformed Bars." American Concrete Institute, (Feb. 1959).
9. Concrete Pavement Design, Portland Cement Association, Chicago (1951).
10. Witzell, William E., "Welded Wire Fabric Reinforcement for Concrete Pavements." Purdue University, unpublished MSCE thesis, (1956).
11. Houmard, James E., "Welded Wire Fabric Reinforcement for Concrete Pavements." AISI Committee on Welded Wire Fabric Reinforcement Research (1958).
12. Gutzwiller, M.J., and Waling, J.L., "Crack Formation in Continuously Reinforced Concrete Pavements," HRB Bull. 238, (1960).
13. Gutzwiller, M.J., and Waling, J.L., "Stresses and Deflections in Pavements Continuously Reinforced with Welded Wire Fabric," HRB Bull. 238, (1960).

Stresses and Deflections in Concrete Pavements Continuously Reinforced With Welded Wire Fabric

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Laboratory experiments on simulated continuously-reinforced concrete slabs are summarized, with experimental results pertaining to slab deflections, crack widths, and stresses in welded wire fabric reinforcement receiving most emphasis. Some of the significant findings of these laboratory experiments are compared with field observations reported in the literature and several criteria are suggested for optimum structural design of continuously-reinforced pavements.

Some of the conclusions reached as a result of this research, subject to the limitations imposed by the range of variables studied are:

1. In adequately-reinforced continuous pavements, temperature decreases of more than about 30 deg below casting temperature tend to increase the deflections due to vertical (wheel) loads. Temperature decreases less than 30 deg tend toward a slight decrease in deflections.
2. The percentage of mid-depth reinforcement has an influence on maximum deflections due to vertical loads; the maximum deflections vary somewhat inversely with the percentage of reinforcement.
3. Upper surface crack widths vary somewhat linearly with temperature decreases in slabs reinforced with inadequate amounts of welded wire fabric but with adequate splice laps. Pavements with adequate amounts of reinforcement form new cracks during sizable temperature decreases and old cracks do not continue to widen in direct proportion to temperature drop.
4. Maximum active crack widths (due to temperature drops and wheel loads) at the upper and lower surfaces of the slab can be equalized and minimized by proper placement of the steel reinforcement. Laboratory experiments suggest that top and bottom surface active crack widths might be approximately equalized by placement of the fabric about 3/4 in. below mid-depth.
5. An increase in the average steel stress at a crack accompanies increased temperature drops (below concrete casting temperature); furthermore the stresses vary somewhat inversely with percentage of longitudinal reinforcement at mid-depth.
6. Vertical loads contribute significantly to stresses in the reinforcement.
7. Reinforcement placed 1-1/2 in. above mid-depth must resist stresses considerably greater than the same amount of reinforcement placed at mid-depth or below.

● RESEARCH ON continuously-reinforced pavements already summarized (1) indicates the great interest in this relatively new type of pavement which shows promise of providing smooth riding qualities and low maintenance costs over a long service period. The apparent advantages of this type of pavement design are adequate incentive to additional research directed toward the determination of rational procedures for the design of continuously-reinforced pavements.

Field experiments on continuously-reinforced pavements have included studies of pavement deflections including crack widths (2, 3, 4, 5, 6) and stresses in the steel reinforcement (2, 4, 5). Pavement deflections and crack widths must be the basis for important design criteria. Steel stresses, since they are repetitive in nature, must be maintained below the endurance limit of the reinforcement.

The recognized purpose of continuous longitudinal reinforcement in pavements is to control the formation of cracks and to hold the cracks tightly closed. Such closed cracks prevent the easy entry of water and soil particles and provide adequate aggregate interlock to transfer the loads and forces across the cracks. After the formation of these fine cracks (7), the pavement is no longer a continuous structural unit but acts as a series of relatively rigid slab segments connected by relatively flexible reinforced cracked sections (8, 9). Stresses in the steel reinforcement are critical at cracks.

Deflections of pavements are closely associated with subgrade pumping. Although not all subgrades are susceptible to pumping, it is apparent that pavement deflections must occur in some magnitude to initiate pumping of those subgrades which are susceptible to such deterioration.

A complete knowledge of the combined effects of temperature changes and wheel loads on reinforcement stresses, slab deflections, and crack widths for slabs reinforced with various amounts of steel in various positions and resting on subgrades of various stiffnesses must be obtained in order to establish rational criteria for the design of continuously-reinforced pavement slabs.

PURPOSE AND SCOPE

The objectives of this paper are:

1. To report the results of a series of laboratory experiments on simulated continuously-reinforced concrete slabs, with emphasis on those results pertaining to slab deflections, crack widths, and stresses in the continuous welded wire fabric reinforcement.
2. To compare the results of these laboratory experiments with reported field data.

TABLE 1

SPECIMEN CHARACTERISTICS

Slab No.	Reinforcement[1]	Percent Reinforcement	Position of Reinforcement below top	Reinforcement[2] Ultimate Strength, psi	Concrete Compressive Strength, psi
1	WWF 6 x 12 0/3	0.154	4 inches	93,400	5360
2	WWF 6 x 12 00000/0	0.300	4 inches	80,400	4720
3	WWF 4 x 12 00000/0	0.450	4 inches	80,400	4620
4	WWF 4 x 12 00000/0	0.450	5-1/2 inches	80,400	4320
5	WWF 4 x 12 00000/0	0.450	2-1/2 inches	80,400	4450

[1] WWF is abbreviation for welded wire fabric.

[2] Section 4(a) of ASTM Specification A82-34 for Cold Drawn Steel Wire for Concrete Reinforcement specifies yield stress to be 0.8 ultimate tensile strength. Section 4(d) states that "the yield point shall be determined by the drop of the beam or halt in the gage of the testing machine. In case no definite drop of the beam or halt in the gage is observed until final rupture occurs, the test shall be construed as meeting the requirement for yield point in Paragraph (a)." Thus, while the wire reinforcement had no definite yield point, it satisfied the specification. Specification A82-34 has been in effect during the entire time of this project.

3. To arrive at tentative criteria for the design of continuously-reinforced pavements as suggested by this study of stresses and deflections (including crack widths) in pavements continuously reinforced with welded wire fabric.

In using the results herein presented, the limitation imposed by the range of variables introduced in this laboratory should be kept in mind. This range of variables, which is outlined in the following section, will be expanded through the continuation of this research; future results may substantially add to, or otherwise alter, the findings reported here. One important criterion for the design of continuously-reinforced concrete pavements has resulted from the study of crack formation in such pavements (7).

LABORATORY EXPERIMENTS AND RESULTS

Specimens for these experiments were concrete slabs 8 in. thick, 3 ft wide, and 28 ft long (1), reinforced with welded wire fabric. The controlled variables included in the experiments were percentage of longitudinal reinforcement, position of reinforcement, range of simulated temperature drop after casting of the concrete (longitudinal forces), and magnitude and position of simulated wheel loads. The slabs tested and the magnitude assigned to each variable for the various slabs are summarized in Table 1. The subgrade modulus was held constant at 155-160 pci through the use of a rubber subgrade.

Each slab specimen contained three preformed transverse cracks spaced 5 ft apart (Fig. 1). Vertical loads were applied at seven locations on the longitudinal centerline of slabs 1 and 2 and at eight locations on slabs 3, 4, and 5 (Fig. 1).

The general procedure for loading the specimens and collecting the data was as follows:

1. A set of strain and deflection readings was taken with no load on the slab. Vertical loads of 5,000, 10,000, and 15,000 lb were then applied at position No. 1 and a set of readings was taken for each load. This procedure was repeated for vertical load positions No. 2 through 8, successively (2 through 7 for slabs 1 and 2).

2. A set of strain and deflection readings was taken with no load on the slab. An initial longitudinal load of 10,000 lb was applied and a set of readings was taken while holding this longitudinal load constant, the vertical loadings were repeated at positions No. 1 through No. 8 (No. 7, slabs 1 and 2), successively with appropriate data collection, and the loads were then removed.

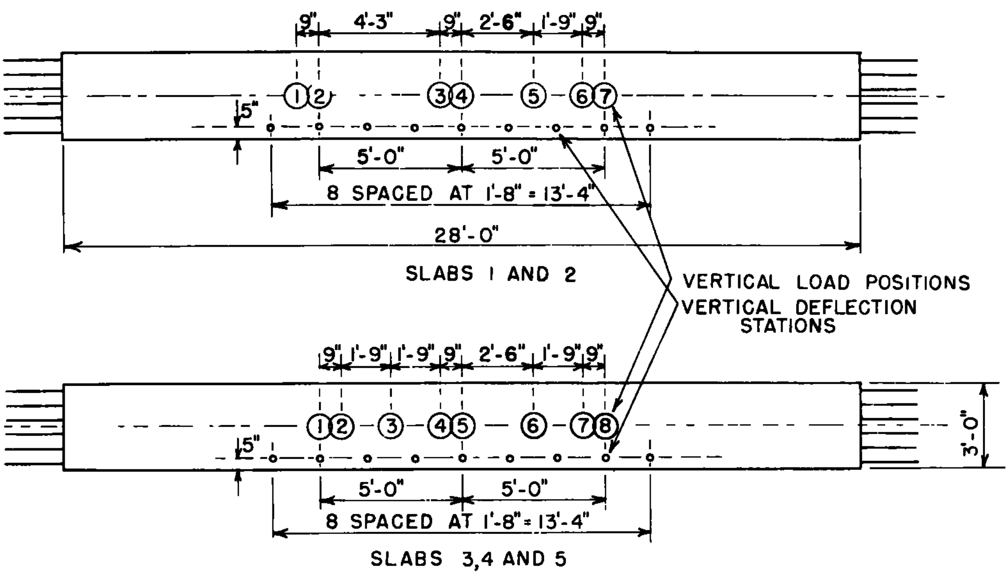

Figure 1. Position of vertical loadings and measurements of vertical deflections.

3. The horizontal load was increased in increments and step 2 was repeated for each increment. In general, 10,000-lb increments were used.

As a trial, the loading sequence was altered slightly for slab 2. This slab was tested continuously without removing the longitudinal load after each set of vertical loads was applied. The longitudinal load was simply increased by the 10,000-lb increment before the next set of vertical loads was applied.

During the entire loading sequence, measurement of concrete strains and crack widths was made at the upper surface of each slab. Steel plugs had been embedded in the surface in the test area of each slab at 10-in. intervals (Fig. 2). Gage holes were drilled in these plugs and a 10-in. Whittenmore strain gage was used to make surface strain and crack width measurements. For an uncracked gage length the average unit strain was taken as the change in gage length divided by ten. When a transverse crack occurred between two plugs, it was assumed that the total change in that gage length was due to a change in the crack width. This continuous set of gage lengths thus made it possible to determine the over-all change in length of the test region due to various combinations of horizontal and vertical loads.

The longitudinal tensile loads simulated the effect of temperature drops. Correlation of the magnitude of longitudinal load to temperature drop was determined by calculations. The over-all increase in length of the central test region of a slab due to longitudinal load, as determined by the surface strain readings, was equated to the decrease in length of the same span of reinforced concrete due to a temperature drop, Δt. Thus the condition of effective full restraint against longitudinal movement in a continuously-reinforced slab away from the end regions was assumed.

$$e = \alpha L \Delta t \qquad (1a).$$

or

$$\Delta t = \frac{e}{\alpha L} \qquad (1b)$$

in which

e = elongation (inches) of central test region
L = length (inches) of central test region
α = coefficient of linear contraction, assumed to be 0.000006 in./in./degree F for both steel and concrete.
Δt = temperature drop (degrees F)

Further details concerning the calculations necessary for the correlation of longitudinal forces and temperature decrease may be found elsewhere (7).

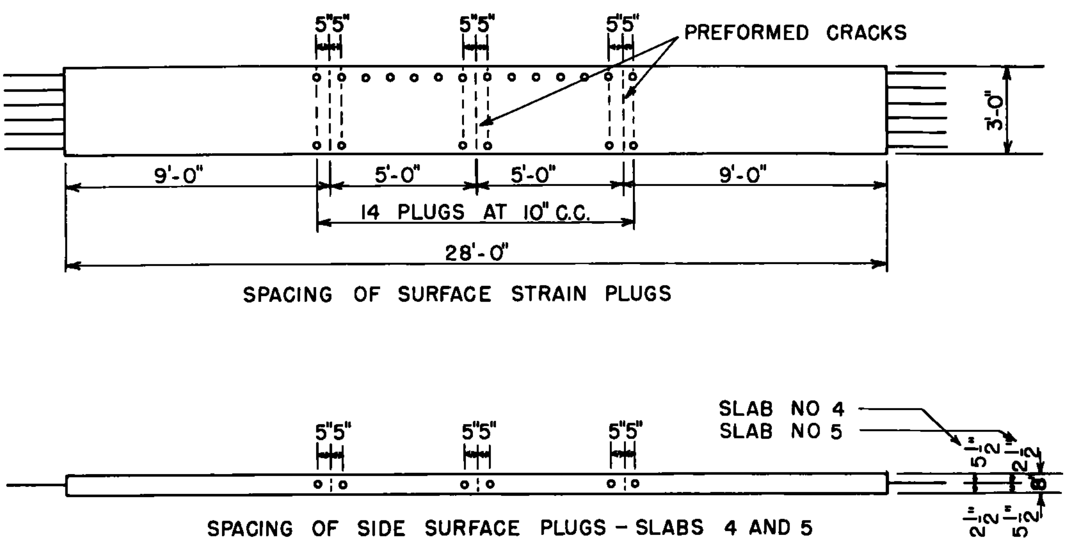

Figure 2.

Vertical Deflections

It is generally agreed that continuously-reinforced concrete pavements without joints are less susceptible to the damaging effects of pumping than rigid pavements of other current designs, but that under certain conditions of subgrade type and traffic they are not immune from pumping action at the free edge of the pavement. The possibility of such pumping may be reduced through proper pavement design to control the vertical movements of the slab under vertical loading.

Vertical deflections of each slab were measured at nine stations (Fig. 1). To indicate the effects of vertical loads on slab deflections, each slab was assumed to be straight when subjected to the various longitudinal loads with no vertical loads acting. The additional deflections due to 5-, 10-, and 15-kip vertical loads placed at each of the load positions were then measured. A series of graphs similar to that shown in Figure 3 was drawn for each slab subjected to a full range of the longitudinal loads which simulated temperature drops. These served as a basis for determining average values of maximum deflections of preformed cracks and for obtaining the magnitudes of angle changes at cracked sections of the slabs.

A graphical indication of the influence of temperature drop and percentage reinforcement on maximum vertical deflections due to 15-kip wheel loads is given in Figure 4. The data points are the average of the maximum deflections of the three preformed cracks when each was loaded in turn by the 15-kip simulated wheel load, with the slab subjected to various longitudinal loads (temperature drops). The three slabs which contained different percentages of longitudinal reinforcement sustained deflections ranging from 0.07 to 0.10 in. in magnitude throughout the total range of simulated temperature drops. At the greatest temperature drop, slab 3 with the greatest amount of longitudinal reinforcement (0.450 percent) showed about 18 percent less deflection than slab 2 (0.300 percent steel) and about 26 percent less deflection than slab 1 (0.154 percent steel).

The influence of vertical location of the reinforcement on deflections due to wheel loads is shown in Figure 5. Of the three slabs which contained the same percentage of steel at three different locations, slab 5 with its reinforcement 1-1/2 in. above mid-depth allowed fewer deflections than the slab reinforced at mid-depth; furthermore the

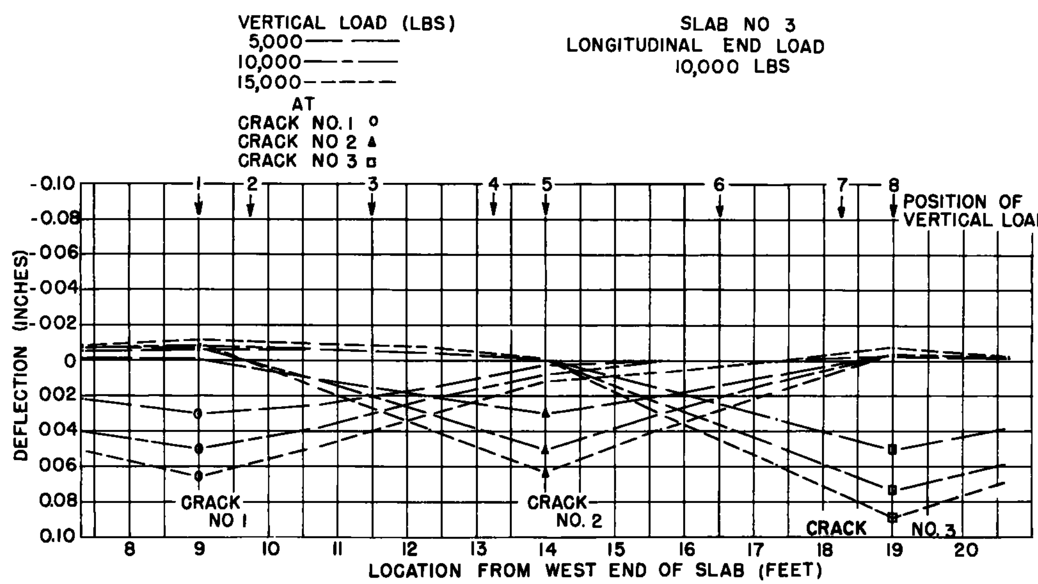

Figure 3. Vertical deflection of slab due to combined loading at a crack.

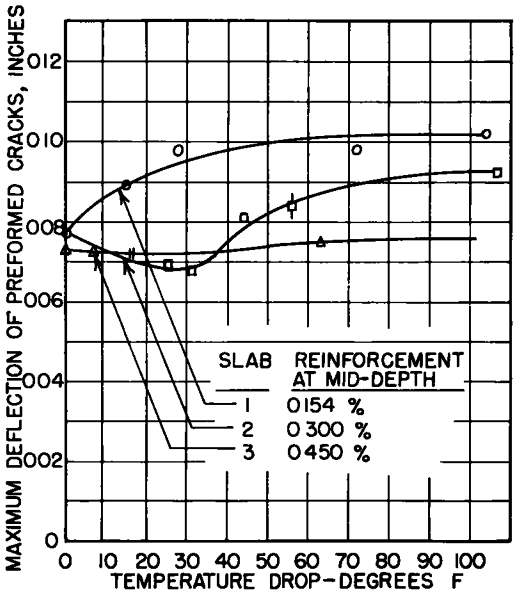

Figure 4. Influence of temperature drop and percentage of reinforcement on deflections due to 15-kip wheel loads.

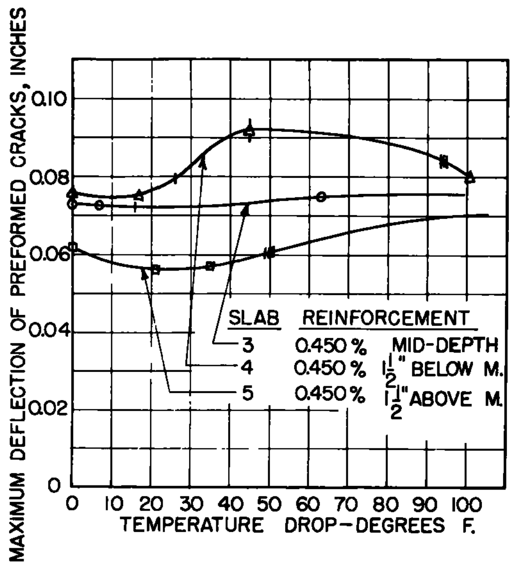

Figure 5. Influence of temperature drop and position of reinforcement on deflections due to 15-kip wheel loads.

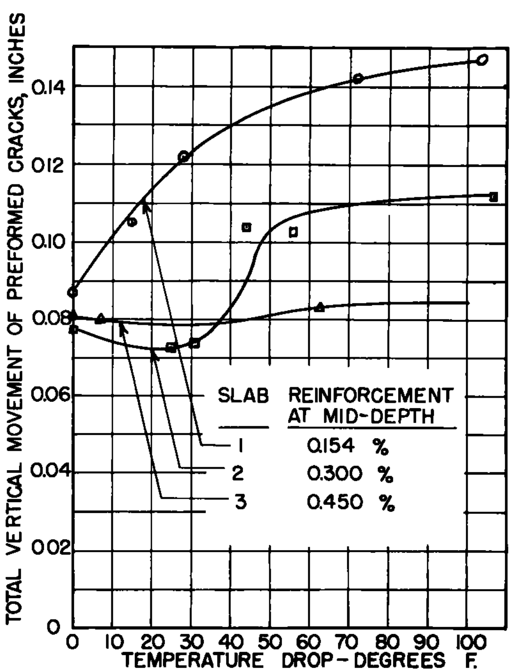

Figure 6. Influence of temperature drop and percentage of reinforcement on total vertical movement due to 15-kip wheel loads.

slab reinforced at 1-1/2 in. below mid-depth allowed more deflection than the other two, throughout the full range of 100 deg of temperature decrease. This result was undoubtedly influenced by the presence of the double percentage of steel at the two outside preformed cracks, introduced by lapping of the fabric at these two cracks.

Downward deflections under simulated wheel loads were usually accompanied by some negative deflection (upheaval) at adjacent cracks. Thus under a moving wheel load, each crack section was subjected to vertical movements greater in magnitude than the maximum downward deflection. The total vertical movement of cracks is probably more indicative of the possibility of initiating pavement pumping than is maximum downward deflection of cracks.

Figure 6 shows the influence of temperature drop and percentage reinforcement on total vertical movements due to 15-kip wheel loads. The data for this graph were also obtained by averaging the total vertical movements of the three preformed cracks. Slab 3 with its 0.450 percent longitudinal reinforcement offered more restraint against total vertical movement at temperature drops greater than 40 deg than slabs 1 and 2 which are considered (7) to have been under-reinforced. The increase in total vertical movements of cracks with increased temperature drops was negligibly small in slab 3, but was of significance in slabs 1 and 2.

The effect of position of 0.450 percent longitudinal reinforcement on total vertical movement of cracks is shown in Figure 7. In this respect, it appears that there is negligible basis for choice between mid-depth and above-center reinforcement, both having furnished some more restraint against total vertical movements throughout the greater portion of temperature drops than the low-level reinforcement.

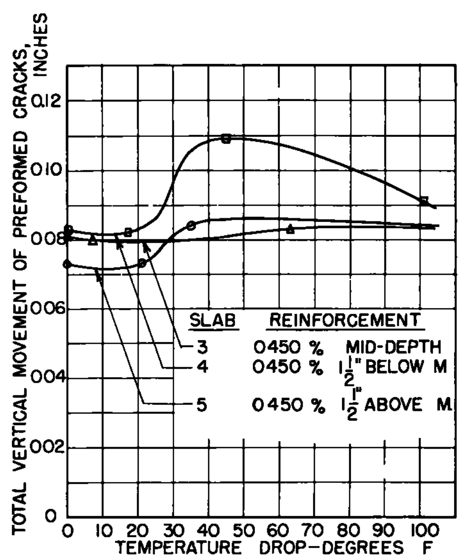

Figure 7. Influence of temperature drop and position of reinforcement on total vertical movement due to 15-kip wheel loads.

Crack Widths

All engineers interested in design or research on continuously-reinforced concrete pavements are concerned with studies of crack formation and crack widths in the concrete slabs. It is generally recognized that the function of the longitudinal steel is to maintain the cracks, which must necessarily form, in a tightly-closed condition so that fine-grained soils cannot enter the cracks from either the top or the bottom. In discussing the probability of blowups in continuously-reinforced pavements, Woolley (10) cited progressive spalling due to the accumulation of dirt in cracks, from the top and perhaps also from the bottom of the slab, as the first step toward blowups. He reasoned that with time, as a result of spalling, the compression across a crack, which develops with rising temperature, must be carried by a progressively smaller section of concrete than the original full depth. Blowups then occur as a natural consequence.

Thus, it is generally agreed that in properly designed continuously-reinforced pavements, the cracks are held so tight that soil cannot get into them and spalling does not occur to an appreciable extent. Therefore, one key to optimum design is the control of crack widths.

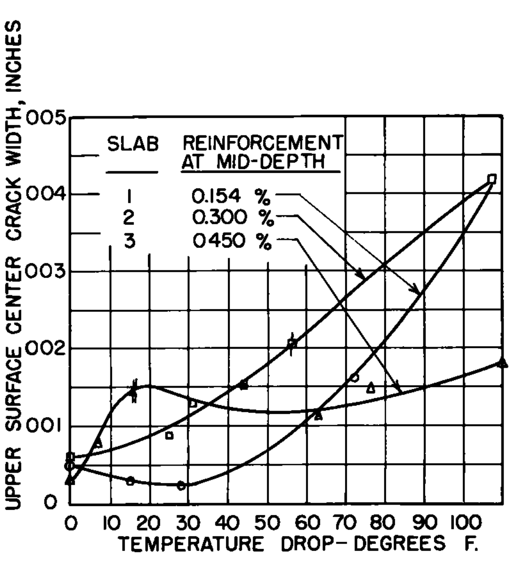

Figure 8. Influence of temperature drop and percentage of reinforcement on upper surface crack widths.

In the slabs tested, the center preformed crack carried a single layer of welded wire fabric. Splices were placed at (or near, in slab 1) the two outer preformed cracks (15- and 16-in. laps in slab 1, and 2-ft and 4-ft laps in slabs 2 through 5). The center crack widths were considered to be representative of most cracks which would occur in a pavement slab and were therefore studied in detail; however, it was found that the splices influenced somewhat the widths of adjacent cracks.

Crack width data obtained from the upper surface strain plugs with the slabs subjected to longitudinal loads (temperature drops) only is sampled on the graphs in Figure 8. This shows the variation in the upper surface center crack widths with temperature drops for slabs 1, 2, and 3.

The center crack openings for slab 1 were relieved by an early semi-yielding of the lap splices at the adjacent cracks. This accounts for the small crack openings plotted for slab 1 in the lower range of temperature decrease. Slab 3, with 0.450 percent longitudinal reinforcement developed two additional cracks at an early 16 deg temperature change; this relieved the center crack of further opening over a long range of temperature drop. In fact, no appreciable increase in this center crack width resulted until a total temperature change of about 90 deg. Except for the results of slab 1, which were effected by the semi-yielding of the outer cracks, it is seen that, through the greater range of temperature drops, crack widths varied somewhat inversely with percentage of longitudinal reinforcement.

The influence of temperature drops and position of reinforcement on upper surface crack widths are shown in Figure 9. Again the beneficial effects of early formation of additional cracks is seen. The center crack of slab 3, which had 0.450 percent steel at mid-depth and which developed the two additional cracks at 16 deg decrease, was maintained more tightly closed than the center crack of either slab 4 with the same steel below mid-depth or slab 5 with its steel above mid-depth. It is also seen that the center crack of slab 4 closed somewhat after the formation of six additional cracks in or near the test region of the slab.

In the design of a continuously-reinforced pavement, the upper surface crack widths due to temperature changes alone are of less importance than top and bottom surface maximum crack widths due to temperature changes and wheel loads. As the wheel loads move along the pavement the slab deflects and the top and bottom crack widths vary over significant ranges. Water and soil can work into cracks at the upper surface under the action of gravity and at the bottom surface under the pressure developed between the slab and the subgrade. Thus it is important to minimize active crack widths at both top and bottom surfaces of the slab.

The laboratory experiments involved measurement of the variation of upper surface crack widths with temperature changes and wheel loads for those slabs reinforced by steel at mid-depth. In addition, crack widths on the sides of the slabs at the elevation of the steel were measured on the slabs reinforced with steel below and above mid-depth. Good estimates of bottom surface crack widths were obtained by calculations. For slabs reinforced with steel below (or above) mid-depth, top surface and side crack width data for temperature changes alone indicated cracks increasing in width with the distance from the steel as shown exaggerated in Figure 10a. The measured crack width

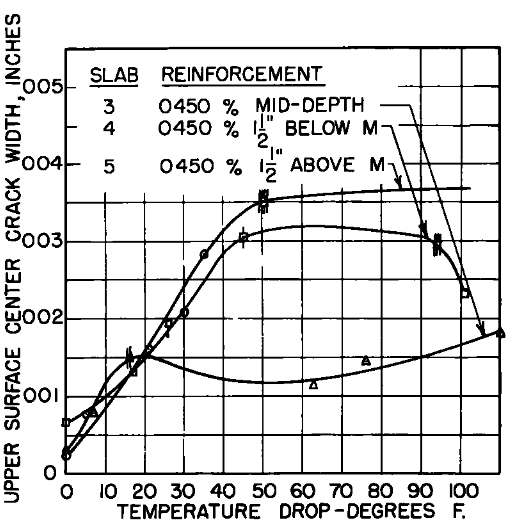

Figure 9. Influence of temperature drop and position of reinforcement on upper surface crack widths.

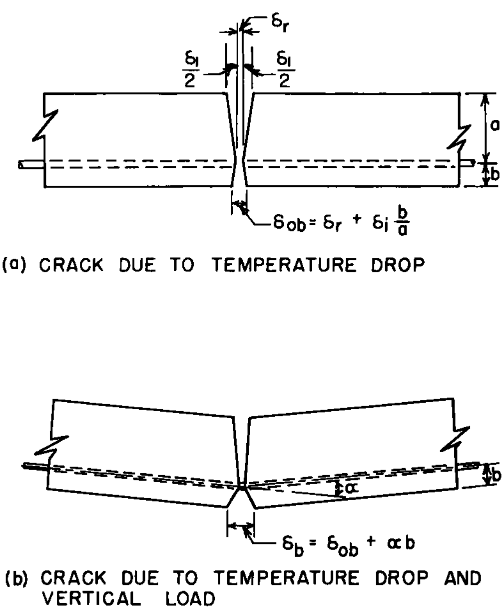

Figure 10. Deformations at a crack.

at the elevation of reinforcement is δ_r. The upper surface crack width ($\delta_r + \delta_i$) was also measured. It was assumed that the increase in crack width both above and below the reinforcement was linear. Thus the crack width δ_{ob} at the bottom surface due to temperature alone was

$$\delta_{ob} = \delta_r + \delta_i \frac{b}{a} \tag{2}$$

Applications of vertical loads changed the upper and lower crack widths appreciably. From the deflection diagrams (similar to Fig. 2), angle changes due to 15-kip wheel loads were scaled accurately for the center preformed crack for each horizontal load on each slab. These angle changes were assumed to be centered at the reinforcement in the cracked section. When multiplied by the distance of the reinforcement from the lower surface, they gave the additional bottom surface crack widths, δ_α , due to the angle changes induced by the wheel loading. Thus, the total bottom surface crack width, δ_b, due to temperature change and wheel loading is (Fig. 10b)

$$\delta_b = \delta_{ob} + \delta_\alpha = \delta_r + \delta_i \frac{b}{a} + \alpha b \tag{3}$$

For the special case of mid-depth reinforcement, the ratio b/a is equal to unity and the first two terms of the bottom surface crack width (the last member of the equality) are the same as the top surface crack width due to temperature drops alone. That is to say, for mid-depth reinforcement it is assumed that due to temperature changes alone, top surface crack widths are equal to bottom surface crack widths.

Figure 11 outlines the influence of temperature decrease and position of steel on maximum upper and lower surface crack widths due to 15-kip wheel loads. A comparison of Figures 9 and 11 shows that wheel loads did have appreciable effect on the magnitudes of maximum top surface crack widths. Figure 11 shows that bottom surface crack widths were considerably greater than top surface crack widths in simulated continuous pavement slabs reinforced at mid-depth and above, while top surface crack openings were greater than bottom surface crack openings in the slab reinforced at 1-1/2 in. below mid-depth. A study of these curves suggests that top and bottom surface active crack widths might be approximately equalized by placement of the steel about 3/4 in. below mid-depth.

Steel Stresses

In continuous concrete pavements reinforced with welded wire fabric, the longitudinal wires are subjected primarily to uniaxial tensile or compressive stresses. Although this longitudinal steel does transfer a portion of the loads across cracks in the concrete, its primary contribution is made by holding the cracks tightly closed so that concrete aggregate interlock resists the greater portion of the shear forces at a crack.

To determine the stresses in the longitudinal wires of the welded wire fabric at cracks, several SR-4 type A-7 electrical strain gages were applied to the wires at each of the preformed cracks as previously described (1). Gages were also placed on the longitudinal wires between cracks to determine the role of the reinforcement in unbroken segments of the concrete. Strain gage readings were taken during the experiments in the same sequence as the deflection and crack width measurements. Strains were multiplied

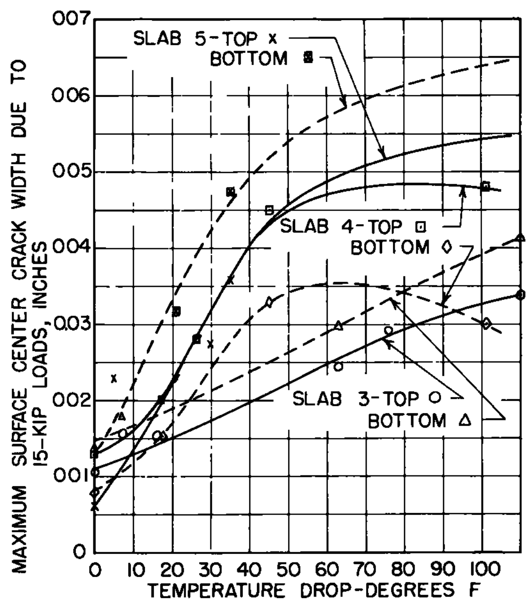

Figure 11. Influence of temperature drop and position of reinforcement on maximum surface crack widths due to 15-kip wheel loads.

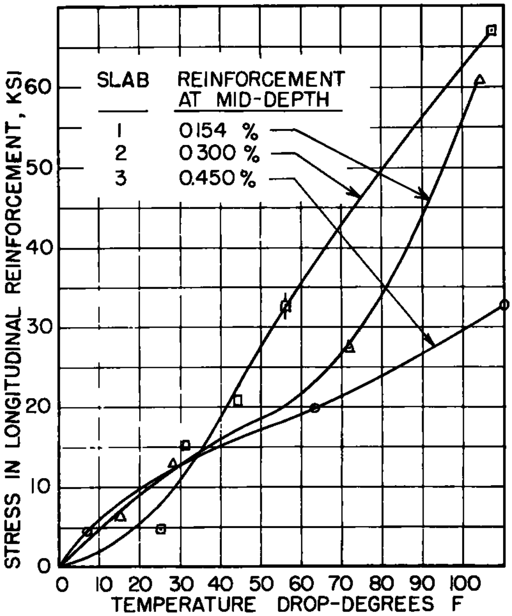

Figure 12. Influence of temperature drop and percentage of reinforcement on stress in steel at center crack.

directly by the modulus of elasticity to obtain the steel stresses.

Significant differences were obtained between the steel stresses at cracks and those farther away than one transverse wire from the cracks; the latter were always quite small while the stresses at the cracks were of such magnitude as to require special attention in the design of a continuously-reinforced concrete pavement.

Variations in steel strains, some as great as 20 percent from the average, were obtained from the several gages at a single crack, under a given load arrangement. These variations were probably due to unequal loss of bond as a result of strain gage waterproofing and normal variations in anchorage furnished by the welded transverse wires. Furthermore, the irregular shape of some of the cracked transverse sections of the slabs may have caused uneven pressures to develop in the concrete at cracks under vertical loads. Obviously, exact values of either maximum or average stresses in the wires at cracks cannot be predicted; however, it is practically as important to know the order of magnitude of stresses as to know the exact values. Average values serve as an adequate basis for comparison of the various percentages and positions of reinforcement in arriving at optimum pavement design.

To obtain representative values of steel stresses at cracks, the eight strain gage measurements at the center crack were averaged. Figure 12 shows the variation in longitudinal steel stress due to the simulated temperature changes only, for the various percentages of reinforcement. These curves show, in general, an increase in steel stress with increased temperature drops and, except for slab 1 in which the adjacent cracks semi-yielded, show a trend in favor of an increase in percentage of reinforcement for temperature drops greater than 35 deg. The position of the fabric reinforcement had considerable influence on the maximum steel stresses at cracks due to temperature changes alone (Fig. 13). The mid-depth reinforcement of slab 3 was subjected to a little less stress than the lowered reinforcement of slab 4 at temperature drops greater than 35 deg and to considerably less stress than the high reinforcement of slab 5 at all temperature ranges. The stress relief induced in slab 3 by the formation of two additional cracks at 16 deg temperature drop is apparent.

Wheel loads contribute significantly to the stresses in the reinforcement of continuously-reinforced pavements. Figure 14 shows the combined effect of 15-kip wheel loads and temperature decreases

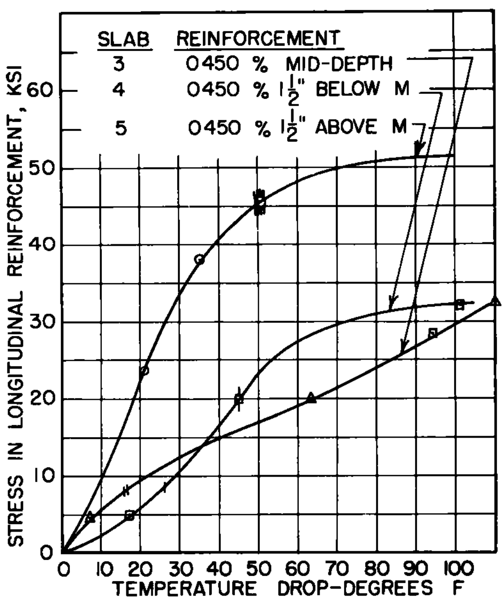

Figure 13. Influence of temperature drop and position of reinforcement on stress in steel at center crack.

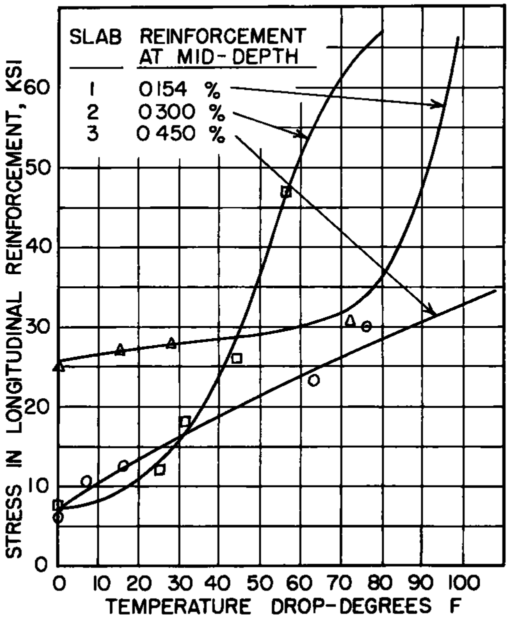

Figure 14. Influence of temperature drop and percentage of reinforcement on stress in steel at center crack due to 15-kip wheel loads.

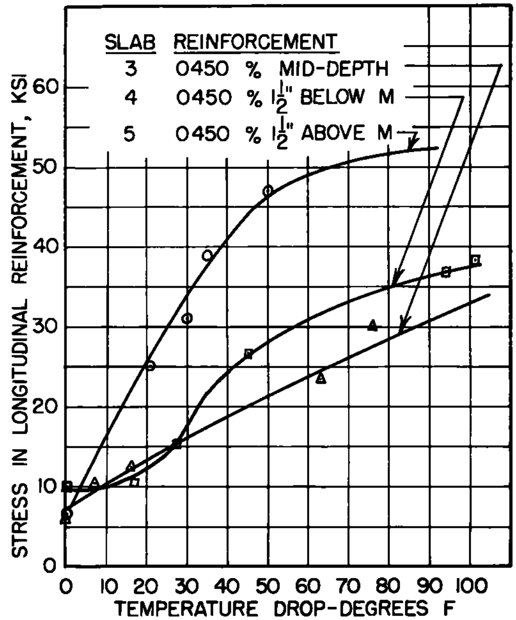

Figure 15. Influence of temperature drop and position of reinforcement on stress in steel at center crack due to 15-kip wheel loads.

on the steel stresses at the center cracks for the various percentages of longitudinal steel. This set of curves shows definitely the inadequacy of the 0.150 and 0.300 percent steel of slabs 1 and 2, respectively. The 0.450 percent reinforcement of slab 3 appears adequate in regard to stresses. Here, also, the influence of the early formation of additional cracks in slab 3 was beneficial. The position of the reinforcement had considerable influence upon the maximum steel stresses at cracks due to temperature changes and 15-kip wheel loads. Figure 15 shows that slab 3, with its reinforcement at mid-depth was subjected to slightly fewer steel stresses than slab 4 with its reinforcement 1-1/2 in. below mid-depth. Both slabs 3 and 4 were subjected to considerably fewer steel stresses than slab 5 with its reinforcement 1-1/2 in. above mid-depth. The early formation of the two additional cracks in the test region of slab 3 undoubtedly contributed to sufficient stress relief to make the steel stresses fall below those of slab 4. One might normally expect the resulting curve for slab 3 to fall between the curves for slabs 4 and 5, at least in the region of large temperature decreases.

Additional details concerning the results of these experiments may be found in research reports by Witzell (18) and Houmard (19).

CORRELATION OF LABORATORY RESULTS WITH FIELD TESTS

Field experiments with continuously-reinforced concrete pavements have included a limited number of test pavements reinforced with welded wire fabric. The Indiana (Stilesville) test road (11) included two test sections 310 ft long reinforced with fabric to provide 0.42 percent longitudinal steel placed at 2-1/2 in. below the top surface of a standard 9-7-9 in. thickened edge pavement. Other shorter sections were also included. The New Jersey test road (12) included two sections 5,430 and 5,130 ft long reinforced with fabric giving 0.90 and 0.72 percent longitudinal steel in slabs 8 and 10 in. thick, respectively. That fabric was placed in two layers by the strike-off method.

Very little, if any, data on pavement deflections due to wheel loadings were produced by the reported field experiments on continuously-reinforced pavements. Investigators (13) have experienced difficulties in establishing adequate datum planes for such field measurements, due to the compressibility of considerable depth of subgrade. Field experiments on continuous reinforcement of pavement slabs have, in general, been directed toward other more pressing problems.

The variation of widths of crack openings due to temperature changes has been studied in several of the field experiments. At the end of ten years of service, the Indiana (Stilesville) test road (14) showed crack widths ranging from 0.02 to 0.06 in. with an average of 0.038 in. in the lane reinforced with 0.45 percent steel and carrying light traffic. These crack openings were measured in the fall of the year when the mean pavement temperature was 58 to 60 deg F. Those measurements were made in such a manner as to include raveling and rounding of the edges of the concrete at cracks. Widths of cracks measured at mid-depth on the edge face of the pavement to exclude raveling and rounding showed real widths of less magnitude than the upper surface crack widths. These measurements, made with a microscope in the fall when the mean pavement temperature was 73 to 74 deg F, showed crack widths ranging between 0.007 and 0.010 in. in the same section of pavement previously mentioned. Similar data for other percentages of reinforcement showed that the crack widths, for a given temperature, increased with a decrease in the percentage of longitudinal reinforcement.

The three-year performance report on the Illinois (15) continuously-reinforced pavement stated that 7-in. and 8-in. thick slabs reinforced with 0.3, 0.5, 0.7, and 1.0 percent steel had developed transverse cracks ranging in width from 0.007 to 0.021 in. by August 1950. Measurements taken after three years of service from the New Jersey (16) experimental pavement showed a maximum crack width of 0.019 in. and an average crack width of 0.015 in. in the outside lane of an 8-in. pavement, at a net temperature drop of 31 deg. This pavement was longitudinally reinforced with 0.90 percent steel.

Thus, although no direct correlation can be made between the crack widths obtained in the laboratory experiments and the various field experiments, it is seen that the crack width measurements made in the laboratory were in the same order of magnitude as those observed in the field.

Changes in crack widths with passage of wheel loads have not been reported from the field experiments. Therefore, no correlation can be made between the laboratory and field experiments in this important phase of the experiments.

Reinforcement steel stresses have been measured in the Illinois (15), California (17), and Pennsylvania (5) experiments. Stresses as great as 62,000 and 42,000 psi were reported for the Illinois experiment for 7-in. and 8-in. slabs reinforced with 0.7 percent steel. Stresses as great as 50,000 and 80,000 psi were reported from the California experiment at the end of six months after casting the concrete in 8-in. slabs reinforced by 0.62 and 0.50 percent steel, respectively. Stresses exceeding the yield strength of the steel were reported at two months and three months after casting the pavement at York, Pennsylvania. This pavement contains 0.50 percent longitudinal steel.

The California report (17) contained the following statements in regard to reinforcement steel stresses which are of special interest:

> At early ages (less than 3 months) the readings on pairs of SR-4 gages opposite each other on the same reinforcing bar were in agreement within 10 percent. There was, however, a noticeable increase in the variations in readings at 3 months and at 6 months.

As a result of the laboratory experiments, it is believed that such variations are to be expected as a result of unequal stress build-up due to temperature changes and wheel loads rather than as a direct result of the passage of time.

It can be seen that the several field experiments yielded a variety of magnitudes of crack widths and stresses in the various types of reinforcement due to temperature changes. These variations can be attributed to such influencing variables as subgrade stiffness, concrete casting temperatures, and subsequent moisture conditions. The crack widths and steel strains reported for the field experiments were of the same order of magnitude as those obtained in these laboratory experiments.

Data on crack widths and steel stresses due to wheel loads as well as temperature changes have not been reported from the several experiments.

DESIGN CRITERIA

A study of laboratory data on vertical deflections, crack widths, and reinforcement steel stresses, on an individual basis, suggests the following criteria for the design of continuously-reinforced pavements:

1. For the temperature range between the temperature of casting of the concrete and the freezing temperature of the subgrade, the total vertical movement composed of downward deflection under wheel loads and upheaval as the loads move away should be minimized as nearly as practicable.
2. For the same practical temperature range delineated in criterion 1, the upper surface and lower surface crack widths should be considered equally important and the maximum active crack widths due to wheel loads as well as temperature-changes at both the upper and lower surface should be minimized as nearly as practicable.
3. For the practical temperature range, the maximum reinforcement steel stress (average) due to temperature changes and wheel loads should be maintained below the yield strength of the steel with as great a margin as is economically feasible.

In general, all three of these design criteria cannot be completely satisfied by a single choice of amount and position of reinforcement. Furthermore, economic considerations may place practical restrictions on the designer in his choice of concrete thickness and steel reinforcement for continuous pavements which might make these criteria unattainable; thus the criteria must serve as a guide for compromise in the choice of design characteristics of continuously-reinforced pavements.

A previous study (7) of formation of cracks in continuous pavements has suggested another criterion which should be satisfied along with the three listed above. It can be summarized by the following statement:

4. Sufficient amount of longitudinal reinforcing steel must be provided to maintain all transverse cracks in a tightly closed condition without adverse effects on the concrete. For mid-depth reinforcement, this condition is met when increasing longitudinal forces caused by temperature drops, in combination with live loads, tend to cause additional cracks to form rather than to cause excessive opening of existing cracks.

These four criteria can be applied in judging the relative merits and acceptability of the five slabs included in the laboratory experiments. It must be remembered, however, that the experiments reported here have included to date only slabs 8 in. thick resting on a subgrade having a modulus of 160 pci. Thus the optimum pavement design may not be represented among these slabs. Table 2 summarizes the application of the four criteria to the slabs tested.

Examination of Table 2 reveals the general inadequacy of the light reinforcement in slabs 1 and 2. Slab 5 with its reinforcement 1-1/2 in. above mid-depth is rated inadequate because of excessive lower surface active crack widths and because of the high steel stresses. Slab 3 with its reinforcement at mid-depth proved to be somewhat better in every respect than slab 4 with its reinforcement 1-1/2 in. below mid-depth except for the item of formation of cracks. Thus, from the five specimens tested to simulate 8-in. continuous concrete pavements reinforced with welded wire fabric and resting on subgrades having a modulus of 160 pci, reinforcement with 0.450 percent longitudinal steel placed at mid-depth appears to represent the most economical design and best performance. However, if additional slabs were tested, perhaps 0.450 percent reinforcement placed about 3/4 in. below mid-depth might prove advantageous.

CONCLUSIONS

As a result of these laboratory experiments on continuous pavements reinforced with welded wire fabric, and within the limitations imposed by the range of variables studied, the following conclusions are drawn:

TABLE 2

SPECIMEN RATINGS

Slab No.	Percent Longitudinal Reinforcement	Position of Reinforcement	Rating* - According to Criteria			
			1. Vertical Movement	2. Crack Width	3. Steel Stress	4. Crack Formation
1	0.154	Mid-depth	D	D	D	D
2	0.300	Mid-depth	C	D	D	C
3	0.450	Mid-depth	A	A	A	B
4	0.450	1-1/2 in. below M.	B	B	B	A**
5	0.450	1-1/2 in. above M.	A	D	C	A**

* Rating Definitions:
A - satisfies criterion completely.
B - satisfies criterion nearly completely in comparison with other specimens.
C - satisfies criterion partially - less than other specimens.
D - Criterion rules out the design completely.
** - Criterion not yet intended to apply to slab reinforcement placed other than at mid-depth.

1. In adequately-reinforced continuous pavements, temperature decreases of more than about 30 deg below casting temperature tend to increase the deflections due to vertical (wheel) loads. Temperature decreases less than 30 deg tend to cause slight decrease in deflections.

2. The percentage of mid-depth reinforcement has an influence on maximum deflections due to vertical loads; the maximum deflections vary somewhat inversely with the percentage of reinforcement.

3. Total vertical movements (downward deflection plus upheaval) of the pavements due to live loads vary with temperature changes and percentage of reinforcement much the same as deflections.

4. Total vertical movements of the slabs reinforced at mid-depth and above are about the same; both are of less magnitude than the total movement of the slab reinforced below mid-depth.

5. Upper surface crack widths vary somewhat linearly with temperature decreases in slabs reinforced with inadequate amounts of welded wire fabric, but with adequate splice laps. Pavements with adequate amounts of reinforcement form new cracks during sizable temperature decreases and old cracks do not continue to widen in direct proportion to temperature drop.

6. The upper surface control of crack widths due to temperature changes alone appears to be accomplished the best by mid-depth reinforcement.

7. Maximum active crack widths (due to temperature drops and wheel loads) at the upper and lower surfaces of the slab can be equalized and minimized by proper placement of the steel reinforcement. These experiments suggest that top and bottom surface active crack widths might be approximately equalized by placement of the fabric about 3/4 in. below mid-depth.

8. Significant differences exist between the steel stresses at cracks and those farther away than one transverse wire from the cracks; the latter are quite small while the the stresses at cracks are of such magnitude as to require special attention in the design of a continuously-reinforced concrete pavement.

9. Variations in steel strains (and stresses), some as great as 20 percent, exist among the several longitudinal wires of the fabric at a single crack, under given sizable temperature changes and live loads.

10. An increase in the average steel stress at a crack accompanies increased temperature drops (below concrete casting temperature); furthermore the stresses vary somewhat inversely with percentage of longitudinal reinforcement at mid-depth.

11. Position of reinforcement influences the steel stresses at cracks due to temperature drops alone. Reinforcement placed 1-1/2 in. above mid-depth is subjected to stresses considerably greater than that placed at mid-depth or below.

12. Vertical loads contribute significantly to stresses in the reinforcement.

13. An increase in the percentage of steel placed at mid-depth is accompanied by a decrease in the average steel stress at a crack due to a given temperature decrease and live load.

14. Reinforcement placed 1-1/2 in. above mid-depth must resist stresses, due to temperature drops and live loads, considerably greater than the same amount of reinforcement placed at mid-depth or below.

15. These experiments yield results pertaining to crack widths and steel stresses which are of the same order of magnitude as those reported from field experiments, although exact conditions of any one field test are not duplicated in the laboratory.

16. Three criteria are suggested as guides in the choice of design characteristics of continuous pavements. These criteria are supplemented by one derived elsewhere (7).

17. Application of the four suggested design criteria indicates that slab 3 simulating a continuous pavement reinforced with 0.450 percent longitudinal steel at mid-depth was the best of those tested. Reinforcement at 1-1/2 in. below mid-depth rated second best. Perhaps additional experiments might prove that 0.450 percent reinforcement placed about 3/4 in. below mid-depth might prove advantageous.

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REFERENCES

1. Gutzwiller, M.J., and Waling, J.L., "Laboratory Research on Concrete Pavements Continuously Reinforced With Welded Wire Fabric," HRB Bulletin 238 (1960).
2. Lindsay, J.D., "A Ten-Year Report on the Illinois Continuously-Reinforced Pavement," HRB Bull. 214 (1959).
3. Van Breemen, W., "Ten-Year Report on Continuously-Reinforced Concrete Pavements in New Jersey," HRB Bull. 214 (1959).
4. Tremper, B., "Continuously-Reinforced Concrete Pavement in California After Eight Years Service," HRB Bull. 214 (1959).
5. Taylor, I.J., and Eney, W.J., "First-Year Performance Report on Continuously-Reinforced Concrete Pavements in Pennsylvania," HRB Bull. 214 (1959).
6. Cashell, H.D., and Teske, W.F., "Continuous Reinforcement in Concrete Pavements," HRB Proceedings, Vol. 34 (1955).
7. Gutzwiller, M.J., and Waling, J.L., "Crack Formation in Continuously-Reinforced Pavements," HRB Bull. 238 (1960).
8. Zuk, William, "Analysis of Special Problems in Continuously-Reinforced Concrete Pavements," HRB Bull. 214 (1959).

9. Miller, M. M., and Gutzwiller, M. J., "Mechanics of Continuously-Reinforced Concrete Pavements," HRB Bull. 238 (1960).
10. Woolley, W. R., "Continuously Reinforced Concrete Pavements Without Joints," HRB Proceedings, Vol. 27 (1947).
11. Sutherland, E. C., and Benham, S. W., "Experiments With Continuous Reinforcement in Concrete Pavements," Public Roads, 20:11 (January 1940).
12. Van Breemen, William, "Preliminary Report of Current Experiment With Continuous Reinforcement in New Jersey," HRB Proceedings, Vol. 27 (1947).
13. Geldmacher, R., et al., "Subgrade Support Characteristics as Indicated by Measurements of Deflection and Strain," HRB Proceedings, Vol. 36 (1957).
14. Cashell, H. D., and Benham, S. W., "Experiments with Continuous Reinforcement in Concrete Pavements," HRB Proceedings, Vol. 29 (1949).
15. Russell, H. W., and Lindsay, J. D., "Three Year Performance Report on Experimental Continuously Reinforced Concrete Pavement in Illinois," HRB Proceedings, Vol. 30 (1950).
16. Van Breemen, William, "Report on Experiment with Continuous Reinforcement in Concrete Pavement—New Jersey," HRB Proceedings, Vol. 30 (1950).
17. Stanton, T. E., "Reports on Experiments with Continuous Reinforcement in Concrete Pavements—California," HRB Proceedings, Vol. 30 (1950).
18. Witzell, W. E., "Welded Wire Fabric Reinforcement in Concrete Pavements," Purdue University, unpublished MSCE thesis (1956).
19. Houmard, J. E., "Welded Wire Fabric Reinforcement in Concrete Pavements," AISI Committee on Welded Wire Fabric Reinforcement Research (1958).

Crack Formation in Continuously-Reinforced Pavements

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The mechanics of crack formation in continuously reinforced pavement slabs, as understood from the literature on several field test pavements, are summarized.

Results of a series of laboratory experiments on simulated continuously reinforced concrete slabs are given, with those results pertaining to the formation of cracks being emphasized. The findings of these laboratory experiments are correlated with the field observations reported in the literature and one criterion for the design of continuously reinforced pavements is suggested.

Some of the more important conclusions reached as a result of this research, subject to the limitations imposed by the range of variables studied, are as follows:

1. The effects of concrete shrinkage, temperature changes, and wheel loads on crack formation in continuously reinforced pavements can be adequately simulated in the laboratory without producing actual temperature changes in an infinitely long pavement slab.
2. The formation of a complete crack pattern in a continuously reinforced pavement is the result of a superposition of the effects of concrete shrinkage, temperature changes, and live (wheel) loads. Perhaps the importance of live loads has been somewhat underestimated in past discussions of this problem.
3. Crack spacing in a continuously reinforced pavement with some age and use varies inversely with the percentage of longitudinal steel reinforcement, if the steel is placed at mid-depth, with a minimum average crack spacing of about 2 ft in a slab having 0.768 percent longitudinal steel.
4. It is generally agreed that a continuously reinforced pavement slab must be reinforced with longitudinal steel in sufficient amount to maintain all transverse cracks in a tightly closed condition. For mid-depth reinforcement this condition is met when increasing longitudinal forces caused by temperature drops, in combination with live loads, tend to cause additional cracks rather than to open existing cracks forever wider. This standard is suggested as one criterion (among others) for the design of continuously reinforced pavements.
5. The suggested design criterion is satisfied for an 8-in. pavement resting on a subgrade having a modulus of 160 pci, by 0.45 percent longitudinal reinforcement either in the form of deformed bars or welded wire fabric reinforcement placed at mid-depth in the slab.

● THE MECHANICS of crack formation in continuously reinforced pavements is qualitatively understood as a result of long-term observation of the several test strips of such pavements now in existance (1, 2, 3, 4, 6). The formation of the cracks has, in fact, received more attention that any other phase of the experiments — correctly so, since the spacing and configuration of the cracks in continuously reinforced pavements form the common denominator of the other important design parameters such as crack widths, pavement deflections, and steel stresses. Cracks are generally considered to have been formed in these pavements by the combination of concrete shrinkage during curing and climatological temperature changes during and after the curing period.

Longitudinal reinforcement steel in a concrete slab offers restraint against shrinkage of the concrete. The tendency of concrete to shrink builds up longitudinal tension in the concrete and compression in the steel. If the tensile stress reaches the magnitude of the tensile strength of the concrete a crack forms, thus relieving the longitudinal stresses at and near the crack. This action presupposes the presence of adequate steel to resist the compressive force induced in the steel by the shrinking concrete. An infinitely long, adequately reinforced slab will crack at fairly regular intervals due to such concrete shrinkage.

Climatological temperature changes tend to cause similar volume changes of concrete and steel; that is, the coefficient of linear thermal expansion of the two materials is on the same order of magnitude, and an increase in temperature above casting temperature tends to make both materials expand about the same. However, a long continuously reinforced concrete slab is, in all regions at some distance from the slab ends, effectively restrained against longitudinal expansion, and the tendency of the materials to expand tends to close any transverse cracks and to build up longitudinal compressive stresses in the two materials. Since both materials are quite strong in compression, no damage normally comes from such expansion. On the other hand, a decrease in temperature tends to cause both materials to shorten. The effective restraint against this contraction tends to widen any existing transverse cracks, to build up tensile forces in the steel at cracks, to induce tensile forces in the concrete and steel between cracks, and to develop bond stresses between the steel and concrete near the cracks. The tensile stress in the concrete between cracks may reach the magnitude of the tensile strength of the concrete and thus form additional cracks.

The formation of additional cracks by temperature decrease completely relieves the concrete of tensile stress only at the cracks. The flexural action between cracks in slab segments caused by vertical loads (wheel loads) on the pavement superimposes longitudinal compressive and tensile stresses on the upper and lower portions of the slab, respectively, directly under the load and vice versa, but of lesser magnitude, some distance away from the load. The superposition of these flexural stresses upon the temperature and shrinkage stresses may cause the tensile strength of the concrete to be surpassed and may thus form additional cracks.

The formation of the complete crack pattern in a given continuously reinforced pavement slab results from the superposition of concrete shrinkage, temperature changes, and wheel loadings. The first cracks develop as a result of shrinkage alone, more cracks are formed by the combination of shrinkage and temperature change, and the crack pattern finally becomes complete as the result of the combination of all three causes over a long period of time. In the literature, much emphasis has been given to the first two of these three primary causes of slab cracking, shrinkage and temperature changes.

PURPOSE AND SCOPE

The objectives of this paper are:

1. To report the results of a series of laboratory experiments on simulated continously reinforced concrete slabs, with emphasis on those results pertaining to the formation of cracks in the slabs.
2. To correlate the results of these laboratory experiments with reported field data on the cracking of continuously reinforced pavements.
3. To arrive at tentative criteria for the design of continuously reinforced pavements as suggested by this study of crack pattern formation.

In using the information included in this report, the limitations delineated by the range of the variables introduced in the laboratory experiments should be kept in mind. This range of variables, which is outlined in the following section, is being expanded

TABLE 1

DESCRIPTION OF SLAB SPECIMENS

Slab No.	Reinforcement[2]	Percent Reinforcement	Position of Reinforcement below top	Reinforcement[3] Yield Point Stress, psi	Reinforcement Ultimate Strength, psi	Concrete Compressive Strength, psi
1	D.B. #4 at 9" O.C.	0.278	4 inches	57,300	94,600	4800
2	D.B. #5 at 9" O.C.	0.430	4 inches	56,300	97,500	5200
3	D.B. #5 at 7" O.C.	0.533	4 inches	56,300	97,500	4270
4	D.B. #6 at 7" O.C.	0.768	4 inches	55,800	97,600	4730
5	D.B. #5 at 7" O.C.	0.533	6 inches	69,500	104,400	5690
6	D.B. #5 at 7" O.C.	0.533	5 inches	69,500	104,400	3500
7	WWF 6 x 12 0/3	0.154	4 inches	—	93,400	5360
8	WWF 6 x 12 00000/0	0.303	4 inches	—	80,400	4720
9	WWF 4 x 12 00000/0	0.450	4 inches	—	80,400	4620
10	WWF 4 x 12 00000/0	0.450	5-1/2 inches	—	80,400	4230
11	WWF 4 x 12 00000/0	0.450	2-1/2 inches	—	80,400	4450

[2] D.B. is abbreviation for deformed bars; WWF stands for welded wire fabric.

[3] Section f(a) of ASTM Specification A82-34 for Cold Drawn Steel Wire for Concrete Reinforcement specifies yield stress to be 0.8 ultimate tensile strength. Section 4(d) states that "the yield point shall be determined by the drop of the beam or halt in the gage of the testing machine. In case no definite drop of the beam or halt in the gage is observed until final rupture occurs, the test shall be construed as meeting the requirement for yield point in Paragraph (a)." Thus, while the wire reinforcement had no definite yield point, it satisfied the specification. Specification A82-34 has been in effect during the entire time of this project.

through the continuation of this research program; future results may substantially add to or otherwise alter the findings reported here.

Important criteria for the design of continuously reinforced concrete pavements will result from the studies of stresses in the reinforcement and the deflections of the slabs. Results of such studies are reported elsewhere for laboratory experimental slabs continuously reinforced with deformed bars (7) and welded wire fabirc (8).

LABORATORY EXPERIMENTS AND RESULTS

The specimens for these experiments were reinforced concrete slabs 8 in. thick, 3 ft wide, and 28 ft long (9, 10). The controlled variables included in the experiments were type of reinforcement, percentage of longitudinal reinforcement, position of reinforcement, range of simulated temperature drop after casting of the concrete, and magnitude and position of simulated wheel loading. The partially controlled variables included primarily the strength and stiffness of the concrete. The slabs tested and the magnitude assigned to each variable for the various slabs are summarized in Table 1. The subgrade modulus was maintained constant at 160 pci through the use of a rubber subgrade.

After each slab was cured, it was lowered onto the rubber subgrade and cracked through at each of the three weakened planes by slight non-uniform lifting of the slab. Thus, the central test region of each slab had preformed cracks at 5-ft intervals. This was done to be certain that cracks would occur at the location of SR-4 strain gages on the reinforcement steel, although it was anticipated from the results of reported field tests that additional cracks would develop during the laboratory experiments.

Vertical loads were applied to each slab successively at eight load points on slabs 1 through 6 and 9 through 11 and at seven load points on slabs 7 and 8. The locations of these vertical load points are shown in Figures 1 through 11. The vertical loads were applied in magnitudes of 5,000, 10,000 and 15,000 lb through bearing plates and rubber pads 12, 18, and 18 in. in diameter, respectively.

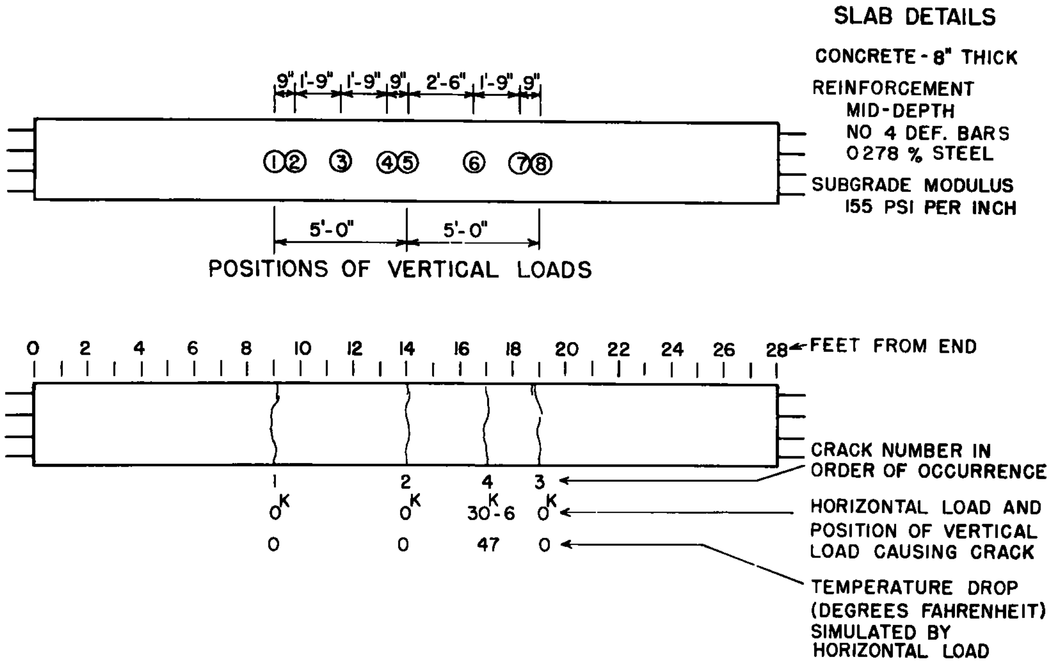

Figure 1. Crack formation — Slab 1.

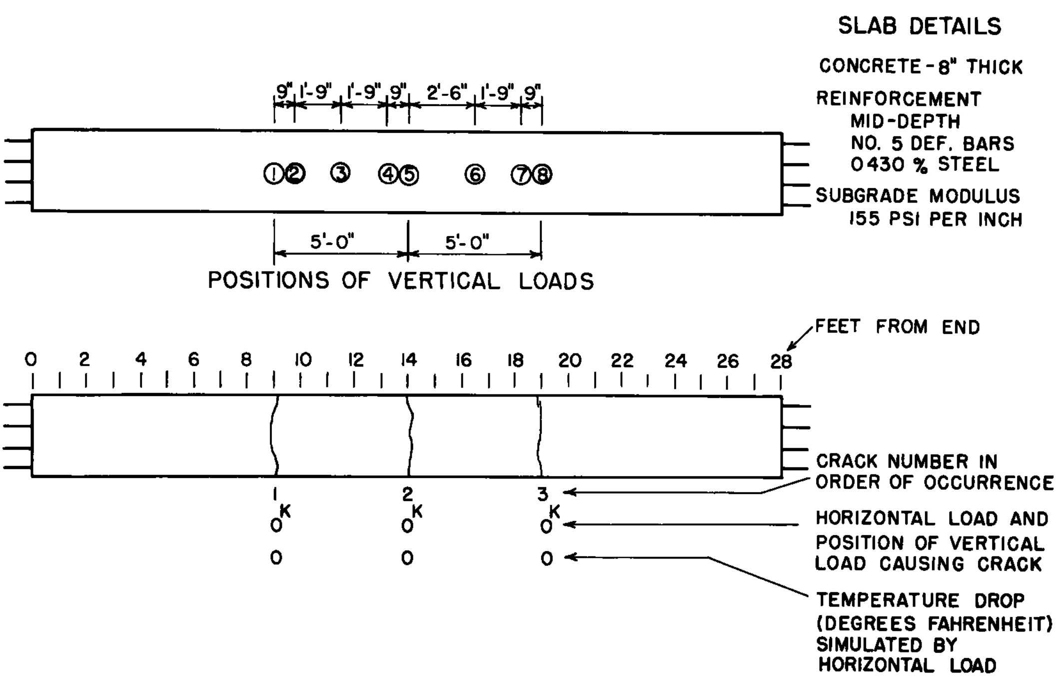

Figure 2. Crack formation — Slab 1.

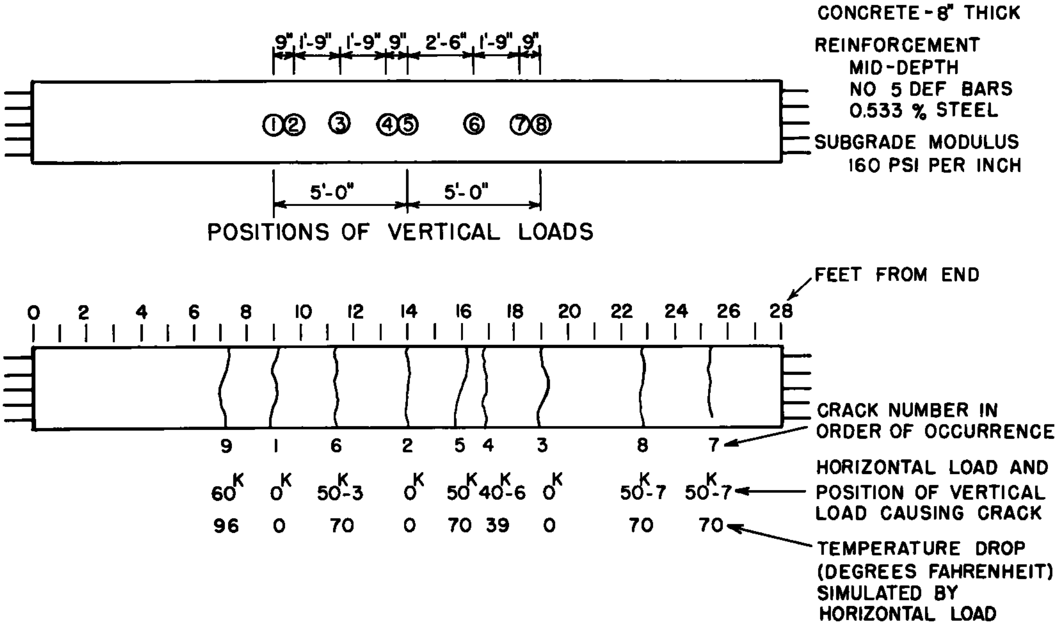

Figure 3. Crack formation — Slab 3.

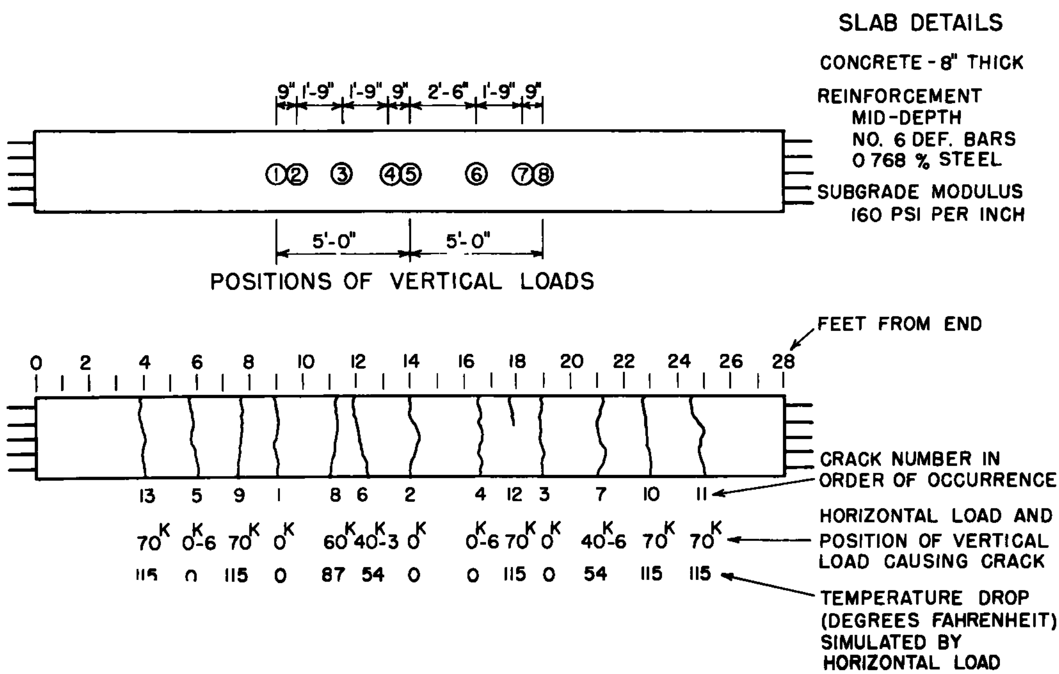

Figure 4. Crack formation — Slab 4.

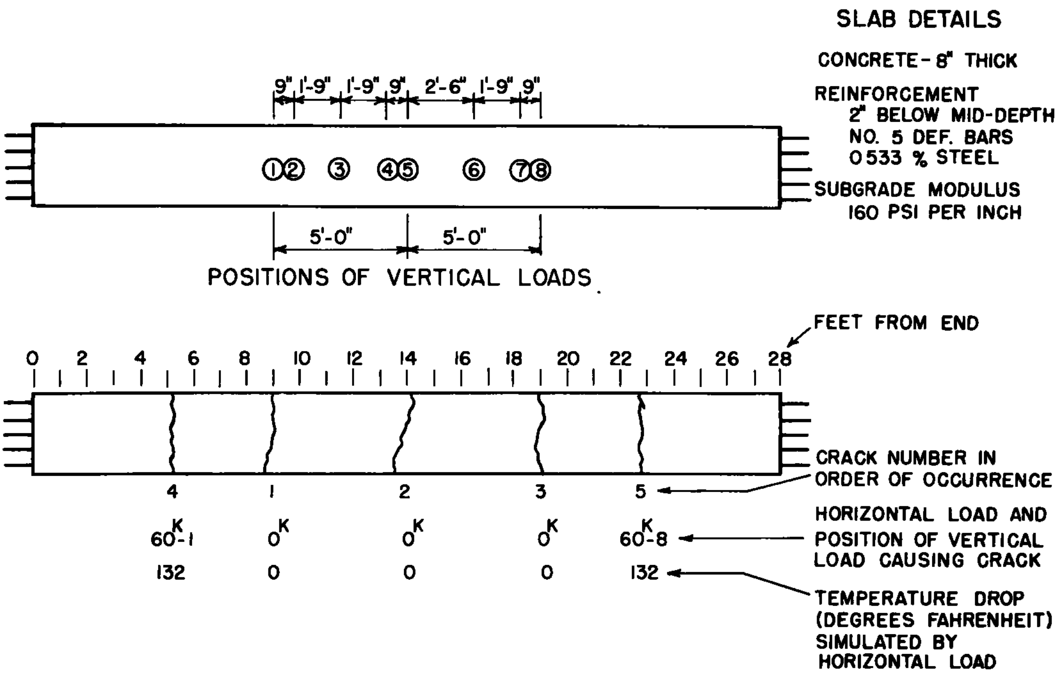

Figure 5. Crack formation — Slab 5.

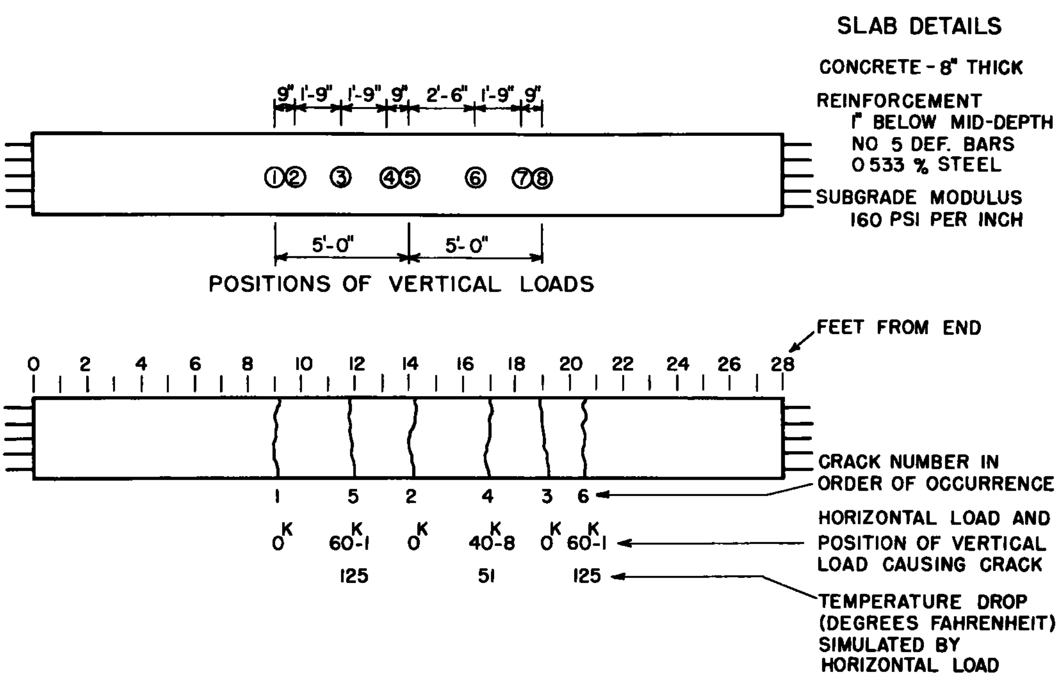

Figure 6. Crack formation — Slab 6.

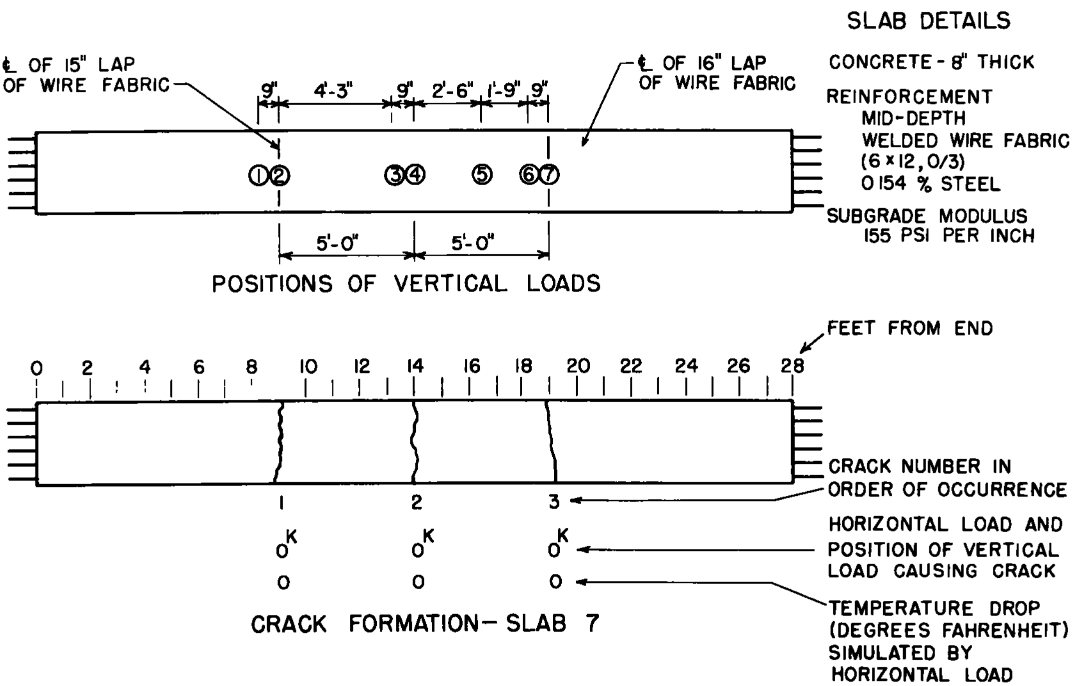

Figure 7. Crack formation — Slab 7.

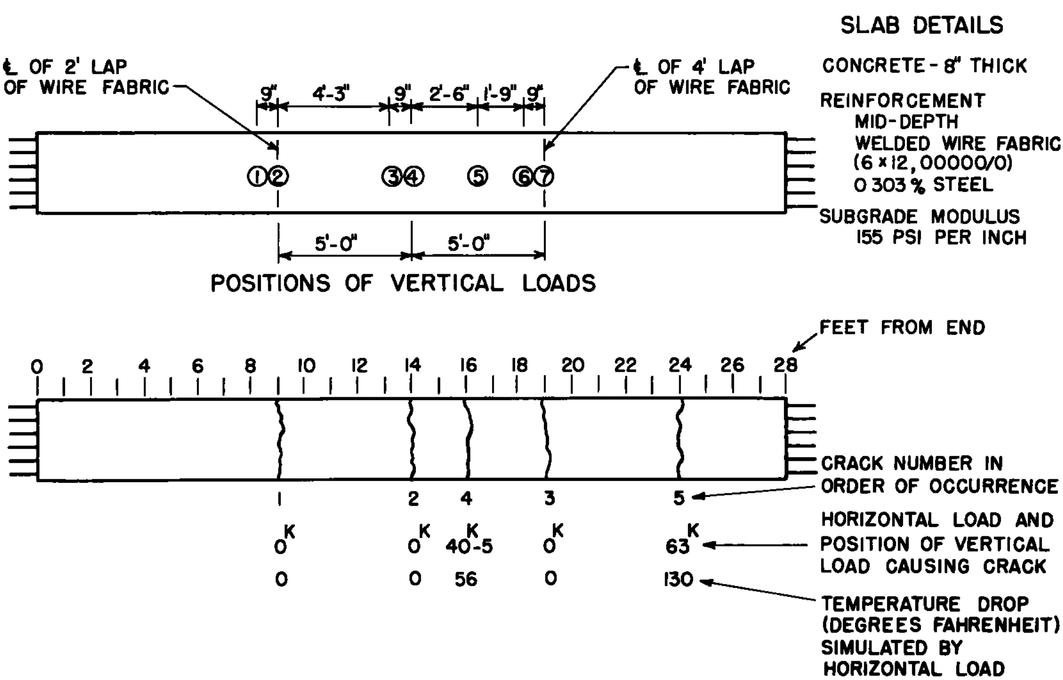

Figure 8. Crack formation — Slab 8.

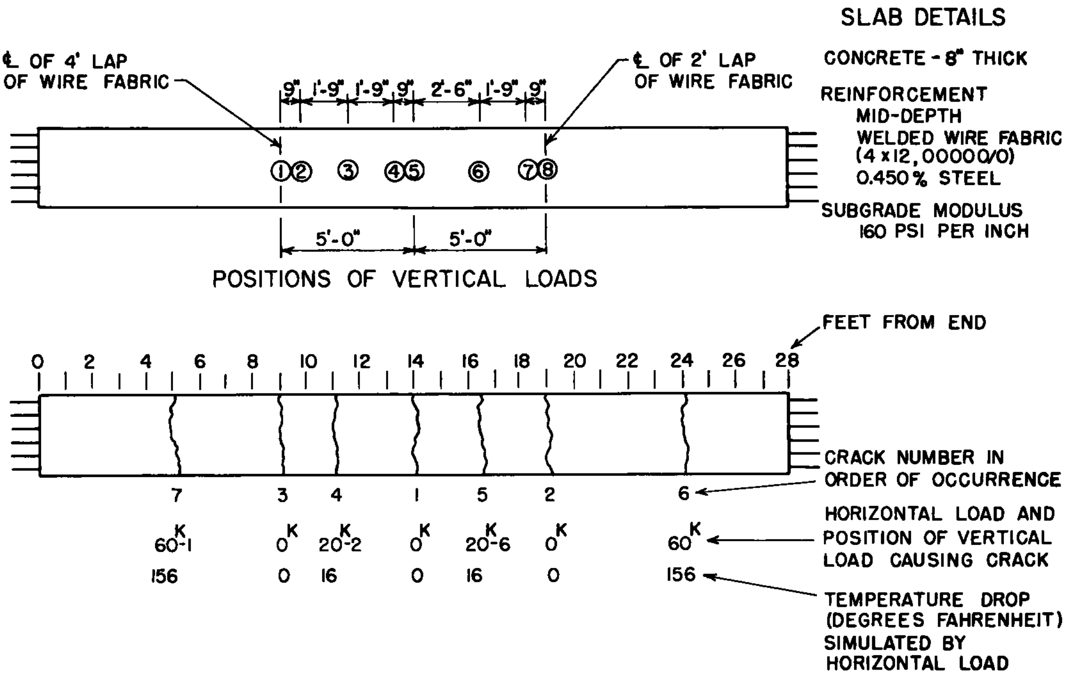

Figure 9. Crack formation — Slab 9.

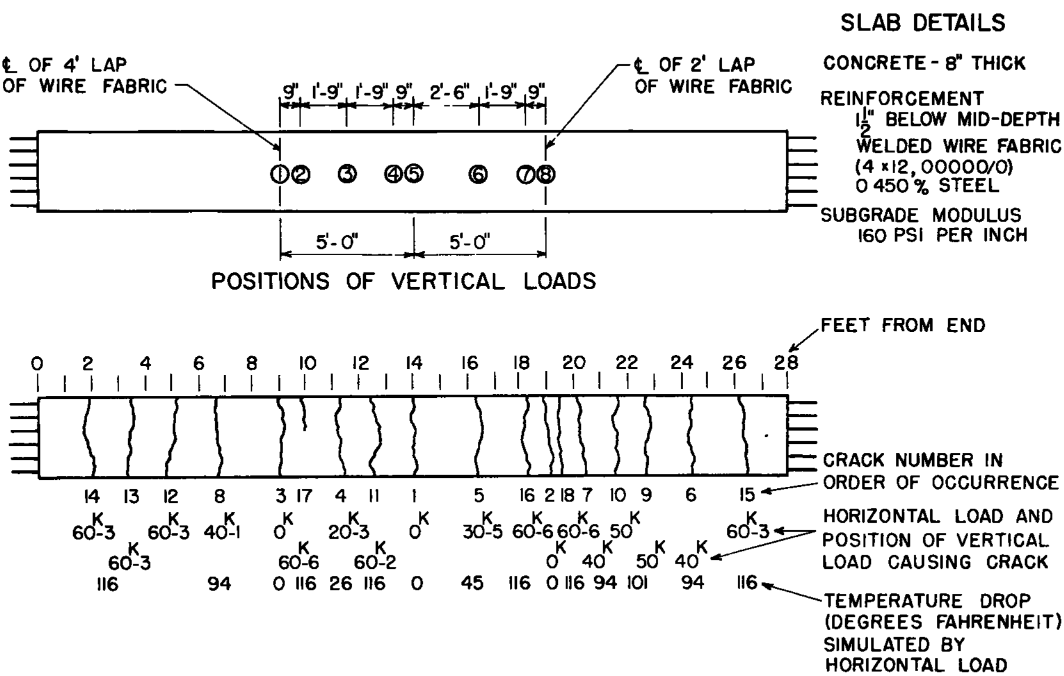

Figure 10. Crack formation — Slab 10.

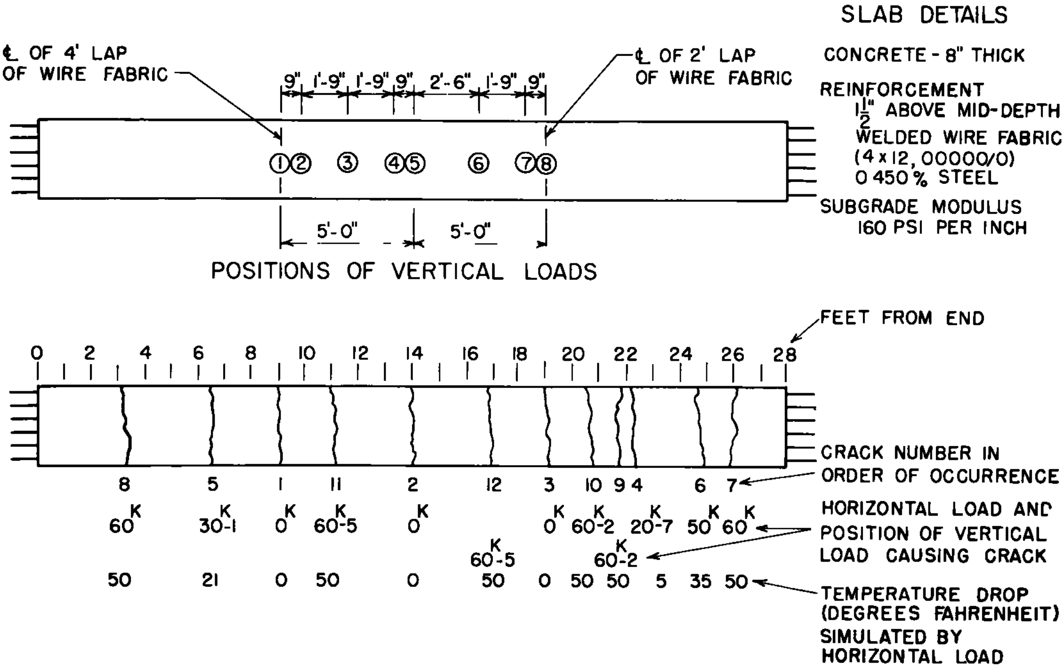

Figure 11. Crack formation — Slab 11.

In general, horizontal (longitudinal) loads were applied to each slab in increasing 10,000-lb increments, although 5,000-lb increments were used for slab 7 which had only 0.154 percent reinforcement. Horizontal loads were increased in each case until it became apparent that the longitudinal reinforcement steel had been stressed beyond its yield strength.

The usual sequence of applying combined vertical and longitudinal loads was as follows:

1. No longitudinal load — vertical loads (5, 10, and 15 kips) at load points 1 through 8 successively.
2. 10,000-lb longitudinal load — vertical load sequence repeated.
3. 20,000-lb longitudinal load — vertical load sequence repeated.
4. And so forth — to yielding of reinforcement.

During the entire loading sequence, measurements of concrete strain and crack widths were made at the upper surface of each slab. Steel plugs had been embedded in the surface of each slab in the test area at 10-in. intervals, longitudinally. Gage holes were drilled in these plugs and a 10-in. Whittemore strain gage was used to make surface strain and crack width measurements. For an uncracked gage length the average unit strain was taken as the change in the gage length divided by ten. When a transverse crack occurred between two plugs, it was assumed that the total change in that gage length was due to a change in the crack width. This continuous set of gage lengths thus made it possible to determine the over-all change in length of the 130-in. test region due to the various combinations of horizontal and vertical loads. Throughout the testing of each slab specimen, the formation of new cracks (in addition to the preformed cracks) was observed and recorded.

The longitudinal tensile loads simulated the effects of temperature drops. Correlation of the magnitude of longitudinal load to temperature drop was made by calculations. The over-all increase in length of the central test region of a slab due to longitudinal load was equated to the decrease in length of the same span of reinforced concrete due to a temperature drop, Δt. Thus, the condition of effective full restrain against longitudinal movement in a continuously reinforced slab away from the end regions was assumed.

$$e = \alpha L \Delta t$$

or

$$\Delta t = \frac{e}{\alpha L}$$

where

e = elongation (inches) of central test region
L = length (inches) of central test region
α = coefficient of linear contraction — assumed to be 0.000006 in./in./deg F for both steel and concrete
Δt = temperature drop (deg F)

Concrete curing shrinkage does not affect appreciably the foregoing computation. Shrinkage of the concrete in a continuously reinforced pavement slab tends to cause the length of concrete between adjacent cracks to shorten and the cracks to widen. In the measurement of the over-all length of a central region of slab the shortening of concrete and the widening of cracks compensate for each other. Shrinkage is accompanied by stress changes in the concrete and steel; however, these stress changes are nonuniform throughout the length, with tendencies toward tensile stresses in the steel at cracks, compressive stresses in the steel between cracks, tensile stresses in the concrete between cracks, and compatible bond stresses between the concrete and the steel. These alternating type stresses have no appreciable influence on the over-all change in length of a central test region and thus can be neglected in the temperature correlation calculations.

During the experiments, as the loading sequence was followed, each slab specimen was scrutinized carefully for the development of new cracks. New cracks would usually begin in the vicinity of the vertical load position and advance under increasing loads from the bottom to the top of the slab. The progress of crack formation was observed and marked with chalk on the side of the slab. Figures 1 through 11 summarize the cracking history of slabs 1 through 11, respectively. In each of these figures, the upper plan view shows the vertical (wheel) loading positions; the lower plan view shows the number of the cracks in the order of their occurrence, the horizontal load and the position of the vertical load causing each crack, and the temperature drop (deg F) simulated by the horizontal load.

Figures 1 through 4 show the effect of percentage of deformed bar reinforcement (placed at mid-depth) on the crack pattern. In the lightly reinforced slab (0.278 percent steel), one additional crack formed — at a 47-deg temperature drop with the vertical load at load point 6. Simulated temperature drops of 86 deg combined with vertical loads caused no more cracking. No cracks in addition to the three preformed ones formed in slab 2 which had 0.430 percent steel. Longitudinal loads simulating a 103-deg temperature drop, along with the vertical loads, only altered the widths of the preformed cracks in this slab. Slab 3 with 0.533 percent steel showed additional cracks at 39-deg drop and vertical load at point 6, 70-deg drop and no vertical load, 70-deg drop and vertical load at point 3, 70-deg drop and vertical load at point 7 (2 cracks), and 96-deg drop and no vertical load. Thus, slab 3 showed a total of nine cracks in a length of about 21 ft (average crack spacing 2.3 ft) after a simulated temperature drop of about 100 deg. In slab 4, which had 0.768 percent steel at mid-depth, two additional cracks developed during loading of point 6 with no simulated temperature drop, one developed at 54-deg drop with vertical load at point 3, one at 54-deg drop with vertical load at point 6, one at 87-deg drop without vertical load and five at 115-deg drop with no vertical load. Slab 4 contained a total of 13 cracks in a length of about 24 ft (average crack spacing about 2 ft) after a simulated temperature drop of 115 deg.

Thus, it is seen that within the range of variables studied, an increase in percentage of deformed bar steel reinforcement at mid-depth tends to result in a more closely spaced crack pattern in the slab, with individual crack widths of less magnitude. The two slabs having the lower percentages of steel (0.278 and 0.430) remained essentially with the preformed crack pattern. This indicates that one could expect crack spacings on the order of magnitude of 5 ft or more in slabs having these amounts of reinforcement if the experiments were conducted without preforming cracks in the slabs. The slabs with the higher percentages of steel showed a definite decrease in crack spacing (to about 2 to 2½ ft) with very little difference between the slab reinforced with 0.533 percent steel and that having 0.768 percent steel. It is doubtful that the crack spacing would average significantly less than 2 ft for any practicable amount of reinforcement.

The nature of the formation of individual cracks in the slabs reinforced at mid-depth is of interest. When one of the more significant horizontal loads (temperature drops) was applied to either slab 3 or 4, a crack would usually begin at the bottom of the slab under a 5,000-lb vertical load and would break completely through under the 10,000- or 15,000-lb vertical load. During the late stages of testing each of these slabs, several cracks appeared to be caused directly by the large horizontal loads being applied. The influence of vertical loads can be seen, however, since most of the cracks formed in the region of influence of vertical loads until the simulated temperature drop was extremely large.

Comparison of the cracking of slabs 3, 5, and 6 having 0.533 percent deformed bar reinforcement at mid-depth, 2 in. below mid-depth, and 1 in below mid-depth, respectively, shows the influence of position of steel on the crack pattern. Slab 5 differed markedly from slab 3 in that the simulated temperature drops and the vertical loads both caused many cracks to start in the lower portion of slab 5, but caused only few to break completely through. In fact, up to 132-deg equivalent temperature drop, no complete cracks in addition to the three preformed cracks at 5-ft spacing had formed. At 132-deg equivalent temperature drop, one complete crack developed at each end of the slab, each at 4 ft from the nearest preformed crack. These cracks each formed completely as a result of the negative bending moment produced during the application of vertical load at the adjacent preformed crack, superimposed upon the simulated temperature change. Slab 6 with its 0.533 percent reinforcement at 1 in. below mid-depth contained six complete cracks after a temperature drop of 125 deg. These six cracks were spaced nearly uniformly at 2½ ft. This slab also contained some of the partial cracks that characterized slab 5, but not as many as appeared in slab 5.

The eccentricity of the longitudinal forces which simulated the temperature drops in slabs 5 and 6 produced positive bending moments in slab segments between complete cracks which tended to cause compression in the top surface of the slab and tension in the bottom surface. This effect, when superimposed upon the influence of the direct tensile force of temperature drop and the effects of vertical loading, accounts for the partial cracks produced by the eccentrically placed steel reinforcement. The partial cracking was most pronounced in slab 5 which had its steel placed with greatest eccentricity.

The influence of percentage of welded wire fabric reinforcement placed at mid-depth is shown in Figures 7, 8, and 9 for slabs designated by the same numbers. In studying these figures, one must keep in mind that the reinforcement in each 28-ft specimen consisted of three separate lengths of fabric, lap spliced as shown in each figure. Slab 7 with 0.154 percent longitudinal steel in its fabric reinforcement developed no cracks in addition to the three preformed cracks, even though it was subjected to horizontal (longitudinal) tensile forces equivalent to temperature drops greater than any practical value. Slab 8 which had 0.300 percent longitudinal steel sustained one additional crack at a temperature drop of about 60 deg, approximately under a vertical load at position 5. No other cracks developed until the slab was in a condition of general yielding to longitudinal tension force. Slab 9 with 0.450 percent longitudinal steel in its welded wire fabric received an additional crack under a vertical load at point 2 accompanying a temperature drop of 16 deg and another at the same temperature drop under a vertical load at point 6. In addition, this slab cracked 5 ft from one of the outermost preformed cracks and 4 ft from the preformed crack nearest the other end. These two cracks occurred, however, at a horizontal load equivalent to more than 150-deg temperature drop — beyond any practical range of temperature change. Cracks would probably have occurred somewhere in the same vicinity at less temperature drop if the slab had been subjected to vertical loads throughout its length. Thus, slab 9 indicated that a crack spacing of about 2½ ft could be expected in a slab 8 in. thick, reinforced at mid-depth with fabric providing 0.45 percent longitudinal steel, and resting on a fairly flexible subgrade (k = 160 pci).

Slab 10 was also reinforced with 4 x 12 00000/0 fabric but had its reinforcement placed 1½ in. below mid-depth. The crack pattern which developed in this slab did not differ greatly from the pattern of slab 9; however, slightly greater temperature drops were required, in combination with the vertical loads, to cause the first cracks to appear. Crack 7 appeared at the end of the 2 ft lap over preformed crack 2 during a horizontal force of 40 kips which was equivalent to a temperature drop of 94 deg. Stress concentration in the concrete caused by the lap ending was probably the cause of this crack at only 1 ft away from the preformed crack.

Slab 11 with its 0.450 percent reinforcement placed 1½ in. above mid-depth developed a crack pattern having little significant difference from that of slab 10 in the central test portion; however, it showed more top surface cracks near its ends at practical temperature drops than did slab 10. The greater number of top surface cracks was obviously due to the eccentricity of the reinforcement. It could not be ascertained if these cracks extended clear through to the bottom of the slab.

CORRELATION OF LABORATORY RESULTS WITH FIELD TESTS

The crack patterns developed in the seven laboratory slabs with the reinforcement at mid-depth agree in general with the patterns reported for the field experiments in Indiana, Illinois, New Jersey, California, and Texas which contained somewhat comparable reinforcement (11). The minimum crack spacing reported for each of those field test pavements was in the same order of magnitude as the crack spacing finally obtained in these laboratory experiments; that is, 2 to 2.5 ft for slabs having 0.45 percent or more longitudinal reinforcement and up to 5 ft or more for less reinforcement. Although these laboratory experiments did not include the effects of the great numbers of repeated loads obtained in the field tests, the maximum combined effects of shrinkage, maximum wheel loads, and equivalent temperature drops were obtained in a relatively short time in the laboratory with slabs resting on a relatively flexible subgrade.

The laboratory slabs in which the reinforcement was placed below mid-depth have no counterparts among the continuously reinforced test pavements in existence. It is anticipated that the results reported here for these eccentrically reinforced test specimens will be comparatively verified in the laboratory where additional slabs having eccentric reinforcement are to be tested on a stiffer subgrade. Results of these laboratory experiments may suggest the need for additional field tests.

It is to be noted that the data presented show, in general, the effects of equivalent temperature drops as great as 100 deg with some crack formation described for temperature drops of 125 deg. The horizontal forces applied to the slabs in the laboratory were in most cases carried to magnitudes corresponding to considerably greater temperature drops. Those data which extended far beyond limits of practicality have been omitted in this presentation.

Depending upon the construction locale and the temperature at the time of pouring the concrete, data presented for temperature drops greater than 90 to 100 deg may be unreliable in predicting the behavior of continuously reinforced pavement. A temperature decrease to below freezing would freeze and stiffen the concrete and would thus temporarily provide a greater subgrade modulus.

DESIGN CRITERION

It is the common theme of all engineers involved with design or research on continuously reinforced pavements that the slab must be reinforced with continuous longitudinal steel in sufficient amount to maintain all transverse cracks in a tightly closed condition without adverse effect on the concrete. The laboratory experimental results obtained for slabs having reinforcement at mid-depth indicate that this condition is met when increasing longitudinal forces (temperature decrease), in combination with live (wheel) loads tend to cause additional cracks to form rather than to cause excessive opening of existing cracks. If this criterion were to be applied to choose slabs adequately reinforced at mid-depth from among those tested in the laboratory, 0.430 percent deformed bar reinforcement would be inadequate while 0.533 percent deformed bar reinforcement would be judged adequate. Likewise, 0.30 percent welded wire fabric longitudinal reinforcement would be inadequate while 0.45 percent welded wire longitudinal reinforcement would be judged adequate. Within the limitations of the properties of the materials used in these experiments one could conclude from the research data that 0.450 percent longitudinal reinforcement, either in the form of deformed bars or welded wire fabric, would satisfy this one criterion for an 8-in. continuously reinforced concrete pavement slab resting on a subgrade having a modulus of 160 pci.

While the design criterion discussed above appears to be fundamental, one should expect that other criteria will come out of the study of stresses and deflections in continuously reinforced concrete pavements. Such other criteria may be more severe than this one, thus making this one automatically satisfied. Too little data now exist to determine what alterations to this design criterion should be made in order to judge the adequacy of eccentrically placed reinforcement. This question will receive attention in the laboratory experiments which are continuing.

CONCLUSIONS

As a result of this study of the formation of cracks in continuously reinforced pavements, and within the limitations imposed by the range of variables studied, the following conclusions are draw:

1. The effects of concrete shrinkage, temperature changes, and wheel loads on the crack formation in continuously reinforced pavements can be adequately simulated in the laboratory without producing actual temperature changes in an infinitely long pavement slab.
2. The formation of a complete crack pattern in a continuously reinforced pavement is the result of a superposition of the effects of concrete shrinkage, temperature changes and live (wheel) loads. Perhaps the importance of live loads has been somewhat underestimated in past discussions of this problem.
3. Crack spacing in a continuously reinforced pavement with some age and use varies inversely with the percentage of longitudinal steel reinforcement if the steel is placed at mid-depth. A minimum average crack spacing of about 2 ft may be expected in a slab having 0.768 percent longitudinal steel in the form of deformed bars.
4. The position of 0.533 percent deformed bar reinforcement in an 8-in. slab has a marked difference on the crack pattern. With the steel 2 in. below mid-depth, temperature decreases and vertical loads cause many cracks to start in the lower part of the slab but only a few to break completely through. Five feet seems to be about the minimum spacing between complete cracks in a slab thus reinforced. With the same amount of steel 1 in. below mid-depth, the spacing of complete cracks decreases to about 2½ to 3 ft, and less partial cracks appear.
5. Welded wire fabric reinforcement of 0.450 percent longitudinal steel placed 1½ in. below mid-depth in the slab produces a crack pattern only slightly different from that produced by the same steel placed at mid-depth. However, slightly greater temperature drops are required to cause the first cracks to appear at the surface of the slab having the lowered reinforcement.
6. It is generally agreed that a continuously reinforced pavement slab must be reinforced with longitudinal steel in sufficient amount to maintain all transverse cracks in a tightly closed condition without adverse effect on the concrete. For mid-depth reinforcement this condition is met when increasing longitudinal forces caused by temperature drops, in combination with live loads, tend to cause additional cracks to form rather than to cause excessive opening of existing cracks. This standard is suggested as one criterion (among others) for the design of continuously reinforced pavements.
7. The single design criterion stated in conclusion number 6 is satisfied in these laboratory tests for an 8-in. pavement slab resting on a subgrade having a modulus of

160 pci, by 0.45 percent longitudinal reinforcement either in the form of deformed bars or welded wire fabric reinforcement placed at mid-depth in the slab.

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REFERENCES

1. Lindsay, J.D., " A Ten-Year Report on the Illinois Continuously Reinforced Pavement." HRB Bull. 214 (1959).
2. Van Breemen, W., "Ten-Year Report on Continuously Reinforced Concrete Pavements in New Jersey." HRB Bull. 214 (1959).
3. Tremper, B., "Continuously Reinforced Concrete Pavement in California After Eight Years Service." HRB Bull. 214 (1959).
4. Taylor, I.J., and Eney, W.J., "First Year Performance Report on Continuously Reinforced Concrete Pavements in Pennsylvania." HRB Bull. 214 (1959).
5. Zuk, William, "Analysis of Special Problems in Continuously Reinforced Concrete Pavements." HRB Bull. 214 (1959).
6. Cashell, H.D., and Teske, W.E., "Continuous Reinforcement in Concrete Pavements." HRB Proc., Vol. 34 (1955).
7. Gutzwiller, M.J., and Waling, J.L., "Stresses and Deflections in Pavements Continuously Reinforced with Deformed Bars." American Concrete Institute (Feb. 1959).
8. Gutzwiller, M.J., and Waling, J.L., "Stresses and Deflections in Pavements Reinforced with Welded Wire Fabric." HRB Bull. 238 (1960).
9. Gutzwiller, M.J., and Waling, J.L., "Laboratory Research on Pavements Continuously Reinforced with Deformed Bars." American Concrete Institute (Feb. 1959).
10. Gutzwiller, M.J., and Waling, J.L., "Laboratory Research on Pavements Continuously Reinforced with Welded Wire Fabric." HRB Bull. 238 (1960).
11. Jacobs, W.H., "Survey and Correlation Report on Continuously Reinforced Pavements Without Joints." Rail Steel Bar Association (1953).

Bond and Transflexural Anchorage Behavior of Welded Wire Fabric

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In this paper, mathematical theories are developed to predict the interactive bond-anchorage behavior of welded wire fabric when subject to a pull-out action, simulating the behavior of welded wire fabric in continuously reinforced concrete pavements. The theories are predicated on the concepts that the bond between the smooth longitudinal wires and the concrete is a function of minute movement between the steel and the concrete and that the anchorage of the transverse wires is achieved through the restrained flexural action of the transverse wires.

A summary of various general experimental techniques that other investigators have developed to measure bond is also presented as a background to the author's method, not hitherto used on small bars, in which electrical resistance strain gages are embedded in slots inside the bar. Based on this method, a description of the experiments on bond-anchorage behavior of welded wire fabric in concrete is presented. The tests are of a pull-out nature encompassing bond-anchorage behavior at first loading and after repeated cyclic loading.

A comparison of the theory and experiments shows general agreement.

THE PRESENT investigation of bond-anchorage behavior of welded wire fabric in concrete had its origin in the study of continuously reinforced concrete pavements. In one of the author's previous papers (1) certain bond assumptions had to be made in the theory to predict the amount of crack opening and required wire steel. The only information available on this subject was some experimental work performed by Anderson (2). Although Anderson's findings are of value to the designer, the findings are lacking in certain needed areas for theoretical and research use. Through the cooperation of T. E. Shelburne of the Virginia Council of Highway Investigation and Research and Henry Aaron of the Wire Reinforcement Institute, facilities were made available to determine the basic information needed.

The first section of this work is theoretical; where equations are derived explaining the fundamental nature of the bond and anchorage action of a wire mesh in the region of a crack in a continuously reinforced pavement. The action at a crack is identical with pull-out action of a mesh embedded in a concrete slab, since at a crack the entire force is transmitted through the longitudinal wires. Exactly how this wire force is transmitted to the surrounding concrete is the essence of the problem at hand.

THEORY

For convenience of analysis only a single longitudinal wire is studied, as it is assumed that, at a crack, all the longitudinal wires act in the same manner. It is assumed that the steel and concrete remain elastic, that there is no creep effect, that the welds between the wires remain unbroken, that the stresses and strains in the mass concrete are very small, and that under certain conditions the adhesive bond between the steel and the concrete is negligibly small. In connection with the last assumption, elaboration is necessary.

First of all, bond is considered that restraining force acting like frictional shear only in the region of the surface of the longitudinal wire, as contradistinct to transflexural anchorage which is the restraint offered by the transverse wires. Although the effects are interactive, their basic behavior is different. Thus, for clarity of understanding as well as for other practical reasons, it will be initially assumed that no bond exists on the longitudinal wires. Once this transflexural anchorage behavior is explained, the bond behavior will then be separately analyzed. However, there are certain engineering bases for neglecting bond because of its unreliable and uncertain nature. Such things as the presence of grease or dirt on the wires, repeated loads causing agitation and erosion of the concrete around the wires, and reduction of wire diameter caused by Poisson's effect all tend to reduce bond. Only that form of bond created by bearing action (as in deformed bars) is reliable in all situations.

Figure 1 is a general plan view of the bar fabric in place in a cracked continuously reinforced concrete pavement. For information on the values of P_1 see (1).

Figure 2 is a skeleton view of one strand of wire fabric embedded in concrete with a crack formed at the right end. The force P in the wire is distributed along the length of the wire with decreasing value away from the crack. It is assumed that at the end of five transverse wire anchorages, this force P_6 is so small that it may be neglected.

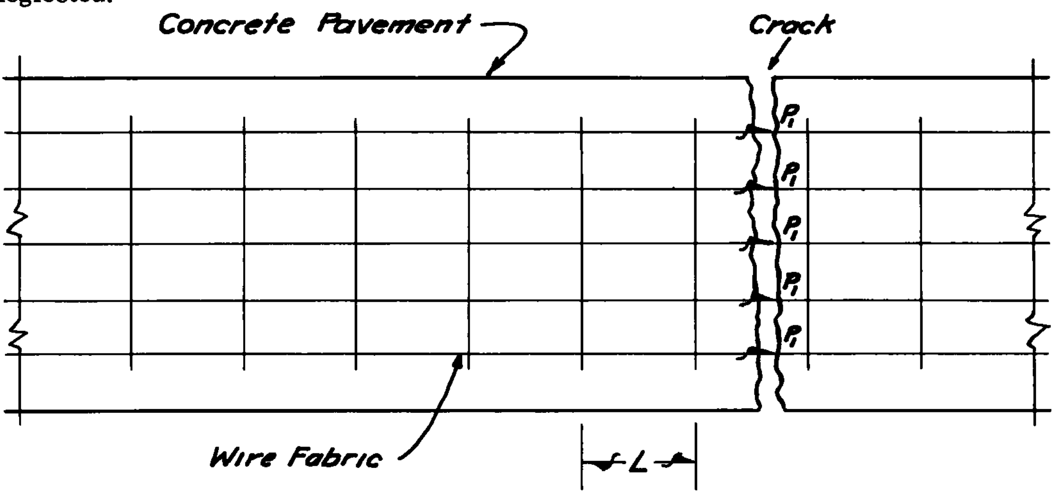

Figure 1. Plan view of fabric in pavement with crack.

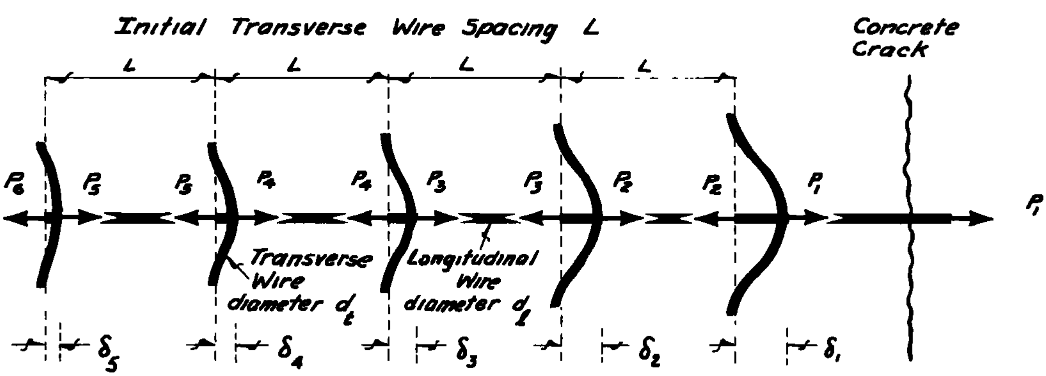

Figure 2. Longitudinal wire forces and transverse wire deflections.

The transverse wires act as flexural beams, restrained from bending by the compression of the surrounding concrete. (This condition is similar to that assumed by Friberg (14).) This situation is entirely analogous to the well known theory of beams on elastic foundations where the elastic support modulus is E_c (modulus of elasticity in compression of concrete) and the load is the unbalanced concentrated load across each joint (3). Interaction effect from adjacent longitudinal wires is negligibly small.

Thus the deflection δ, is

$$\delta_1 = \frac{(P_1 - P_2)\,\beta}{2\,E_c}$$

where

$$\beta = \sqrt[4]{\frac{E_c}{4\,E_s\,I_t}}$$

and E_s is the modulus of elasticity of the steel and I_t is the moment of inertia of the transverse wire = $\dfrac{d_t^{\,4}}{64}$

Likewise:

$$\delta_2 = \frac{(P_2 - P_3)\,\beta}{2\,E_c}$$

$$\delta_3 = \frac{(P_3 - P_4)\,\beta}{2\,E_c}$$

$$\delta_4 = \frac{(P_4 - P_5)\,\beta}{2\,E_c}$$

$$\delta_5 = \frac{(P_5)\,\beta}{2\,E_c}$$

However, the longitudinal wire stretches as it is subjected to load. From elementary theory

$$\delta_1 - \delta_2 = \frac{P_2\,L}{A_l\,E_s}$$

where A_l is the area of the longitudinal steel = $\dfrac{\pi\,d_l^{\,2}}{4}$

Likewise:

$$\delta_2 - \delta_3 = \frac{P_3\,L}{A_l\,E_s}$$

$$\delta_3 - \delta_4 = \frac{P_4\,L}{A_l\,E_s}$$

$$\delta_4 - \delta_5 = \frac{P_5\,L}{A_l\,E_s}$$

Assuming P_1 is known, there are nine unknown quantities P_2, P_3, P_4, P_5, δ_1, δ_2, δ_3, δ_4, δ_5, and nine independent equations in P and δ as just presented. By standard elimination methods of simultaneous solutions, the following results for P may be obtained.

$$P_2 = \frac{(\gamma\phi^2 - 2\gamma^3\phi)}{(\gamma^4 + \phi^4 - 3\gamma^2\phi^2)} P_1$$

$$P_3 = \frac{(\gamma\phi^2 - \gamma^3)}{(\phi^3 - 2\phi\gamma^2)} P_2$$

$$P_4 = \frac{(\gamma\phi)}{(\phi^2 - \gamma^2)} P_3$$

$$P_5 = \frac{(\gamma)}{(\phi)} P_4$$

where

$$\gamma = \frac{3}{4E_c d_t} \sqrt[4]{\frac{E_c}{E_s}}$$

$$\phi = 2\gamma + \frac{\gamma L}{A_1 E_s}$$

To simplify these results it is observed that the fraction of longitudinal wire force R transmitted across each transverse wire is constant, and will be called the transfer ratio.

$$R = \frac{P_{m+1}}{P_m} \simeq \frac{\gamma\phi^3 - 2\gamma^3\phi}{\gamma^4 + \phi^4 - 3\gamma^2\phi^2} \simeq \frac{\gamma}{\phi} \qquad (1)$$

The simple ratio $\frac{\gamma}{\phi}$ is accurate to about 3 percent and the other longer expression in Eq. 1 is accurate to less than 1 percent.

Thus it is seen that the force in the longitudinal wire progressively diminishes away from the crack.

Example

$L = 10$ in.
$E_c = 3 \times 10^6$ psi
$E_s = 30 \times 10^6$ psi
Transverse wire size No. 1 ($d_t = 0.283$ in.)
Longitudinal wire size No. 5/0 ($d_1 = 0.431$ in.)
$P_1 = 8150$ lb (limit value based on 56,000-psi stress)

Based on the preceding calculations, Figure 3 shows the distribution of force.

The force in the longitudinal wire diminishes very rapidly. Should bond exist, the longitudinal wire force would diminish even more rapidly, and the force in the longitudinal wire beyond the third transverse wire would essentially vanish as both Anderson's (2) and the author's tests show.

Bond

Since some force in the longitudinal wires is known to exist, strains must also exist. The steel must therefore move with respect to the concrete. The order of magnitude

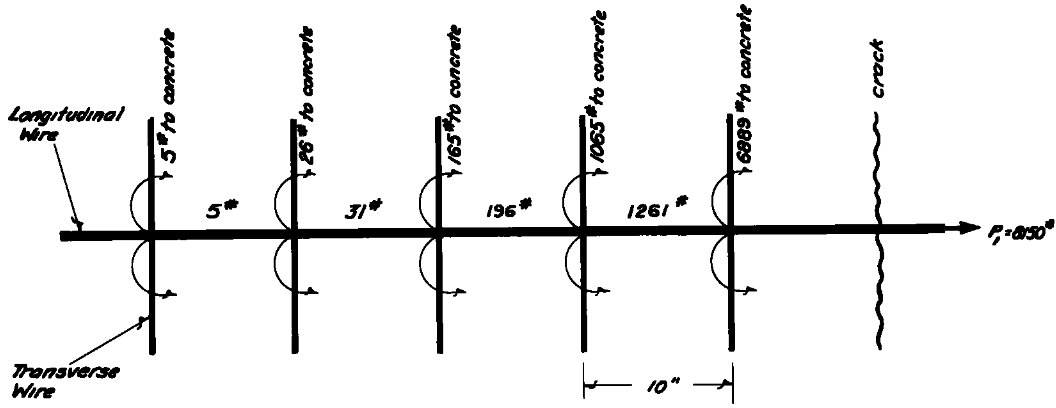

Figure 3. Distribution of wire forces.

of this movement is very small and cannot easily be measured. This movement may be due to actual slip or to a thin boundary layer of concrete adjacent to the steel being shear strained along with the steel. The latter condition implies true adhesive bond. However, either condition is covered by the proposed theory. Osterman (12) also found that this working hypothesis was very satisfactory for his studies of bond as used in a different application.

The first panel adjacent to the crack in Figure 2, is now assumed to have bond forces acting on the longitudinal wire. If the displacement of any element in the X direction as shown in Figure 4 is defined as u, then the unit surface bond stress is

$$\tau = K(u + \delta)$$

where K is an empirical constant dependent on such factors as concrete strength, roughness of wire, and moisture content of concrete.

In terms of the displacement the force in the wire at any position is

$$\frac{du}{dx} A_l E_s$$

which from equilibrium must equal

$$P_1' + K \pi d_l \int_0^x (u + \delta)\, dx$$

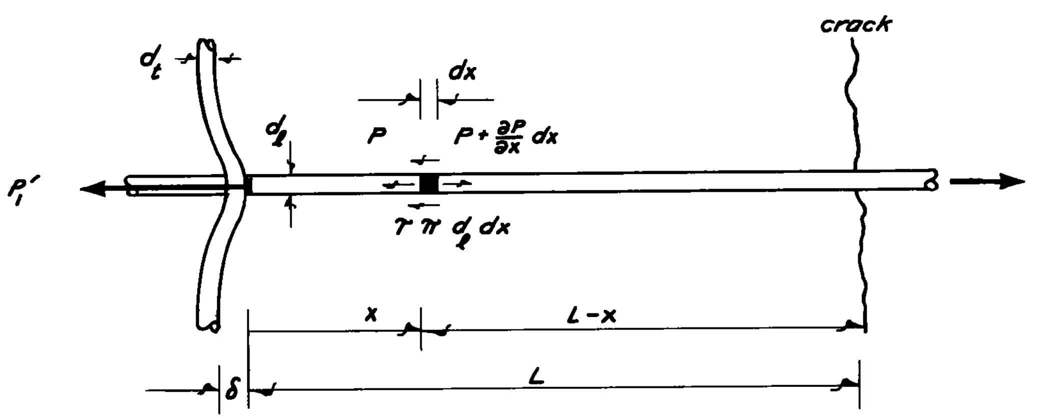

Figure 4. Bond forces on wire element.

Thus, the equation is formed as follows

$$\frac{du}{dx} A_1 E_s = P_1' + K\pi d_1 \int_0^x (u + \delta)\, dx \tag{2}$$

The solution to Eq. 2 which satisfies the boundary conditions is in the form of

$$u = Qe^{\alpha x} - Q \tag{3}$$

where α and Q are undetermined parameters. By substituting Eq. 3 into Eq. 2 and equating like terms, α and Q may be obtained as follows:

$$\alpha = \sqrt{\frac{K\pi d_1}{A_1 E_s}}$$

$$Q = \delta$$

Thus the solution to Eq. 2 is

$$u = \delta\, e^{x\sqrt{\frac{K\pi d_1}{A_1 E_s}}} - \delta \tag{4}$$

With the displacement u known, the force in the wire at any position may be obtained from the equilibrium equation below:

$$P(x) = P_1' + K\pi d_1 \int_0^x (u + \delta)\, dx$$

$$P(x) = P_1' + \frac{K\pi d_1 \delta}{\alpha} + \frac{K\pi d_1 \delta}{\alpha} e^{\alpha x}$$

Since

$$P_1 - P_1' = \int_0^l \tau \pi d_1\, dx$$

the expression for P (x) may also be expressed in terms of P_1 as follows:

$$P(x) = P_1 - \frac{K\pi d_1 \delta}{\alpha} (e^{\alpha l} - e^{\alpha x}) \tag{5}$$

A typical plot of this equation is shown in Figure 5.

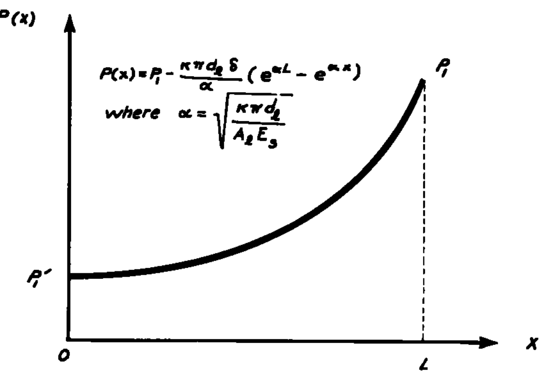

Figure 5. Wire force variation.

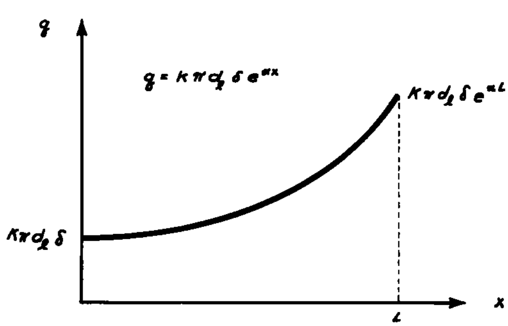

Figure 6. Bond force variation.

The bond force variation q may also be plotted from the equation

$$q = K\pi d_1 (u + \delta) = K\pi d_1 \delta e^{\alpha x} \tag{6}$$

as shown in Figure 6.

Both the wire force and bond force have the same general exponential shape. Experimental evidence to be cited later will show that this theory does agree with experiments.

EXPERIMENTAL METHODS FOR MEASURING BOND

A survey of existing literature on the subject of experimental methods for measuring the actual bond stress and its variation on plain and deformed bars reveals nine different techniques used by previous investigators. These nine methods will be discussed briefly here as background to the author's method. This survey and discussion of techniques may also be of benefit to other investigators faced with instrumenting for bond measurements.

Two simple methods (4) use extensometers to measure the strains in the steel at intervals. The steel bar may be made accessible for measurement in either of two ways. One method is to leave access holes in the concrete to the steel. The disadvantage here is that these holes may create stress irregularities as well as reduce the bond area. Also the gage length cannot practically be less than about 2 in., thereby resulting in loss of accuracy. Furthermore, if the concrete layer is thick, various extensions or levers must be attached to the extensometers to extend to the bars. The second alternative is to weld stubs on at intervals along the steel bar, protruding through access holes to the surface of the concrete. Here again the access holes must be carefully preformed such that the stubs will at all times be free of contact. Again accuracy is lost in that the stubs physically cannot be too close together. Loss of bond area and stress concentrations also play a detrimental role.

In another direction, much has been done with electrical resistance strain gages such as the SR-4 type. There are four techniques in this category. The first attempts were made by merely bonding the gages to the surface of the bar. Waterproofing and shielding of leads are important in this technique. Anderson (2) and McHenry and Walker (5) used this method. Some disadvantages of this method are: much bond area is lost (especially by the waterproofing application), wedging action of the protruding gage and the concrete takes place, and waterproofing is difficult.

To overcome some of these objections, Mains (6) split a reinforcing bar for its entire length and milled a small groove along the axis of the bar on the inner face. After attaching and waterproofing SR-4 gages on this inner slot, the two halves of the bar were spot welded back together. The leads of the gages all extended out the ends of the bar and no bond area was lost due to instrumentation. However, even this method is not without some drawbacks. Of course the major objection is in the trouble and expense of splitting and milling such bars; but in addition, such a hollow bar cannot be loaded to the same failure force as a normal bar, which means that information at normal failure loads cannot be obtained.

Bernander (7), using large diameter bars, inserted electrical resistance strain gages in preformed narrow slots along the bar. The slots extended only part way through the bars, and the gages were cemented to one side of these slots along the diameter line. The slots were filled with resin for moisture proofing. These slots were easy to form and waterproof and little bond area was lost. However, because the slots did not extend completely through the bar some bending or eccentric stresses were caused in the bar, even where direct tension was exerted at the ends.

Janney (8), using SR-4 gages, obtained bond information on very small diameter wire by a clever indirect method. The wires to be studied were cast in concrete prisms, with the ends of the wires extending beyond the prisms. SR-4 gages were then cemented to the exterior surface of the concrete along the axis of the prism in the direction of the wire. When the ends of the wire were pulled, the variation of the concrete strains could be measured and related to the wire strains. Certain weaknesses are inherent in this method, such as the shear lag between the wire strains and the concrete surface strains, possible concrete cracking and the difficulty of relating non-linear concrete strains to steel stresses. However, for small diameter wires, no gaging could be attached to the wire itself, and so this method was used.

X-Ray techniques have been used in England to obtain bond stresses. Evans and Robinson (9) embed thin strips of platinum or lead in slots in the steel and X-Ray photograph the concrete and encased steel bars as load is applied to the steel. Images cast by the markers are thus recorded on film without disturbing the steel or concrete. By use of microscopes these filmed images may be studied and information on strains, stresses and bond may thus be obtained.

At the State Institute for Technical Research in Helsinki, a unique method was used for bond study (10). The steel stresses in the concrete were obtained without physically disturbing the steel or concrete in any way by a magneto-strictive method. This method is based on the principle that an alternating current passing through a steel bar creates a magnetic field, perpendicular to the longitudinal axis of the bar, which in conjunction with the resistance determines the electrical impedance of the steel. If the bar is subjected to tension while a constant current is applied, the impedance will be altered by magneto-striction. If the relation between the change in impedance and the stress in the steel is known (by pre-calibration), the average stress over a small given distance can be determined by measuring the voltage difference over this distance. Thus, in this way the bond variation may be obtained. The technique is not yet fully perfected as shrinkage, moisture and cracks influence the results.

Finally, to emphasize the variety of techniques used for bond studies, a ninth method of photoelasticity is mentioned. Beyer and Salakian (11) used a metal wire molded in photoelastic bakelite to determine bond stresses between the wire and the model, simulating reinforcing steel in concrete.

The method that served best for this particular study of bond of wire mesh was a variation of Bernander's method, in which the slot was extended all the way through the bar to eliminate the undesirable effects of unsymmetrical straining. Figure 7 shows how the slots were made and how the small SR-4 gages were attached. Minimum bond area was lost and successful waterproofing was easily accomplished by filling the slots with melted Petrosene wax. Extended water immersion tests showed that this method was quite satisfactory for the purpose intended. However, due to the removal of metal, the slotted wires had to be precalibrated to determine the relation

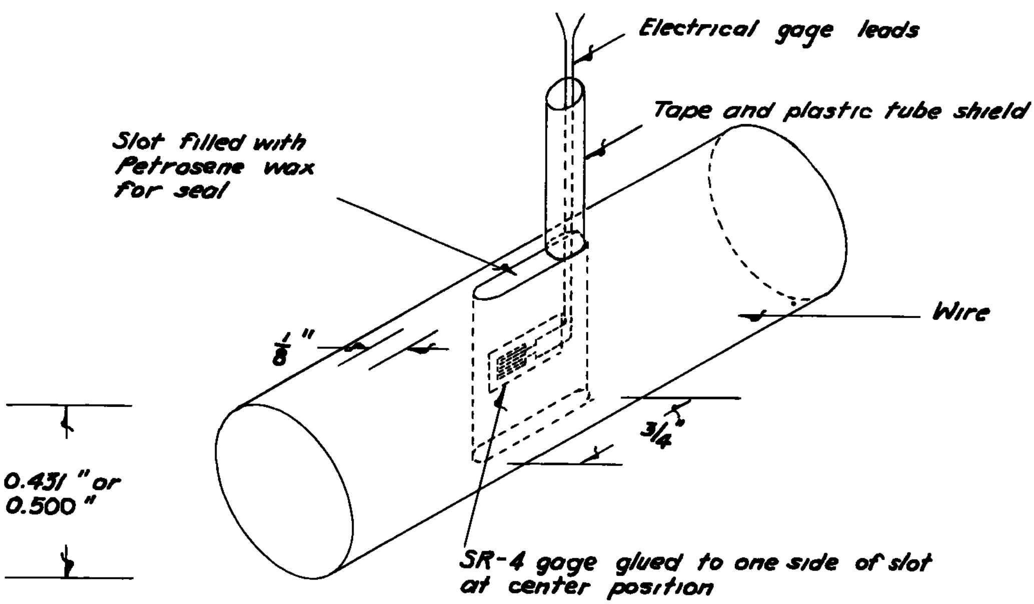

Figure 7. Attachment of gage to wire.

of the stress at the gage to the stress in the normal unslotted wire. This relation proved to be linear up to the yield stress of 62, 000 psi.

TESTS

To determine the actual bond stress and its variation in welded wire mesh embedded in concrete, several tests were performed. Two mesh configurations were studied. These are shown in Figure 8.

Mesh A was embedded at mid-depth in a panel of concrete 16 in. by 4 in. by 42 in. and mesh B was embedded in a panel of concrete 16 in. by 4 in. by 62 in. Figure 9 shows these wire meshes in the forms prior to pouring. Note that only the center longitudinal wires are extended on which the pull out force is applied. Figure 8 also shows the locations of the SR-4 gages as positioned along the center longitudinal wires. At the time of pouring, test cylinders were made in order to obtain the concrete properties. The eye bolts shown are simply for the purpose of handling. After curing and hardening of the concrete for four weeks, the test panels were placed in a Universal Testing Machine and loaded (Fig. 10).

The center wire freely passed through the lower head and was gripped by the upper tension head. A bearing plate was provided to prevent crushing of the concrete. By this arrangement, a pull-out action was achieved.

Strain readings at each of the gage points were taken only at the first loading cycle

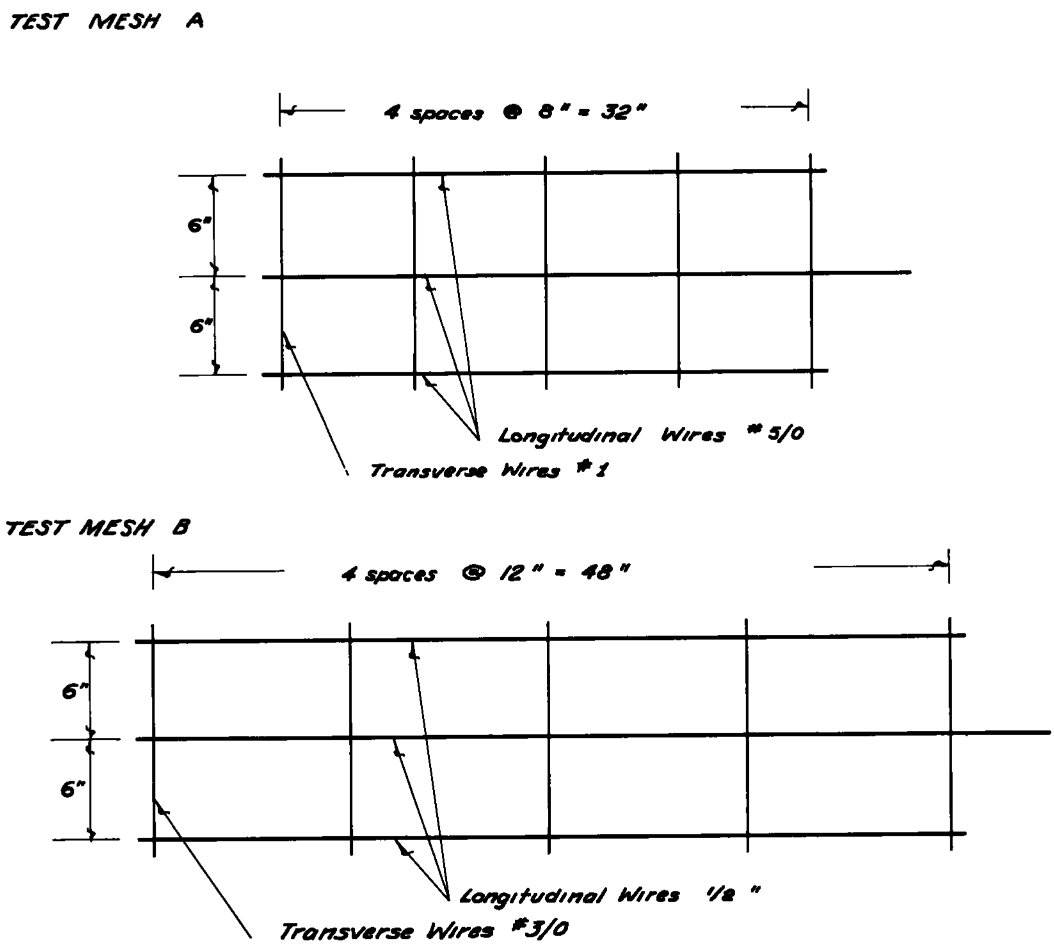

Figure 8. Test meshes.

and at the 60th loading cycle. Cyclic readings did not go beyond 60 cycles as the Universal Testing Machine used is awkward and time consuming to cycle. As the results of cyclic loading are of most importance, these results are plotted in Figures 11 and 12 for the A and B panels respectively based on an average center wire force of 1,000 lb. It is perhaps well to note that total loads on the wires were imposed to the extent of 80 percent of the yield stress load at the slot. At the range of load imposed, there

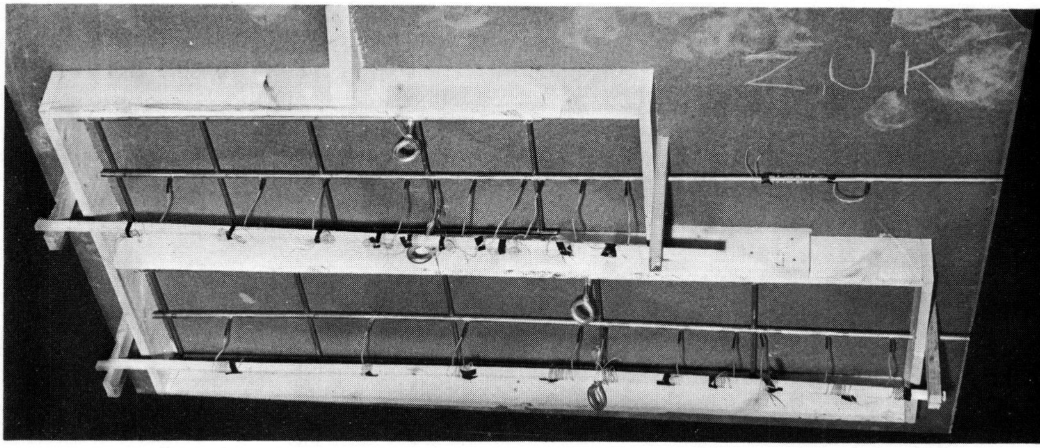

Figure 9. Test panels before pouring concrete.

did not seem to be any particular deviation from the same variation of wire force.

COMPARISON OF THEORY AND TESTS

The shape of the wire force curves between transverse wires compares well with the theory. That is, theoretical curves in Figures 5 and 6 correspond to experimental curves in Figures 11 and 12 between transverse wires. In addition, experimental curves found by Mains (6) in his Figure 7a of the pull-out force variation in a hooked plain bar also show good agreement with the theoretical curves. The anchorage action of a single hook is much the same as the transflexural anchorage of transverse wires as some movement also occurs at the anchorage such that the bar force and bond force do not vanish in this region.

Based on Eq. 5 and the data in the curves of Figures 11 and 12, the empirical bond value of K was determined as 49.8×10^4 lb per sq in. wire surface per in. slip. This value of K looks astronomically large, but the so-called slip is in the order of magnitude of less than one thousandth of an inch. Should large slips occur, entirely different bond-fric-

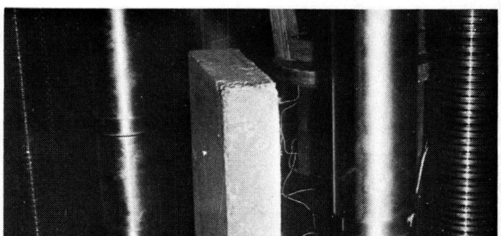

Figure 10. Panel in testing machine.

tion action would take place, invalidating the theory. Bond behavior becomes very irregular at large slip values as pointed out by Mains (6) and Mylrea (13).

Figures 11 and 12 show that no wire and bond forces exist beyond the second transverse wire. If such forces did exist, the strain gage readings were so small that they did not register. This information seems to verify Anderson's (2) observations on his own static tests that only the first two transverse wires are effective, even up to rupture of the steel. However, the full effects of cycling of stress many thousand times was not demonstrated in any of the tests, and the results should perhaps be viewed with this in mind. Such things as fatigue and agitation of the concrete in the region of the steel may well play an influential role in bond behavior under actual service conditions. It was noticed in these tests that the bond value of K was reduced from the first loading cycle to the 60th cycle, indicating a general trend in that direction. Future work using a special fatigue apparatus is thought to be necessary in this connection.

The computed values of the transfer ratio R of the wire force across the first transverse wire show agreement with theory regarding the effects of transverse wire size and spacing. Smaller values of $\frac{L}{A_1}$ mean less longitudinal wire stretching, and thin transverse wires mean less transflexural anchorage; resulting in more force being transmitted across the anchor points (larger values of R). Panel A has both a smaller $\frac{L}{A_1}$ ratio and smaller transverse wire size than panel B, resulting in a higher value of R (Figs. 11 and 12).

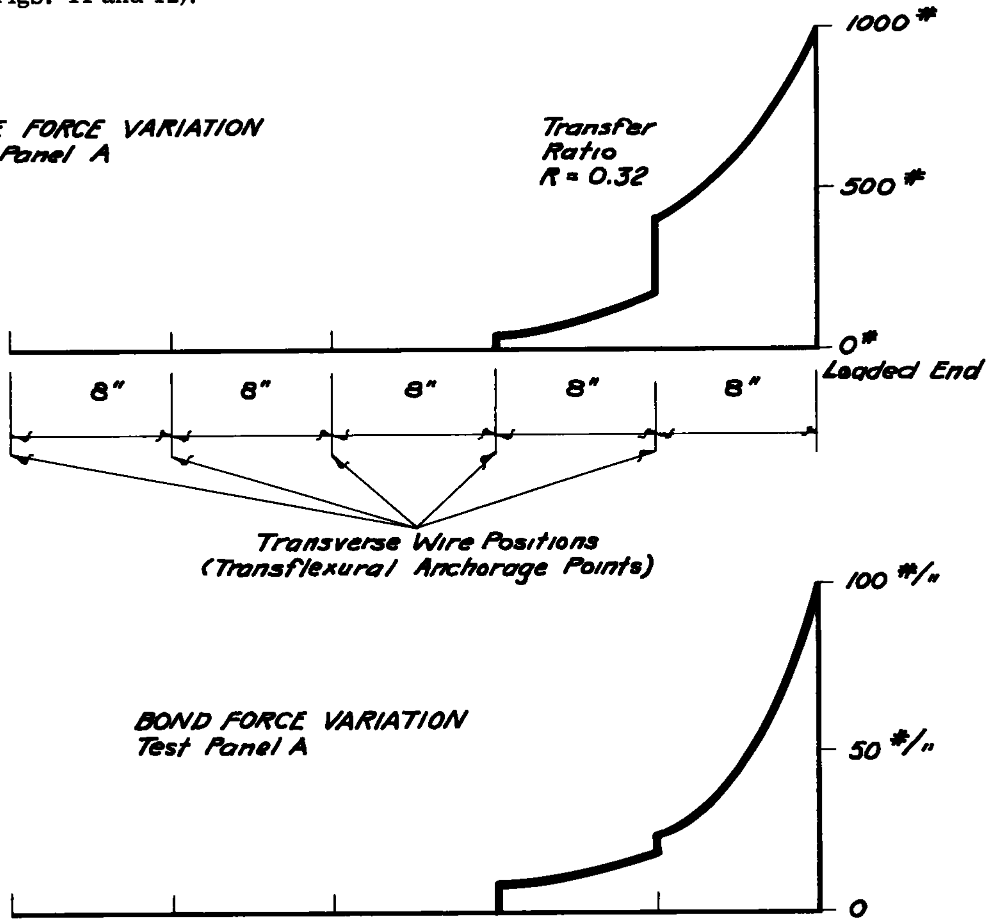

Figure 11. Wire and bond force variation— "A" panel after 60 cycles.

The theoretical value for the transfer ratio R for panels where both bond and transflexural anchorage exist may be obtained from the theory given at the beginning of this report. To do this, Eq. 5 is written for the first two spaces from the end as

$$P_1' = P_1 - \frac{K \pi d_1 \delta_1}{a} e^{aL}$$

and

$$P_2' = P_2 - \frac{K \pi d_1 \delta_2}{a} e^{aL}$$

Two transflexural beam equations are also needed, written as

$$\delta_1 = \frac{(P_1 - P_2)\beta}{2 E_c}$$

and

$$\delta_2 = \frac{P_2' \beta}{2 E_c}$$

In addition, a final equation of distortion, Eq. 4 for the second space is written as

$$\delta_1 = \delta_2 e^{aL}$$

By solving these five equations simultaneously, the transfer ratio R for the combined

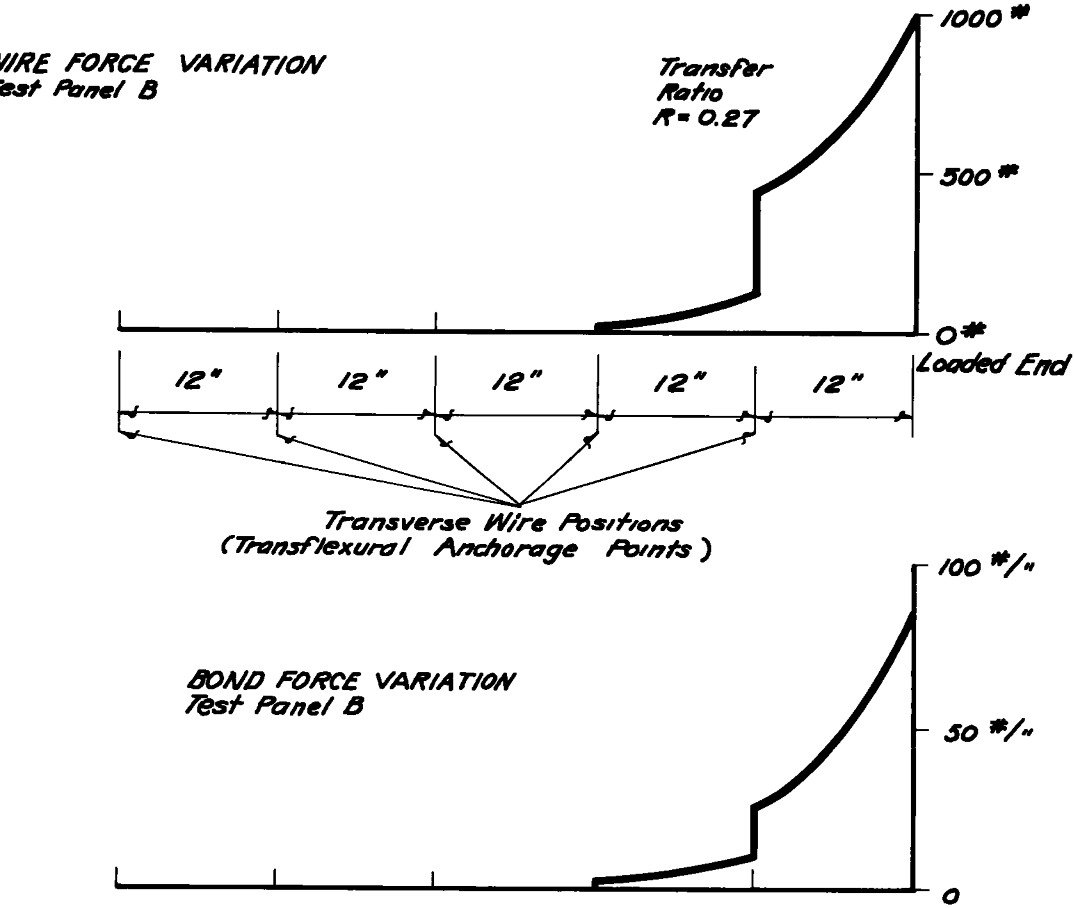

Figure 12. Wire and bond force variation— "B" panel after 60 cycles.

case of bond and anchorage is obtained as

$$R = \frac{N}{N + 2\alpha E_c e \alpha L}$$

in which

$$N = 2\alpha E_c + K\pi d_1 \beta e^{\alpha L}$$

Using Eq. 7, R is 0.48 for panel A and 0.46 for panel B. The difference is in the correct direction but higher than experimental values indicate. The explanation for this probably lies in the adhesion of the transverse wires, which effect is not considered in the theory. Adhesion of the transverse wires would tend to reduce the transfer ratio.

CONCLUSIONS

The technique of using slotted holes employed in this set of tests seems to incorporate many advantages of simplicity, ease of fabricating, east of waterproofing, convenience when tested with standard equipment and good accuracy. Several disadvantages should also be noted. The slots must be milled exactly and the gages set exactly, otherwise the calibration of the stress concentration factor is difficult and unreliable. Special care must also be taken in bonding the gage to the wire under pressure. This is a little awkward in so narrow a slot (1/8 in. wide). An occasional gage in this set of tests did act erratically, due to poor adhesion of the gage. A final objection is that the force exerted on the wire had to be limited to about 60 percent of the rupture force on the normal bar to prevent yielding at the slot. This prevented the study of bond at normal rupture loads.

The theory and experiment showed good general agreement. This gives strong evidence to support the contention that bond of plain wires or welded wire fabric is a direct function of shear "slip" (at small values of slip) and that anchorage is primarily due to the restrained bending of the transverse wires. The theory and tests show that two distinct actions take place between the steel and the concrete to disperse the load in the longitudinal wires. The shearing boundary layer action between the longitudinal steel and the concrete is the one action called bond, and the restrained bending action of the transverse wire is the other action called transflexural anchorage. The theory developed shows the distinction and the relation between these two actions.

Further research is still necessary to study bond under repeated loads of many thousand cycles to determine the extent that bond action breaks down.

REFERENCES

1. Zuk, William, "Analysis of Special Problems in Continuously Reinforced Concrete Pavements." HRB Bull. 214 (1959).
2. Anderson, A.R., "Bond Properties of Welded Wire Fabrics." ACI Proceedings, Vol. 48, pp. 681-692 (1952).
3. Timoshenko, S., "Strength of Materials." Part II, pp. 1-6, Van Nostrand Company (1956).
4. Plowman, J.M., "The Measurement of Bond Strength." RILEM Symposium on Bond and Crack Formation in Reinforced Concrete, Vol. 1, Stockholm, pp. 154-156 (1957).
5. McHenry, D., and Walker, W.T., "Laboratory Measurements of Stress Distribution in Reinforcing Steel." ACI Proceedings, Vol. 44, pp. 1041-1046 (1948).
6. Mains, R.M., "Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars." ACI Proceedings, Vol. 48, pp. 225-252 (1952).
7. Bernander, K.G., "An Investigation of Bond by Means of Strain Measurements in High-Tensile Bars Embedded in Long Cylindrical Pull-Out Specimens." RILEM Symposium on Bond and Crack Formation in Reinforced Concrete, Vol. 1, Stockholm, p. 205 (1957).

8. Janney, J.R., "Nature of Bond in Pre-Tensioned Prestressed Concrete." ACI Proceedings, Vol. 50, p. 717 (1954).
9. Evans, R.H., and Robinson, G.W., "Bond Stresses in Prestressed Concrete from X-Ray Photographs." Proceedings, Instit. of Civil Engineering, Part 1, Vol. 4, pp. 212-235.
10. Parland, H., "Inelasticity of Bond Between Steel and Concrete; and Distribution of Stress in Steel in Cracked and Uncracked Structural Members." RILEM Symposium on Bond and Crack Formation in Reinforced Concrete, Vol. 2, Stockholm, pp. 312-313 (1957).
11. Beyer, A.H., and Salakian, A.G., "Stresses in Reinforced Concrete Due to Volume Changes." ASCE Transactions, Vol. 98, pp. 1064-1070 (1933).
12. Osterman, J., "Utdragsprovt." Betong, I. Stockholm (1951).
13. Mylrea, T.D., "Bond and Anchorage." ACI Proceedings, Vol. 44, p. 521 (1948).
14. ACI Committee 325, "Design Considerations for Concrete Pavement Reinforcement for Crack Control." ACI Journal, p. 361 (October 1956).

Discussion

J. L. WALING, School of Civil Engineering, Purdue University — Professor Zuk has undertaken an analysis of the distribution of the forces of anchorage of welded wire fabric in concrete. He is to be commended for attempting a solution to this difficult problem, complicated as it is by the uncertainty of the validity of the several assumptions which must be made in the theoretical analysis. The writer's questions and discussion will pertain primarily to these assumptions and will be considered not in the order of importance but in the order in which the material is presented in the paper by Professor Zuk.

The author states that "The action at a crack is identical with pull-out action of a mesh embedded in a concrete slab, since at a crack the entire force is transmitted through the longitudinal wires." This should be questioned since forces large enough to initiate a crack could also break the bond between the longitudinal wires of the fabric and concrete. Thus, the bond resistance available in the two cases may not be identical.

In stating the analogy of the action of the transverse wire to that of a beam on an elastic foundation, the author states that "...the elastic support modulus is E_c...."
In the theory of beams on elastic foundations, the support modulus (usually denoted by K) is a measure of the support resistance in force per unit length of beam per unit deflection of the beam. It can be shown that

$$k = E_c \frac{w}{d}$$

in which

w = effective width of the beam support, and
d = effective depth of compressed supporting material.

It is not clear to the writer that the ratio w/d can be taken as unity. Nor is it clear that this ratio can be reliably approximated by theory alone.

The author stated the equation for unit surface bond stress as $\tau = K(u + \delta)$. However, if he means that $\tau = K$ times the movement of the longitudinal wire relative to the adjacent concrete, then the expression should be $\tau = Ku$, for the adjacent concrete is compressed by the transverse wire an amount equal to δ, at least in a region near $x = o$. Near $x = L$ the relationship of u to the bond shearing strain seems undetermined, since the author appears not to have considered concrete deformations near the crack.

If one assumes that the author is correct in the statement that $\tau = K(u + \delta)$ then the solution stated as Eq. 4 satisfies Eq. 2 only if $P_1' = K\pi d_1 \frac{\delta}{a}$. This seems to have escaped the attention of the author during the process of equating like terms after the substitution of Eq. 3 into Eq. 2.

If on the other hand one assumes that $\tau = Ku$, which would appear to be closer to reality over a greater length of the longitudinal wire, one can derive the following:

$$u = \frac{P_1' \alpha e^{\alpha x}}{K\pi d_1}$$

$$P(x) = P_1' e^{\alpha x}$$

and

$$q = P_1' \alpha e^{\alpha x}$$

where

$$\alpha = \sqrt{\frac{K\pi d_1}{A_1 E_s}} = \sqrt{\frac{4K}{d_1 E_s}}$$

Thus, it is seen that this assumption also yields exponential variations in wire force and bond force. It would be interesting to see a trial analysis of the experimental data made on the basis of these equations.

The method of attaching the SR-4 strain gages to the fabric wires warrants some additional attention. In test panel A, at the location of each gage, approximately 19 percent of the bond area and about 36 percent of the tensile area were lost. In test panel B the corresponding losses were about 16 and 31 percent, respectively. Thus the mechanical relationships governed by the area and perimeter of the longitudinal wires were somewhat distrubed in the length of the strain gage slots, even if the stress concentrations caused by the slots could be ignored. This is not to criticize the author for the use of the method, since no other method is accepted as appreciably better for the purpose.

The loads applied to the test panels (Figs. 11 and 12) seem unusually low. At the free end of the protruding wire at each panel a maximum load of 1,000 lb is indicated. This gives a maximum tensile stress of about 6,900 psi on the gross wire area in test panel A and about 5,100 psi in test panel B. Likewise, the corresponding maximum bond forces shown would give bond stresses of about 75 psi and 55 psi in the same panels, respectively. Because both of these stresses decrease rapidly along the wire toward the first gage slot in the concrete, it is difficult to imagine that stresses in magnitude of 80 percent of the yield strength of the wires were developed at a slot. This would indicate an extremely large stress concentration factor at a slot. In any case, the stresses imposed on gross areas of the wires in the experiments were quite small compared to stresses known to develop in continuously reinforced concrete pavement reinforcement. There is much evidence to indicate that maximum stresses which normally develop at cracks in such pavements cause bond slippage along the longitudinal wires of welded wire fabric, thus transferring the anchorage to the first transverse wire and longitudinal bond and transverse wires beyond.

By all means, the author should be encouraged in his commendable efforts to analyze the many difficult problems involved in the rational design of continuously reinforced pavements. He has made notable contributions to the present state of knowledge of these structural elements.

WILLIAM ZUK, Closure — The discussion of Professor Waling is very much appreciated. In particular, his refinement of the elastic support modulus and opinions on the shear slip are of interest. Perhaps the real fact of slip lies somewhere between the expression $\tau = K(u + \delta)$ and $\tau = Ku$. Instrumentation much more precise than that used in this study is necessary to confirm this.

The oversight error in Eq. 3 is also appreciated. The correct expression for Eq. 3 should have been $u = Q_1 e^{\alpha x} - Q_2$, resulting in Eq. 4 as

$$u = \frac{P_1' \alpha}{K\pi d_1} e^{\alpha x} - \delta$$

P(x) would then be $P_1' \, e^{\alpha x}$ or $\frac{P_1}{e^{\alpha L}} \, e^{\alpha x}$ and "q" would be $P_1' \, \alpha e^{\alpha x}$

The equations thus correspond to Professor Waling's. It should also be mentioned that the correction of Eqs. 3 through 6 also change the equations on "Comparison of Theory and Tests." R is revised to read

$$\frac{K \pi d_1 \, \beta e^{2 \alpha L}}{2 \alpha E_c + K \pi d_1 \, \beta e^{\alpha L}}$$

Perhaps as a result of simplification, there seems to be a misunderstanding in regard to the test load applied on the longitudinal wires. The actual peak loads imposed were of about 4 kips and 6 kips on panels A and B respectively. However, in plotting the curves (11) and (12) a unit value of 1 kip was used for comparison purposes. It is fully realized that at higher loads, more severe slipping would take place and a simple exponential expression for bond would no longer be true.

Mechanics of Continuously-Reinforced Concrete Pavements

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This investigation is primarily concerned with the deflections and the resulting stresses in a continuously-reinforced concrete pavement, loaded simultaneously with longitudinal and transverse loads. The theory developed for the computation of the deflections is based on the common assumptions used in the theory of continuous beams on elastic foundations, as well as three assumptions concerned with a cracked slab on an elastic foundation. The main assumptions in the first class are: (a) The deflection at a point some distance from the transverse load is zero, and (b) the subgrade modulus is constant throughout the full range of deflection. The assumptions peculiar to this problem are: (a) The cracks formed by volume changes in the pavement are equally spaced, (b) the segments between cracks are assumed to be straight, and (c) the moment at a crack is some function of the angle change (that is, $M/\phi = C$).

By considering the geometry of the deflected pavement and the equilibrium of the individual segments, a series of simultaneous equations may be written in terms of either the deflections or the angle changes at the cracks. Equations for shear and moment at the cracks are easily written.

The equations are in such a form as to consider any combination of pavement length, crack spacing, subgrade modulus, transverse load, and longitudinal load. In the solutions presented here, M/ϕ is considered equal to a constant, C. Experimentation is in progress to evaluate C for a series of pavement types.

THE THEORETICAL analysis of a pavement slab, either reinforced or unreinforced, is a highly complex problem. If all the possible physical characteristics of the loads, slab, and the support, that is, the subgrade, could be idealized, the remaining problem of computing deflections and stresses is very complicated if at all possible.

In the development of any basic theory for deflections and stresses in a pavement slab it is necessary to make certain assumptions as to the idealized nature of the physical characteristics of the problem. Having made these assumptions and having developed a theory based on the assumptions, it is necessary to verify by experimentation and observation the results of the theory. Any major discrepancies between theoretical and actual results must be accounted for by adjustments of basic theory. If it is not possible to reconcile the theory with actual conditions, then the theory must be discarded.

This is essentially the history of the analysis of concrete pavements. Early theories were unsuccessful at predicting with reasonable accuracy the stresses in a pavement slab and are of only historical value now (1).

Most of Westergaard's work on pavement slabs followed the pattern described earlier Westergaard developed equations for stresses in loaded pavement slabs, found that the computed stresses did not agree with experimental results and consequently modified his theory to bring about an agreement (2, 3, 4). The fundamental nature of his analysis for stresses is apparent in that the equations are of the form used in practically every other analysis, theoretical or empirical (5).

The development of a theory for continuously-reinforced concrete pavements will follow the same pattern. An idealization of the assumptions is necessary to arrive at a fundamental theory. The assumptions and the theory are then modified or adjusted to account for the variations from the ideal.

The first statement of the principles of continuously-reinforced concrete pavement was made in 1947 by W. R. Woolley who set forth the basic ideals governing crack spacing and steel stress (6). A number of field tests have been established to test some of his theories (7).. Many of the special problems of continuously-reinforced concrete pavements have been treated by Zuk (8). It is hoped that the theory developed in the following discussion will add to the fund of theoretical and experimental knowledge of continuously-reinforced concrete pavements.

PURPOSE AND SCOPE

The purpose of this paper is to develop a theory for the computation of stresses and deflections in continuously-reinforced pavements. A series of four pavement lengths varying from 20 to 50 ft, taken from the "middle" portion of a continuously-reinforced concrete pavement, are studied with respect to deflections, shears, and moments. Various combinations of load and subgrade support are used in the calculations.

The theory is intended to be a basic tool by which any combination of physical conditions in a continuously-reinforced concrete pavement may be represented mathematically. The great speed of calculation afforded by a digital computer will make it possible to represent the results of this theory in curve or table form for easy use by a highway engineer.

THEORETICAL DEVELOPMENT

This analysis of a continuously-reinforced pavement with transverse cracks occurring at intervals involves five major assumptions, as follows:

1. The transverse cracks occur in equally-spaced intervals;
2. The pavement is relatively straight between cracks;
3. The moment at a crack is some function of the angle change, that is, $M/\phi = C$;
4. The subgrade modulus is constant; and
5. The deflection at a point some distance from a transverse load is zero.

The first assumption is reasonably correct in that a pavement of more than a few hundred feet in length may be considered infinite in length and the middle portion, which is completely anchored against movement, will have more or less equally-spaced cracks. The second assumption is also quite correct. Observations on test specimens in the laboratory have shown that the slabs have very small curvatures between cracks. It would be expected that the cracks being relatively flexible compared to the pavement between cracks would account for most of the change in slope of the deflected slab. Assumption 3 is now the subject of laboratory research at Purdue University. Assumptions 4 and 5 are well-known assumptions used in most considerations of long beams on elastic foundations. In the following analysis of the slab an element 1 in. wide is considered.

A slab under the action of a transverse load will deflect until the sum of the subgrade reaction forces and the shearing forces on the ends of the slab equal the value of the load. Such a deflected configuration is assumed in Figure 1. The angle changes, ϕ_n, are shown in accordance with assumption 2. Deflections at each of the cracks are written in terms of the angle changes as follows:

$$\begin{aligned}
\Delta_1 &= \ell\phi_0 \\
\Delta_2 &= \Delta_1 + \ell(\phi_0 + \phi_1) = \ell(2\phi_0 + \phi_1) \\
\Delta_3 &= \Delta_2 + \ell(\phi_0 + \phi_1 + \phi_2) = \ell(3\phi_0 + 2\phi_1 + \phi_2) \\
\Delta_n &= \ell(n\phi_0 + (n-1)\phi_1 + (n-2)\phi_2 + \dots + \phi_{n-1})
\end{aligned} \tag{1}$$

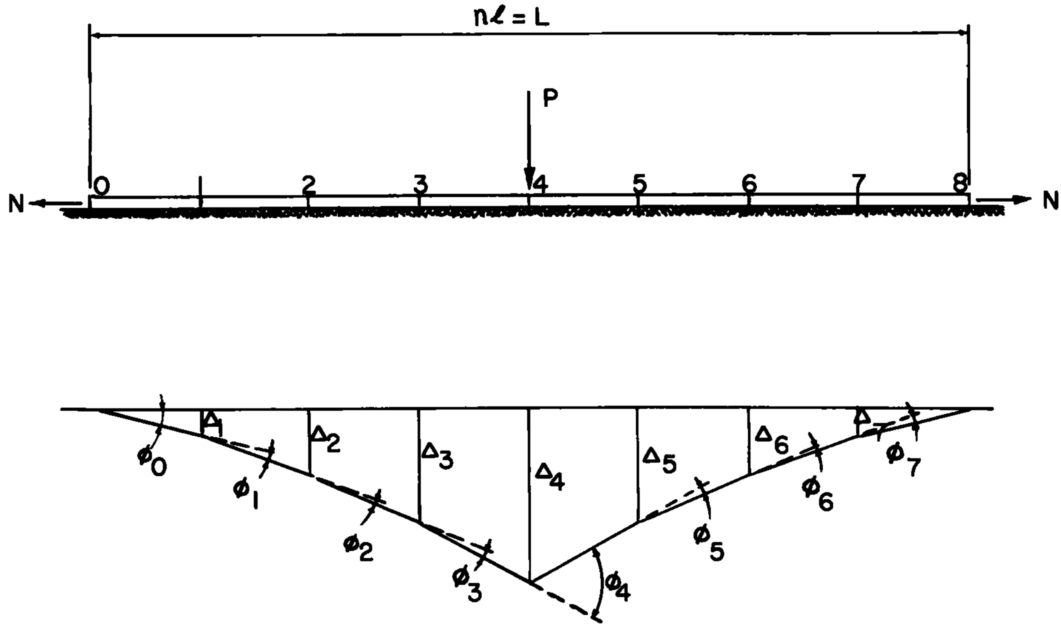

Figure 1. Typical deflected pavement.

The equations

$$\Delta_0 = 0; \Delta_n = 0 \qquad (2)$$

satisfy the requirements of assumption 5, and provide a necessary relationship in the final solution.

The total force exerted on each segment by the subgrade is equal to k times the volume of the subgrade displaced by that segment. Such a typical segment is shown in Figure 2 (a). The volume of the subgrade displaced is equal to the average of the end deflections of the segment times the length, ℓ. The resultant force is:

$$F_j = \ell/2\ (\Delta_j + \Delta_{j+1})k \qquad (3)$$

Figure 2 (b) shows the complete force system acting on a typical segment. The equilibrium of each segment, the vertical equilibrium of the slab as a whole, and the conditions of Eqs. 2 provide sufficient equations to solve for either the unknown deflections or the unknown angle changes. The unknown forces will be expressed in terms of the deflections, and then Eqs. 1 will be used to obtain the final form of the equations in terms of angle changes.

For simplicity the slab will be assumed to be loaded symmetrically, P being at the center line and a horizontal load N at each end. A certain number of segments, say eight, will be used for the purposes of definiteness. Because of the symmetry it is necessary only to consider half the length, the right half being a mirror image of the left half.

The equations for deflections are:

$$\begin{aligned}
\Delta_0 &= 0\\
\Delta_1 &= \ell\phi_0\\
\Delta_2 &= \ell(2\phi_0 + \phi_1)\\
\Delta_3 &= \ell(3\phi_0 + 2\phi_1 + \phi_2) \qquad (4)\\
\Delta_4 &= \ell(4\phi_0 + 3\phi_1 + 2\phi_2 + \phi_3)\\
\Delta_8 &= \ell(8\phi_0 + 7\phi_1 + 6\phi_2 + 5\phi_3\\
&\quad + 4\phi_4 + 3\phi_5 + 2\phi_6 + \phi_7) \text{ or}
\end{aligned}$$

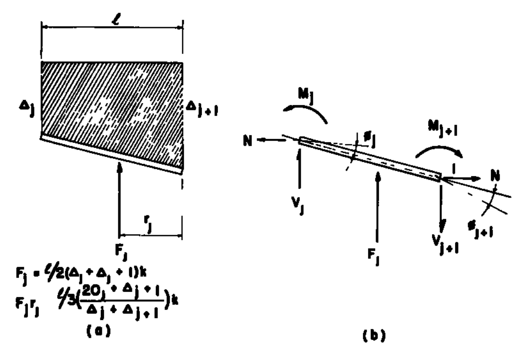

Figure 2. Forces on a typical segment.

$$\Delta_8 = \ell(8\phi_0 + 8\phi_1 + 8\phi_2 + 8\phi_3 + 4\phi_4)$$

The equations for the shears at each crack are:

$$\begin{aligned}
V_0 &= V_0 \\
V_1 &= V_0 + \ell k \Delta_1/2 \\
V_2 &= V_0 + \ell k\,(\Delta_1 + \Delta_2/2) \\
V_3 &= V_0 + \ell k\,(\Delta_1 + \Delta_2 + \Delta_3/2) \\
V_4 &= V_0 + \ell k\,(\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4/2)
\end{aligned} \tag{5}$$

The equilibrium of each segment taken individually yields the following equations:

$$\begin{aligned}
M_1 + V_0\ell + F_1r_1 - M_0 - N\Delta_1 &= 0 \\
M_2 + V_1\ell + F_2r_2 - M_1 - N(\Delta_2 - \Delta_1) &= 0 \\
M_3 + V_2\ell + F_3r_3 - M_2 - N(\Delta_3 - \Delta_2) &= 0 \\
M_4 + V_3\ell + F_4r_4 - M_3 - N(\Delta_4 - \Delta_3) &= 0
\end{aligned} \tag{6'}$$

The form of the final equations is somewhat simplified if the following approximation is made. By substituting $V_n = V'_n - \frac{N\Delta n}{\ell}$ into Eqs. 6, they become:

$$\begin{aligned}
M_1 + V'_0\ell + F_1r_1 - M_0 - N\Delta_1 &= 0 \\
M_2 + V'_1\ell + F_2r_2 - M_1 - N\Delta_2 &= 0 \\
M_3 + V'_2\ell + F_3r_3 - M_2 - N\Delta_3 &= 0 \\
M_4 + V'_3\ell + F_4r_4 - M_3 - N\Delta_4 &= 0
\end{aligned} \tag{6}$$

The values for V_n may be calculated when the solution for the deflections is obtained.

In Eqs. 5, setting $V_4 = P/2$ and solving for V_0 yields:

$$V_0 = P/2 - \ell k\,(\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4/2) \tag{7}$$

Substitution of Eqs. 7 and 4 into the remaining Eqs. 5 and 6 yields the following expressions in terms of the ϕ's alone:

$$\begin{aligned}
&(47/6\ell^2k + N + C/\ell)\phi_0 + (4.5\ell^2k - C/\ell)\phi_1 + \ell^2k\,(2\phi_2 + 0.5\phi_3) = P/2 \\
&(41/6\ell^2k + 2N)\phi_0 + (13/3\ell^2k + N + C/\ell)\phi_1 + (2\ell^2k - C/\ell)\phi_2 + 0.5\ell^2k\phi_3 \\
&\qquad = P/2 \\
&(29/6\ell^2k + 3N)\phi_0 + (10/3\ell^2k + 2N)\phi_1 + (11/6\ell^2k + N + C/\ell)\phi_2 + \\
&\qquad (0.5\ell^2k - C/\ell)\phi_3 = P/2 \\
&(11/6\ell^2k + 4N)\phi_0 + (4/3\ell^2k + 3N)\phi_1 + (5/6\ell^2k + 2N)\phi_2 + \\
&\qquad (\ell^2k/3 + N + C/\ell)\phi_3 = P/2
\end{aligned} \tag{8}$$

In Eqs. 4, setting $\Delta_8 = 0$ yields:

$$\phi_0 + \phi_1 + \phi_2 + \phi_3 + 0.5\phi_4 = 0 \tag{9}$$

This equation along with Eqs. 8 gives five equations with five unknowns which can be solved simultaneously.

In these equations the coefficients may be calculated after a choice is made of the values for the parameters ℓ, k, and N. The parameter C need not be a constant but might be some function of ϕ itself. In this instance the equations might be non-linear in terms of ϕ but would still yield a solution under normal physical conditions. The solutions discussed here are based on C equal to a constant for lack of better information.

The equations for the cases of 20, 16, and 12 segments of equal length are given in Appendix A.

Due to the inherent symmetry of an infinitely long pavement it is unnecessary to consider a case where P is not at the center crack. However, for finite slabs with N = 0, this possibility might be critical. In this case a similar set of equations may be written involving all of the angle changes and deflections as well as two different end shears, V_0 and V_n. A more formal statement of the equilibrium and boundary conditions makes this problem clearer.

Consider a slab with n segments, and a load P at the jth point. Consider also as unknowns the deflections, (n + 1) in number; and the two end shears, V_0 and V_n. This is a total of (n + 3)unknowns. The equilibrium condition of each segment yields n equations. The two boundary conditions $\Delta_0 = 0$, $\Delta_n = 0$ yield two more equations. The last equation which is necessary is obtained by considering the shear condition at the point of load. The condition is that the numerical sum of the shears to the left and to the right must equal the applied loads or:

$$\left| V_j \text{ (right)} \right| + \left| V_j \text{ (left)} \right| = P \qquad (10)$$

By substituting appropriately, the results obtained previously for the symmetrical case may be verified and similar equations may be obtained for the unsymmetrical case.

Although the equations are quite simple when the crack spacing, ℓ, is considered a constant, they are not much more complicated if some other arrangement of cracks is assumed. With a given crack distribution, either symmetrical or unsymmetrical, it would be necessary only to express each interval as some multiple of a unit length and carry these multipliers along in the equations.

RESULTS

The equations which are listed in Appendix A were solved for the combinations of parameters shown in Table 1. In each case P equals 250 lb per in. and ℓ equals 30 in.

The six combinations are used for each of the four slab lengths: namely, 8ℓ, 12ℓ, 16ℓ, and 20ℓ, giving a total of 24 different solutions.

The curves in Figures 3 through 8 show the results of a slab of length 8ℓ. Figures 9 and 10 show partial results for a slab of length 20ℓ. In each of the figures are shown the deflection, shear, and moment diagrams.

A number of interesting observations can be made concerning the results of the computations. The most obvious is the reduction in maximum deflection and maximum moment with an increase in subgrade modulus k. However, the relationship among the maximum deflection, maximum moment, and the subgrade modulus cannot be deduced without more computations.

The presence of the horizontal load N, has a slight effect on the deflections but practically no effect on the moments. The effect of higher values of N on the deflections and moments might be more pronounced, but again to determine this effect will require more computations.

The general shape of the diagrams presented agrees very well with the exact solution for a continuous beam on an elastic foundation (9). The characteristic vanishing of the deflection, shear, and moment at points of increasing distance from the point of application of load is evident. The fact that the curves, in addition to being deflection, shear, and moment diagrams for the fixed position of load are also influence lines for deflection, shear, and moment at a point is useful for the consideration of more than one load.

Figures 9 and 10 are shown for comparison with Figures 3 and 5, respectively. The purpose for the comparison of these two cases is to show specifically a fact that is true generally: namely, that only a small number of crack intervals need be considered in the solution. The use of a number of crack intervals larger than, for example, 10 to 20, depending possibly on the length of the crack interval, will yield no additional information, although it will in-

TABLE 1

Combination No.	C-inch-lbs/ inch/rad.	N-lbs/ inch	k-lbs/ cubic inch
1	2.5×10^6	1,000	150
2	2.5×10^6	1,000	440
3	2.5×10^6	0	150
4	2.5×10^6	0	440
5	0	1,000	150
6	0	1,000	440

DEFLECTION, SHEAR, AND MOMENT DIAGRAMS—SLAB OF LENGTH 8ℓ

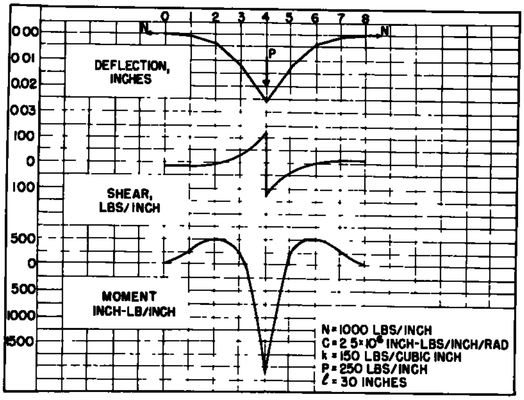

Figure 3.

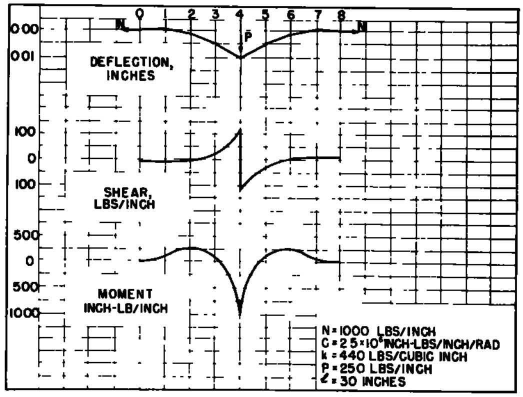

Figure 4.

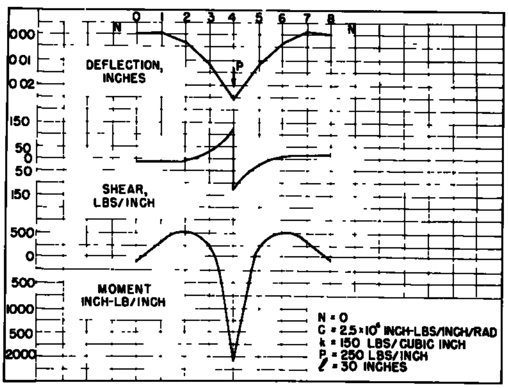

Figure 5.

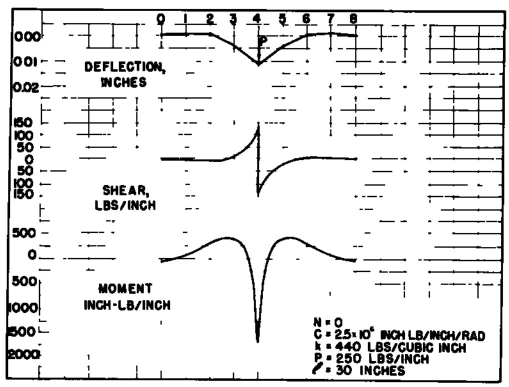

Figure 6.

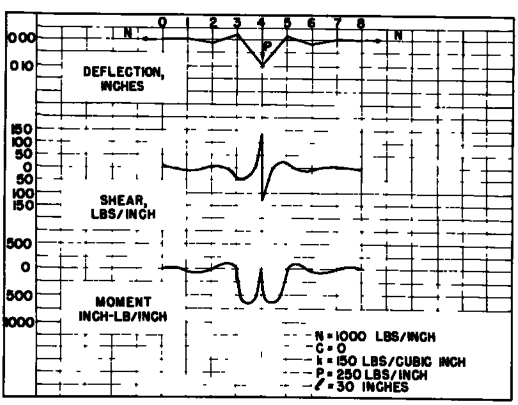

Figure 7.

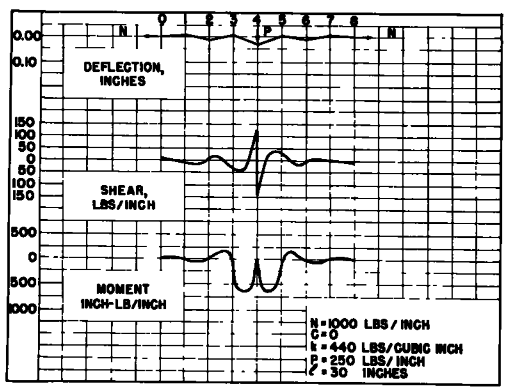

Figure 8.

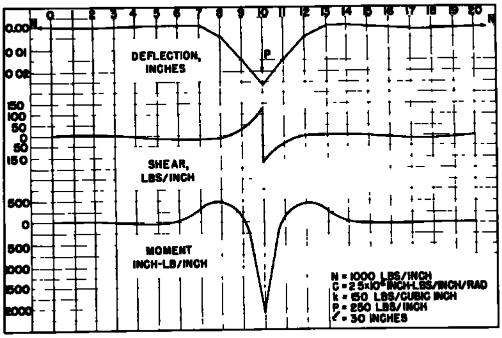

Figure 9.

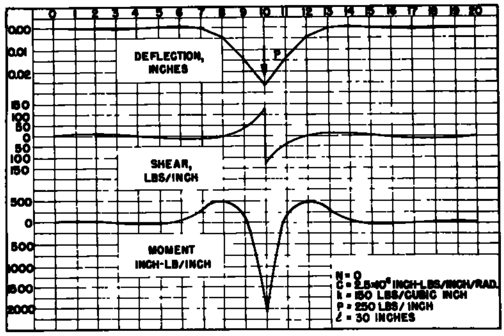

Figure 10.

crease the precision slightly. The optimum number of crack intervals in relationship to the crack spacing is still to be worked out. It seems unreasonable that the consideration of more than 20 intervals would ever be necessary to obtain an accurate solution in the region of the load.

The curves in Figures 7 and 8 are for the case where C = 0. In this case the only interaction between the segments is shear transfer across the crack. This case is likely to arise when, in a continuously-reinforced pavement, the crack width or opening is so large as to prevent the two concrete faces from coming into contact and thus provide resistance to moment. Such a condition in an actual pavement is very near if not at the state of failure. It is of interest here since it is the extreme possibility of the moment-angle change relationship.

The unusual moment diagrams in Figures 7 and 8 are the moment diagrams for the individual segments when the moments at the cracks equal zero (C = 0). Each segment is somewhat like a simply-supported beam loaded with a varying distributed load from the subgrade. The resulting moment diagrams are quite reasonable when considered in this manner.

CONCLUSIONS

Many questions must be answered before the method of analysis developed in this research can be applied to the design of continuously-reinforced concrete pavements.

1. What is the moment-angle change relationship for a wide range of pavement design variables (thickness of concrete, position of reinforcement, percentage of reinforcement)?
2. What is the necessary number of segments to be considered for the necessary accuracy with a given crack spacing?
3. What is the effect of high values of N on the moments?
4. What is the effect of the plate action of each segment?
5. What is the effect of repeated loads on the pavement design?
6. What is the optimum percentage and best position of steel for given loads and field conditions?

The first question is susceptible to laboratory investigation. Experimentation with relatively small specimens which contain only one crack will eliminate some variables which obscure the nature of the moment-angle change relationship. A specimen may be tested by the application at the crack of pure moment, pure shear, or a combination of moment and shear. Longitudinal loads to control crack openings may also be applied. Measurements of the relative angle changes between the two segments formed by the crack may be made. The relative vertical displacement of the segments may also be measured. By varying the percentage and position of the reinforcement as well as the loads on the crack, complete information about crack behavior can be obtained. Such an investigation is now in progress at Purdue University.

The answers to questions 2 and 3 can be readily obtained when the moment-angle change relationship has been established. In this investigation the optimum number of crack intervals is 8 using a crack spacing of 30 in., while it may be some other number for other values of ℓ. By the use of the Purdue University Digital Computer a large number of solutions may be easily worked out. The solutions may be programed in such a manner as to provide the final values of moments, shears, deflections, and stresses. With such a computer available it is possible to consider in a relatively short time the large range of parameters necessary to answer these questions.

In this investigation the pavement is treated as a beam 1 in. wide. In the actual pavement the cracks divide the slab into a series of transverse strips. The action of these strips under the load must be investigated. For rather closely-spaced cracks the strip acts somewhat as a beam, while the larger spacings of cracks yield segments which act more like plates. The effects of this plate action must be determined and appropriate modifications must be incorporated in the final design procedure.

The effect of repeated loads on the pavement design can be investigated in the laboratory. Repeated application of loads in the research outlined above would provide information on the fatigue characteristics of various designs. This information can be combined with traffic surveys for proposed highways in the final design of the pavement.

The answer to the last question is really the ultimate purpose of all these investigations. What combinations of slab thickness, percentage of reinforcement, and position of reinforcement will support the given load in a given field condition? The existence of an answer to this question implies the existence of a design procedure which can account for all of the variables in the problem. It is obviously very difficult experimentally to account for each variable separately and then combine their effects to arrive at a design for each given set of conditions. Perhaps the best design procedure then, is one which utilizes the results of verified mathematical solutions. The results may be presented in table and chart form so that engineers may design safe and economical continuously-reinforced concrete pavements. It is sincerely hoped that this research will help make this possibility a reality.

ACKNOWLEDGMENTS

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REFERENCES

1. Goldbeck, A. T., "Thickness of Concrete Slabs," Public Roads, 1:10, pp. 34-38 (1919).
2. Westergaard, H. M., "Computation of Stresses in Concrete Roads," HRB Proceedings, Vol. 5, pp. 90-110 (1925).
3. Teller, L. W., and Sutherland, E. C., "The Structural Design of Concrete Pavements," Parts I, II, and III, Public Roads, 16:8, 9, 10, pp. 145-158, 169-197, 201-221 (1935).

   Teller, L. W., and Sutherland, E. C., "The Structural Design of Concrete Pavements," Part IV, Public Roads, 17:7, 8, pp. 143-171, 175-192 (1936).
4. Westergaard, H. M., "Analytical Tools for Judging Results of Structural Tests of Concrete Pavements," Public Roads, 14:10, pp. 185-188 (1933).
5. Kelley, E. F., "Application of the Results of Research to the Structural Design of Concrete Pavements," Public Roads, 20:5, 6, pp. 83-105, 107-126 (1939).
6. Woolley, W. R., "Continuously Reinforced Concrete Pavements Without Joints," HRB Proceedings, Vol. 27, pp. 28-33 (1947).
7. Jacobs, W. H., "Continuously Reinforced Concrete Pavement Without Joints," Publication, Rail Steel Bar Association, Chicago, Ill. (1953).
8. Zuk, William, "Analysis of Special Problems in Continuously-Reinforced Concrete Pavements," HRB Bull. 214 (1959).
9. Hetenyi, M., Beams on Elastic Foundation, The University of Michigan Press, Ann Arbor, Michigan (1946).

Appendix A

The general equations for the solution for the values of the angle changes in the cases of L equal to 12ℓ, 16ℓ, and 20ℓ, respectively, are as follows:

$L = 12\ell$

$$(107/6\ell^2k + N + C/\ell)\phi_o + (12.5\ell^2k - C/\ell)\phi_1 +$$
$$\ell^2k(8\phi_2 + 4.5\phi_3 + 2\phi_4 + 0.5\phi_5) = P/2$$

$$(101/6\ell^2k + 2N)\phi_o + (37/3\ell^2k + N + C/\ell)\phi_1 + (8\ell^2k - C/\ell)\phi_2 +$$
$$\ell^2k(4.5\phi_3 + 2\phi_4 + 0.5\phi_5) = P/2$$

$$(89/6\ell^7k + 3N)\phi_o + (34/3\ell^2k + 2N)\phi_1 + (47/6\ell^2k + N + C/\ell)\phi_2 +$$
$$(4.5\ell^2k - C/\ell)\phi_3 + \ell^2k(2\phi_4 + 0.5\phi_5) = P/2$$

$$(71/6\ell^2k + 4N)\phi_o + (28/3\ell^2k + 3N)\phi_1 + (41/6\ell^2k + 2N)\phi_2 +$$
$$(13/3\ell^2k + N + C/\ell)\phi_3 + (2\ell^2k - C/\ell)\phi_4 + 0.5\ell^2k\phi_5 = P/2$$

$$(47/6\ell^2k + 5N)\phi_o + (19/3\ell^2k + 4N)\phi_1 + (29/6\ell^2k + 3N)\phi_2 +$$
$$(10/3\ell^2k + 2N)\phi_3 + (11/6\ell^2k + N + C/\ell)\phi_4 + (0.5\ell^2k - C/\ell)\phi_5 = P/2$$

$$(17/6\ell^2k + 6N)\phi_o + (7/3\ell^2k + 5N)\phi_1 + (11/6\ell^2k + 4N)\phi_2 +$$
$$(4/3\ell^2k + 3N)\phi_3 + (5/6\ell^2k + 2N)\phi_4 + (0.5\ell^2k + N + C/\ell)\phi_5 = P/2$$

$$\phi_o + \phi_1 + \phi_2 + \phi_3 + \phi_4 + \phi_5 + 0.5\phi_6 = 0$$

$L = 16\ell$

$$(191/6\ell^2k + N + C/\ell)\phi_o + (24.5\ell^2k - C/\ell)\phi_1 + \ell^2k(18\phi_2 + 12.5\phi_3 +$$
$$8\phi_4 + 4.5P_5 + 2\phi_6 + 0.5\phi_7) = P/2$$

$$(185/6\ell^2k + 2N)\phi_o + (73/3\ell^2k + N + C/\ell)\phi_1 + (18\ell^2k - C/\ell)\phi_2$$
$$+ \ell^2k(12.5\phi_3 + 8\phi_4 + 4.5\phi_5 + 2\phi_6 + 0.5_7) = P/2$$

$$(173/6\ell^2k + 3N)\phi_o + (70/3\ell^2k + 2N)\phi_1 + 107/6\ell^2k + N + C/\ell)\phi_2 +$$
$$(12.5\ell^2k - C/\ell)\phi_3 + \ell^2k(8\phi_4 + 4.5\phi_5 + 2\phi_6 + 0.5\phi_7) = P/2$$

$$(155/6\ell^2k + 4N)\phi_o + (64/3\ell^2k + 3N)\phi_1 + (101/6\ell^2k + 2N)\phi_2 + (37/3\ell^2k +$$
$$N + C/\ell_3) + (8\ell^2k - C/\ell)\phi_4 + \ell^2k(4.5\phi_5 + 2\phi_6 + 0.5\phi_7) = P/2$$

$$(141/6\ell^2k + 5N)\phi_o + (55/3\ell^2k + 4N)\phi_1 + (89/6\ell^2k + 3N)\phi_2 +$$
$$(34/3\ell^2k + 2N)\phi_3 + (47/6\ell^2k + N + C/\ell)\phi_4 + (4.5\ell^2k - C/\ell)\phi_5 +$$
$$\ell^2k(2\phi_6 + 0.5\phi_7) = P/2$$

$$(101\ell^2k + 6N)\phi_o + (43/3\ell^2k + 5N)\phi_1 + (71/6\ell^2k + 4N)\phi_2 +$$
$$(28/3\ell^2k + 3N)\phi_3 + (41/6\ell^2k + 2N)\phi_4 + (13/3\ell^2k + N + C/\ell)\phi_5 +$$
$$(2\ell^2k - C/\ell)\phi_6 + 0.5\ell^2k\phi_7 = P/2$$

$$(65/6\ell^2k + 7N)\phi_o + (28/3\ell^2k + 6N)\phi_1 + (47/6\ell^2k + 5N)\phi_2 +$$
$$(19/3\ell^2k + 4N)\phi_3 + (29/6\ell^2k + 3N)\phi_4 + (10/3\ell^2k + 2N)\phi_5 +$$
$$(11/6\ell^2k + N + C/\ell)\phi_6 + (0.5\ell^2k - C/\ell)\phi_7 = P/2$$

$$(23/6\ell^2k + 8N)\phi_o + (10/3\ell^2k + 7N)\phi_1 + (17/6\ell^2k + 6N)\phi_2 +$$
$$(7/3\ell^2k + 5N)\phi_3 + (11/6\ell^2k + 4N)\phi_4 + (4/3\ell^2k + 3N)\phi_5 +$$
$$(5/6\ell^2k + 2N)\phi_6 + (0.5\ell^2k + N + C/\ell)\phi_7 = P/2$$

$$\phi_o + \phi_1 + \phi_2 + \phi_3 + \phi_4 + \phi_5 + \phi_6 + \phi_7 + 0.5\phi_8 = 0$$

$L = 20\ell$

$$(299/6\ell^2k + N + C/\ell)\phi_o + (40.5\ell^2k - C/\ell)\phi_1 + \ell^2k(32\phi_2 + 24.5\phi_3 +$$
$$18\phi_4 + 12.5\phi_5 + 8\phi_6 + 4.5\phi_7 + 2\phi_8 + 0.5\phi_9) = P/2$$

$$(293/6\ell^2k + 2N)\phi_o + (121/3\ell^2k + N + C/\ell)\phi_1 + (32\ell^2k - C/\ell)\phi_2 +$$
$$\ell^2k(24.5\phi_3 + 18\phi_4 + 12.5\phi_5 + 8\phi_6 + 4.5\phi_7 + 2\phi_8 + 0.5\phi_9) = P/2$$

$$(281/6\ell^2k + 3N)\phi_0 + (118/3\ell^2k + 2N)\phi_1 + (191/6\ell^2k + N + C/\ell)\phi_2 +$$
$$(24.5\ell^2k - C/\ell)\phi_3 + \ell^2k\,(18\phi_4 + 12.5\phi_6 + 18\phi_6 + 4.5\phi_7 +$$
$$2\phi_8 + 0.5\phi_9) = P/2$$

$$(263/6\ell^2k + 4N)\phi_0 + (112/3\ell^2k + 3N)\phi_1 + (185/6\ell^2k + 2N)\phi_2 +$$
$$(73/3\ell^2k + N + C/\ell)\phi_3 + (18\ell^2k - C/\ell)\phi_4 +$$
$$\ell^2k(12.5\phi_5 + 8\phi_6 + 4/5\phi_7 + 2\phi_8 + 0.5\phi_9) = P/2$$

$$(239/6\ell^2k + 5N)\phi_0 + (103/3\ell^2k + 4N)\phi_1 + (173/6\ell^2k + 3N)\phi_2 +$$
$$(70/3\ell^2k + 2N)\phi_3 + (107/6\ell^2k + N + C/\ell)\phi_4 + (12.5\ell^2k - C/\ell)\phi_5 +$$
$$\ell^2k(8\phi_6 + 4.5\phi_7 + 2\phi_8 + 0.5\phi_9) = P/2$$

$$(209/6\ell^2k + 6N)\phi_0 + (91/3\ell^2k + 5N)\phi_1 + (155/6\ell^2k + 4N)\phi_2 +$$
$$(64/3\ell^2k + 3N)\phi_3 + (101/6\ell^2k + 2N)\phi_4 + (38/3\ell^2k + N + C/\ell)\phi_5 +$$
$$(8\ell^2k - C/\ell)\phi_6 + \ell^2k(45\phi_7 + 2\phi_8 + 0.5\phi_9) = P/2$$

$$(173/6\ell^2k + 7N)\phi_0 + (76/3\ell^2k + 6N)\phi_1 + (141/6\ell^2k + 5N)\phi_2 +$$
$$(55/3\ell^2k + 4N)\phi_3 + (89/6\ell^2k + 3N)\phi_4 + (34/3\ell^2k + 2N)\phi_5 +$$
$$(47/6\ell^2k + N + C/\ell)\phi_6 + (4.5\ell^2k - C/\ell)\phi_7 + (2\phi_8 + 0.5\phi_9) = P/2$$

$$(131/6\ell^2k + 8N)\phi_0 + (58/3\ell^2k + 7N)\phi_1 + (101/6\ell^2k + 6N)\phi_2 + (43/3\ell^2k + 5N)\phi_3$$
$$+ (71/6\ell^2k + 4M)\phi_4 + (28/3\ell^2k + 3N)\phi_5 + (41/6\ell^2k + 2N)\phi_6 +$$
$$(13/3\ell^2k + N + C/\ell)\phi_7 + (2\ell^2k - C/\ell)\phi_8 + 0.5\ell^2k\phi_9 = P/2$$

$$(83/6\ell^2k + 9N)\phi_0 + (37/3\ell^2k + 8N)\phi_1 + (65/6\ell^2k + 7N)\phi_2 +$$
$$(28/3\ell^2k + 6N)\phi_3 + (47.6\ell^2k + 5N)\phi_4 + (19/3\ell^2k + 4N)\phi_5 +$$
$$(29/6\ell^2k + 3N)\phi_6 + (10/3\ell^2k + 2N)\phi_7 + (11/6\ell^2k + N + C/\ell)\phi_8 +$$
$$(0.5\ell^2k - C/\ell)\phi_9 = P/2$$

$$(29/6\ell^2k + 10N)\phi_0 + (13/3\ell^2k + 9N)\phi_1 + (23/6\ell^2k + 8N)\phi_2 +$$
$$(10/3\ell^2k + 7N)\phi_3 + (17/6\ell^2k + 6N)\phi_4 + (7/3\ell^2k + 5N)\phi_5 +$$
$$(11/6\ell^2k + 4N)\phi_6 + (4/3\ell^2k + 3N)\phi_7 + (5/6\ell^2k + 2N)\phi_8 +$$
$$(0.5\ell^2k + N + C/\ell)\phi_9 = P/2$$

$$\phi_0 + \phi_1 + \phi_2 + \phi_3 + \phi_4 + \phi_5 + \phi_6 + \phi_7 + \phi_8 + \phi_9 + 0.5\phi_{10} = 0$$

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