Maryland Continuously-Reinforced Concrete Test Pavement

The Maryland continuously-reinforced concrete pavement project is part of a regional investigation of this type of pavement. A companion study in Pennsylvania holds the steel percentage and bar size constant while varying pavement and subbase thicknesses. In Maryland the pavement and subbase thicknesses are constant while the steel percentage and bar size are the variables.

The first section of this paper describes the project location, materials, design, and construction, the applied traffic and exposure, and observations and test results during the early life of the pavement.

The second section presents observations of strains and temperatures during construction and the first year following, and resulting crack widths correlated with strain and temperature.

I. Materials, Design, Construction and Early Test Observations

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• CONSTRUCTION of an experimental continuously-reinforced concrete pavement in Maryland was discussed with representatives of the Bureau of Public Roads as far back as early 1956. It was agreed that such a pavement would be desirable and that it should be constructed on one of Maryland's major highways. It was also agreed that the pavement should include a different set of variables than those which had already been selected for inclusion in two similar projects in Pennsylvania. Accordingly, plans were prepared for a cooperative experimental project to be incorporated in the Baltimore-Harrisburg Expressway (Interstate 83). This project was built in June and July 1959.

In a sense, the Maryland and Pennsylvania projects constitute a regional investigation of continuously-reinforced pavements. Briefly, the main experimental features in each State are as follows:

In Pennsylvania the percentage of steel and the bar size are constant while pavement and subbase thicknesses are varied.

In Maryland the pavement and subbase thicknesses are constant while percentage of steel and bar size are the variables.

The Maryland project, as previously mentioned, is part of the Baltimore-Harrisburg Expressway (Interstate 83). It is in the general vicinity of Hereford and Parkton, some 20 miles north of Baltimore. This is in the Piedmont area and the terrain traversed by the project is quite rough. Its maximum horizontal curvature is 2° in the northbound roadway and 3°30′ in the southbound roadway. Grades range from —6.0 to +4.5 percent in the northbound roadway and from —5.0 to +4.5 percent in the southbound roadway.

Figure 1 shows the general location of the project. The new facility provides a relocation for US 111.

Figure 2 shows a portion of the project, including the Gunpowder Falls Bridges.

Interstate 83 in December 1960 carried about 5,250 vehicles per day, including 25 percent trucks and buses. Table 1 gives the axle weights and frequencies of trucks using this facility about three months after it was opened to traffic.

PROJECT OBJECTIVES

The principal over-all objective of this experimental project is to compare the general performance of a continuously-reinforced concrete pavement with that of a conventional jointed pavement subject to essentially the same traffic loads.

Within the sections of continuously-reinforced pavement the detailed objectives are as follows:

1. A study of the relative effects of 0.5, 0.6 and 0.7 percent of longitudinal reinforcement consisting of hard-grade billet steel bars.
2. A study of the relative effects of bar sizes Nos. 4, 5 and 6 with the longitudinal steel constant at 0.5 percent.
3. A study of the behavior of three different types of transverse terminal joints.
4. A study of the behavior of the pavement in the vicinity of transverse construction joints, as affected by curing methods of wet earth, cotton mats, burlap, and paper.
5. A study of strain in the concrete and the longitudinal steel reported in detail in the second part of this paper.

TEST SECTIONS AND EXPERIMENTAL FEATURES

The experimental pavement consists of nine continuously-reinforced test sections 8 in. in thickness ranging in length from about 1,800 to 4,150 ft and two 9-in. thick control sections of conventional jointed concrete pavement, one 1,880 and the other 2,850 ft long. Figure 3 shows

TABLE 1

AXLE FREQUENCY WEIGHT DISTRIBUTION BY TRUCK TYPE PER 24-HOUR PERIOD, MARCH 1960, BALTIMORE-HARRISBURG EXPRESSWAY TEST AREA

Axle Weight (lb)	Axle Frequency (no.)						
	Two-Axle Unit			Three-Axle Unit		Four-Axle Unit, Tractor Semi-Trailer	Total
	Panel or Pickup	Single Truck	Single Truck, Dual Tires	Single Truck	Tractor Semi-Trailer		
Under 8,000	226	24	277	37	293	1,035	1,892
8,000–8,499	—	—	13	3	22	121	159
8,500–8,999	—	—	11	2	9	94	116
9,000–9,499	—	—	8	3	10	42	63
9,500–9,999	—	—	9	1	8	41	59
10,000–10,999	—	—	11	2	16	60	89
11,000–11,999	—	—	5	—	20	53	78
12,000–12,999	—	—	3	—	31	51	85
13,000–13,999	—	—	4	2	24	70	100
14,000–14,999	—	—	3	3	25	90	121
15,000–15,999	—	—	3	2	17	107	129
16,000–16,999	—	—	6	1	15	153	175
17,000–17,999	—	—	1	—	18	146	165
18,000–18,999	—	—	6	—	13	117	136
19,000–19,999	—	—	5	—	12	73	90
20,000–21,999	—	—	17	1	14	127	159
22,000–23,999	—	—	3	—	16	57	76
24,000–25,999	—	—	1	—	4	12	17
26,000–27,999	—	—	—	—	—	2	2
28,000–29,999	—	—	—	—	—	1	1
Total axles	226	24	386	57	567	2,452	3,712
Total trucks	113	12	193	19	189	613	1,139

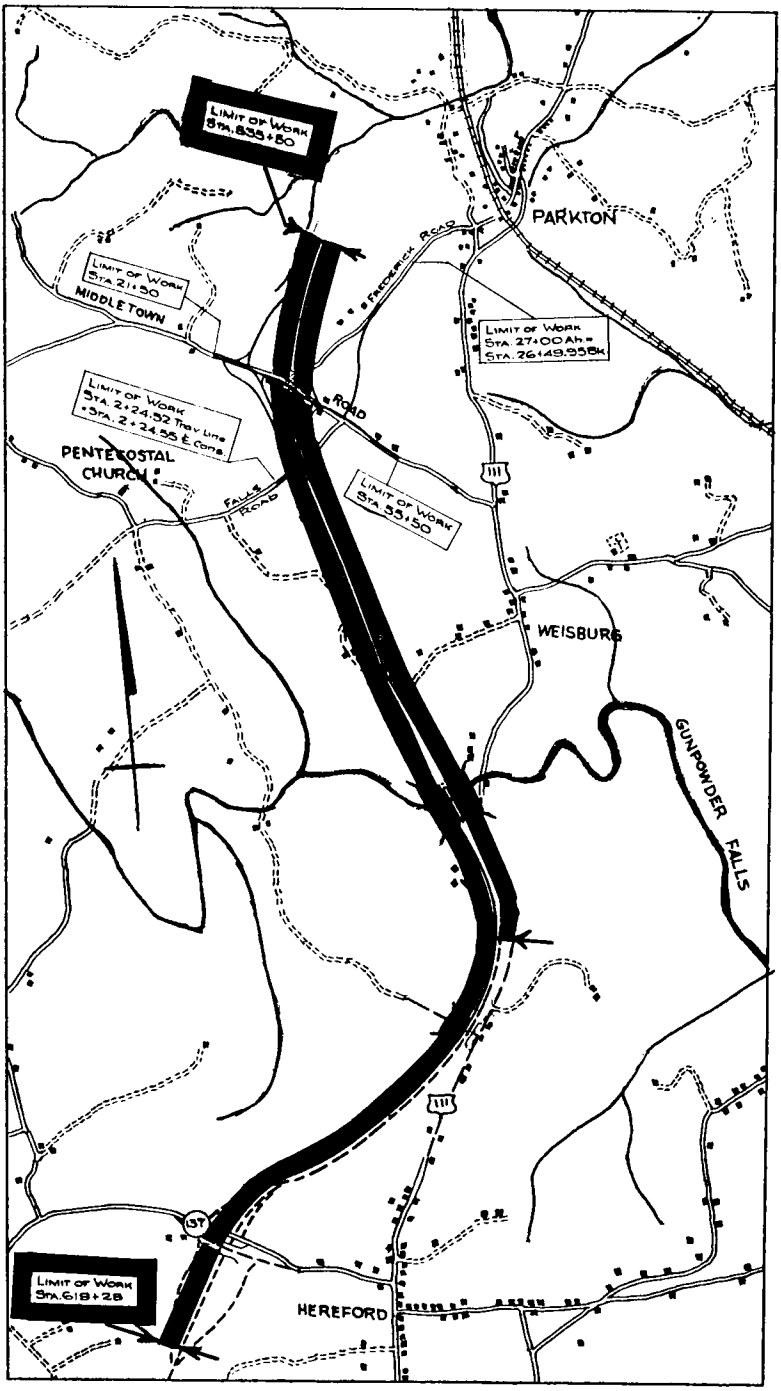

Figure 1. Maryland continuously-reinforced concrete test pavement is part of the Baltimore-Harrisburg Expressway (Interstate 83) about 20 miles north of Baltimore.

Figure 2. Portion of experimental project, showing Gunpowder Falls Bridges.

the locations of the various test sections of continuously-reinforced pavement, the conventional control sections, and the three types of terminal joints. As noted, the project is a part of both the northbound and the southbound roadways of the divided highway, the latter roadway having the greater length of experimental pavement.

Six of the continuously-reinforced test sections are in the southbound roadway. Three of the six contain 0.5, 0.6 and 0.7 percent longitudinal steel provided by No. 5 bars. These are built without intermediate terminal joints, thus making a continuously-reinforced length of nearly 9,750 ft. In the other three sections the longitudinal reinforcing steel is constant at 0.5 percent and bar sizes are Nos. 4, 5 and 6. A section of conventional jointed pavement is also included in this roadway for comparison or control purposes. This section has sawed contraction joints at 40-ft intervals and expansion joints spaced 600 ft apart. The 40-ft slabs are reinforced with 6 x 12 – 2/2 welded wire fabric. Load-transfer devices in both types of joints are ¾-in. diameter dowels, 24 in. long spaced on 12-in. centers.

In the northbound roadway there are three continuously-reinforced test sections and one control section. The three test sections are duplicates of those in the southbound roadway with the varying steel percentage; but unlike their companions are separated so that terminal joints are placed at each end of each section. Figure 4 shows a typical cross-section of one roadway of the continuously-reinforced pavement; Figure 5, the conventional jointed pavement.

The three types of terminal joints included for observation and study are as follows:

1. *Rubber Joint*—This is a modification of the bellows-like expansion joint developed by the B. F. Goodrich Company for the Jones and Laughlin experimental, 5-in. thick, prestressed concrete pavement. The modification consisted of adding a finger-type framework beneath the joint to provide for the greater pavement thickness of this project. Five of these rubber joints were installed for study. In a free state they are 13¼

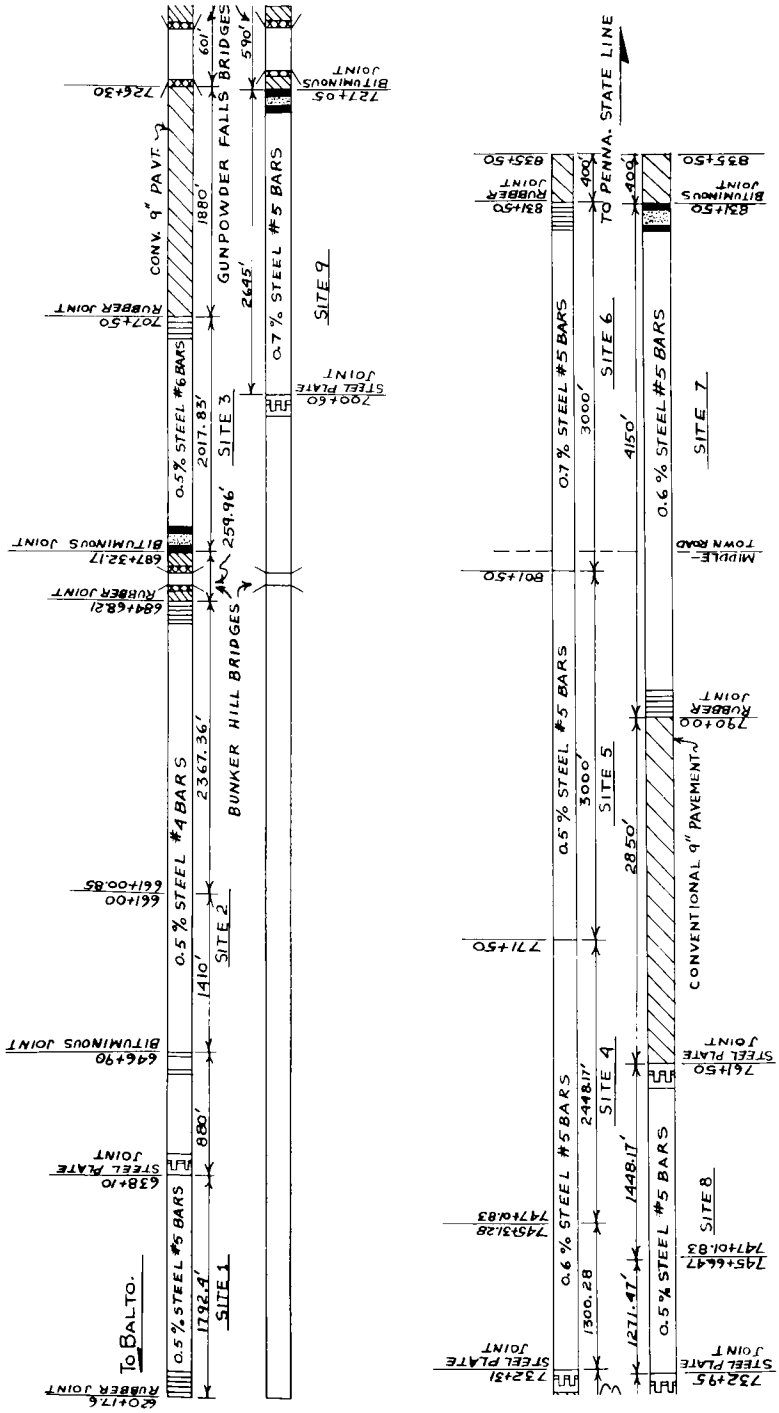

Figure 3. Relation of various test sections, control sections, and terminal joints.

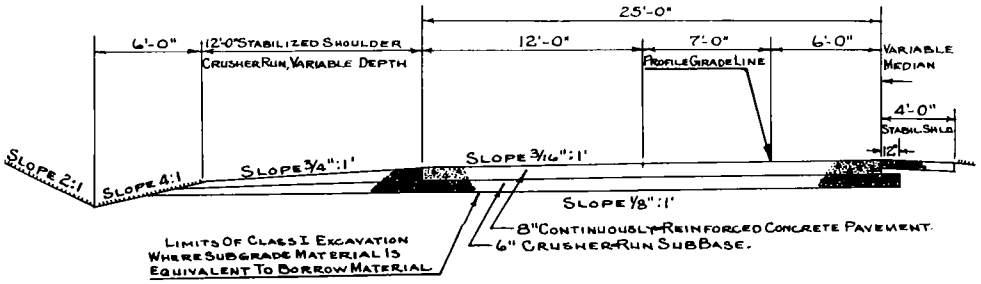

Figure 4. Typical section of 8-in. continuously-reinforced concrete test pavement.

in. wide and are so designed that they can be compressed to 9¾ in. working compression. As installed, the joints were compressed to about 11½ in. to provide for subsequent contraction of the pavement. Figure 6 shows the general features of the joint.

2. *Steel-Plate Joint*—This is a fabricated, steel-plate, finger-type joint which follows a general design suggested by an engineer of the Bureau of Public Roads. Five of these joints are installed in the project. As shown in Figure 7, the sliding joint between the steel fingers is not located directly over the opening between adjacent slabs, but is offset so as to be supported on one side of the joint to prevent infiltration of solids into the joint opening.

3. *Bituminous Joint*—This joint consists simply of a 4-in. gap packed with a mixture of cork and bituminous material. The ends of the pavement slabs forming the gap are armored with a steel angle. Four of these joints are included in the project. Design details are shown in Figure 8.

It is to be noted that a reinforced subslab or sleeper slab was used only in conjunction with the rubber joint, the slab ends forming the other two types of joints being placed directly on the subbase. Unlike the relatively narrow, rigid sills used in the past, this sleeper slab is 10 ft long and 6 in. thick. These dimensions are expected to produce elastic action in the slab comparable to that which develops in the pavement itself and thus, possibly, overcome the failures associated with narrow, rigid sills.

Nineteen construction joints are included in the previously mentioned study of curing methods. Saturated cotton mats were used to cure the pavement in the immediate vicinity of seven of these joints, saturated burlap in five cases, paper with a reflective surface in four cases, wet

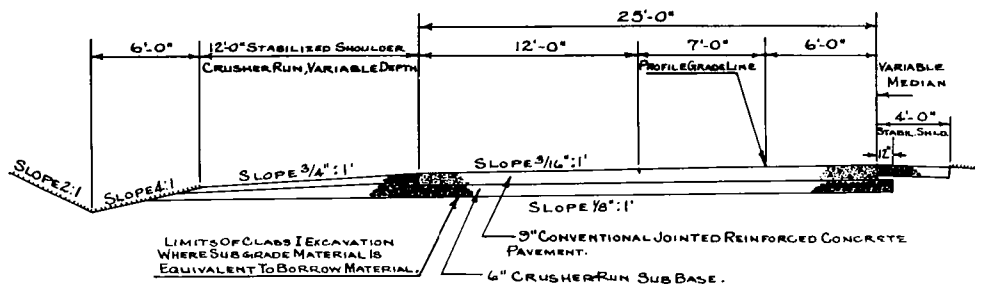

Figure 5. Typical section of 9-in conventional jointed concrete pavement.

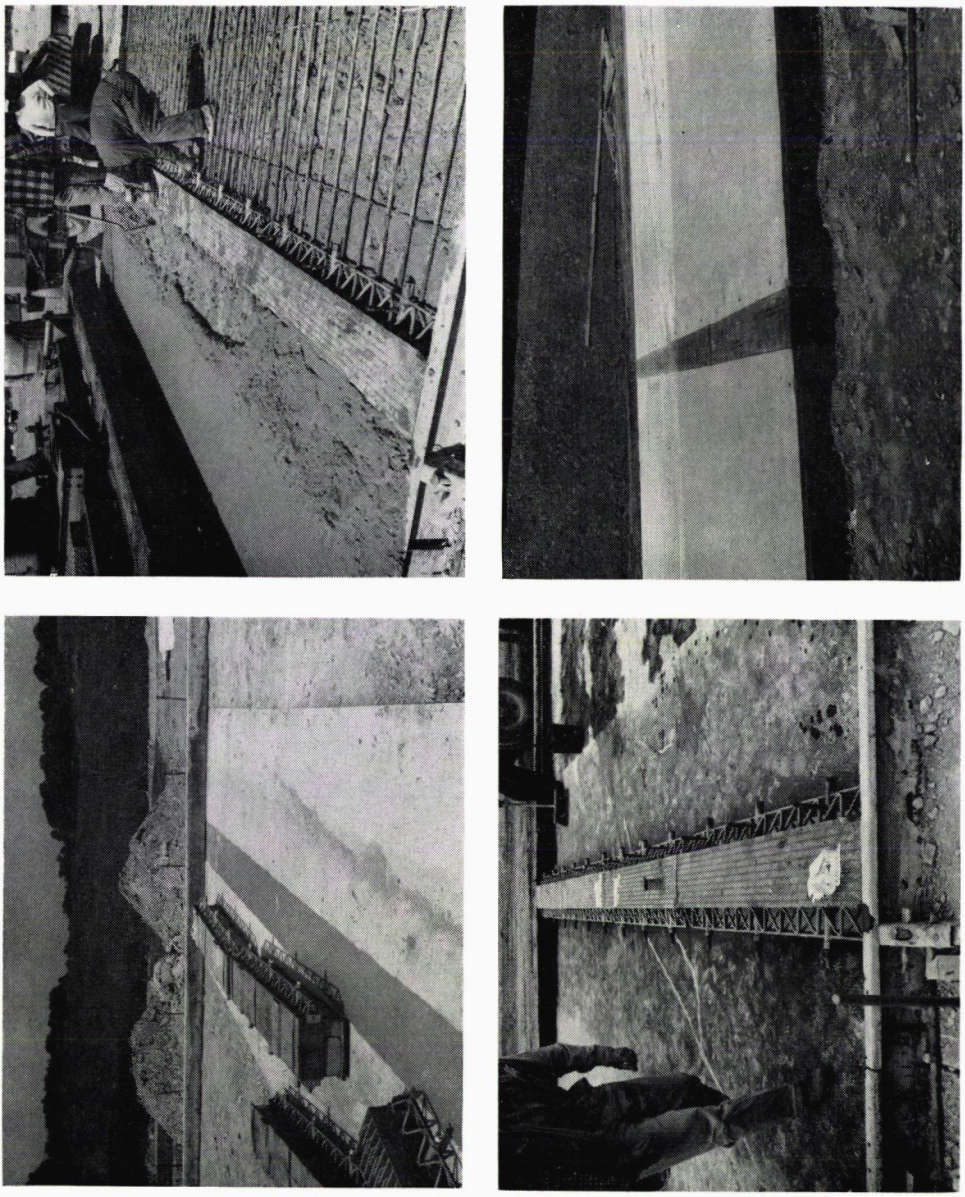

Figure 6. General features of the rubber terminal joint.

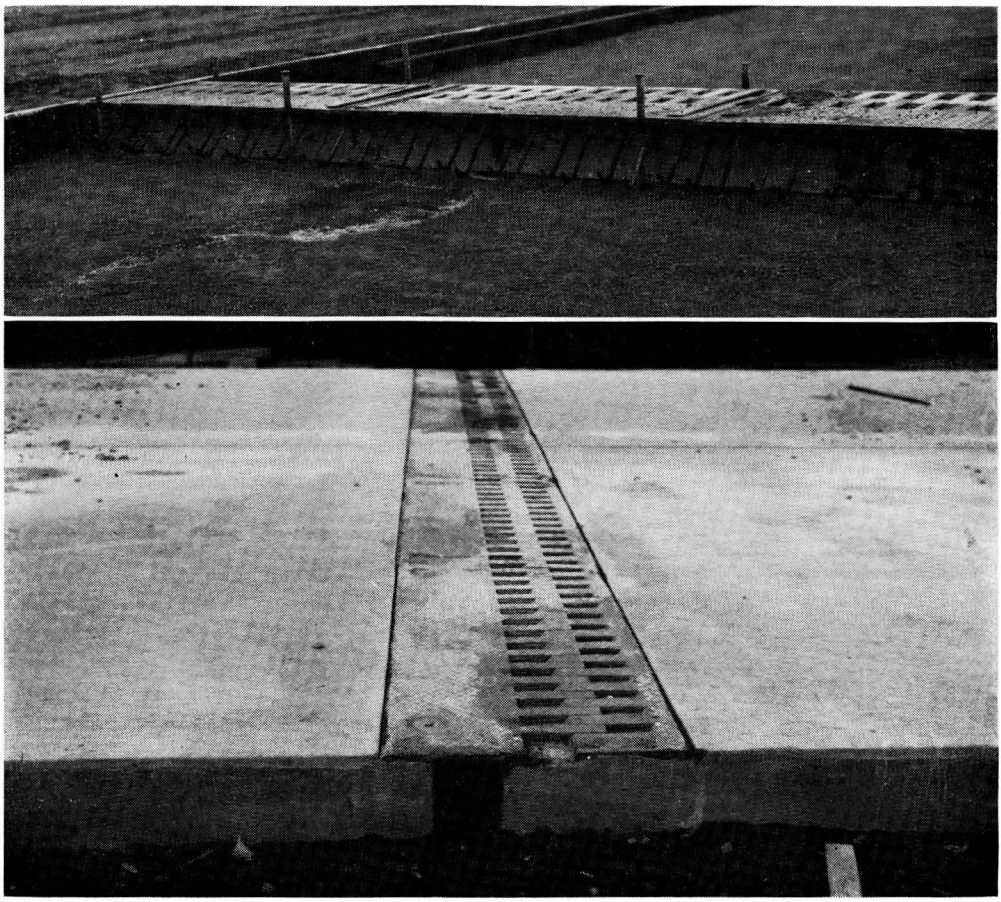

Figure 7. Details of the steel-plate terminal joint.

earth in two cases, and in one instance paper and wet earth were used on opposite sides of the joint. In all cases the curing materials were applied for a distance of at least 75 ft on each side of the joint.

Six instrumented test panels were placed in the southbound roadway, one in the central area of each of the six test sections. These panels are provided with (a) strain gages of the SR-4 type to determine strains in the steel and concrete, (b) resistance coils to measure temperature in the slab, subbase, and subgrade, and (c) lines of Whittemore gage plugs to measure the opening of induced cracks. The observation and analysis of these measurements and correlation with laboratory studies are the objectives of research work being conducted by the University of Maryland for the Maryland State Roads Commission and described more fully in Part II.

CONSTRUCTION MATERIALS

A complete investigation of the underlying soils was made for this project in accordance with the standard procedures of the Maryland State Roads Commission. The predominant type of soil was found to be an

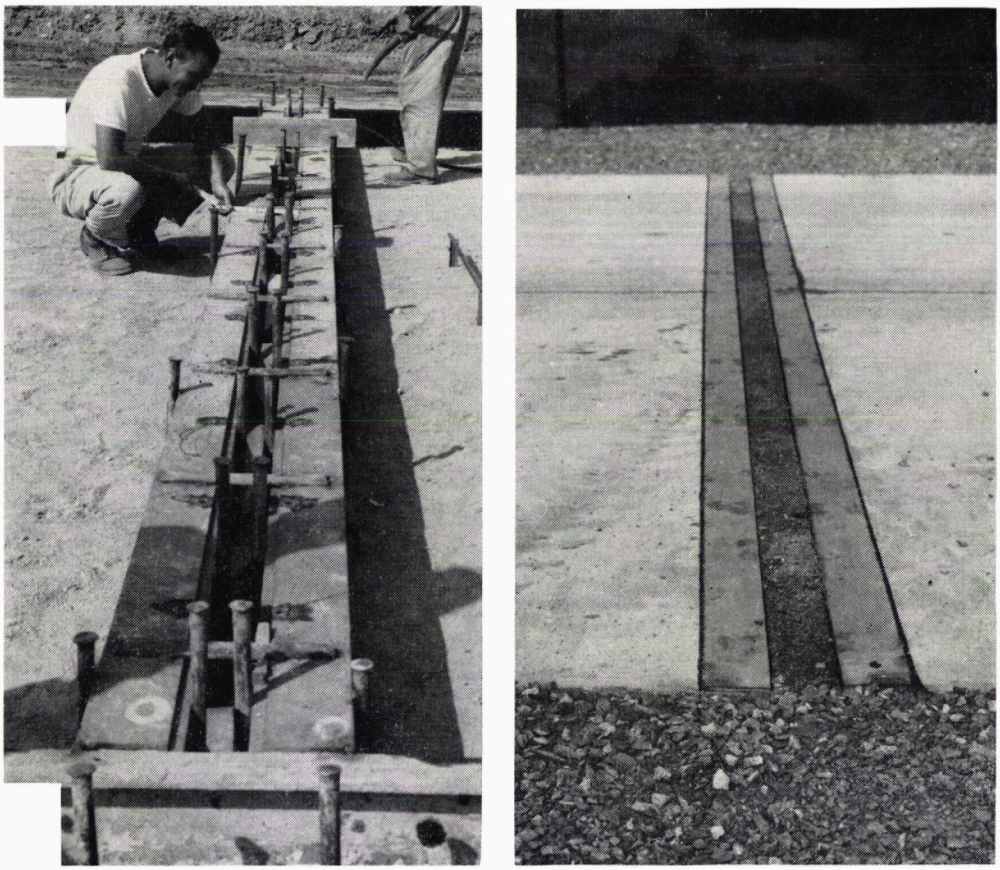

Figure 8. Details of bituminous terminal joint.

A-5 micaceous silt and decomposed rock. However, in many areas along the project blasting was necessary to remove the rock encountered. The entire pavement is supported on a 6-in. thick subbase of selected material consisting of crushed stone with a specified Los Angeles abrasion loss of not more than 50 percent. Just prior to concreting, samples of the subbase and subgrade material were taken from the finished grade at intervals of about 1,000 ft.

The concrete paving mix was designed in accordance with the Maryland State Roads Commission's January 1957 specifications for class P concrete. This is specified as an air-entrained concrete with a minimum cement content of 1.5625 barrels per cubic yard and a slump within the range of 1 to 2½ in. The slump produced was generally within the specified range, but in some periods of high wind and high temperature the mix water had to be adjusted so that the slump was about at or slightly over the maximum limit. The cement was type A-1 air-entrained.

The fine aggregate was a sand obtained from Perryman, Md. Its average fineness modulus as determined from a number of tests was 2.85. The coarse aggregate was a crushed serpentine obtained from Blue Mount, Md. It was about 50 percent No. 2

size and 50 percent No. 6 size, these sizes corresponding to simplified practice sizes Nos. 3 and 57, respectively. The proportions fixed for a batch of concrete were 814:1451:2836, using dry weight in pounds as a basis.

In the continuously-reinforced pavement, the reinforcement consisted of deformed billet-steel bars conforming to the requirements of ASTM Designation A-15 hard grade. Details of typical bar mats are shown in Figure 9. The conventional jointed pavement was reinforced with welded wire fabric conforming to AASHO Designations M-32 and M-55. Details of the welded wire fabric are shown in Figure 10.

Test results and other pertinent information on the subgrade and subbase materials and on the concrete and the reinforcing steel are given in Table 2.

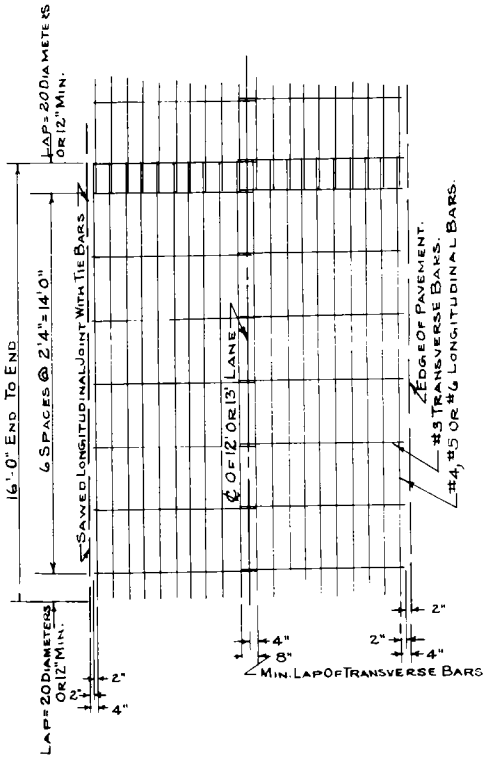

Figure 9. Details of typical bar mats.

PAVEMENT CONSTRUCTION

Paving began at the north end of the southbound roadway in early June of 1959. It proceeded southward to the Gunpowder Falls Bridge, moved over to the northbound roadway, and proceeded northward to the north end of the project. Paving then moved to the south limits of the northbound roadway, proceeded northward to the Gunpowder Falls Bridge, switched over to the southbound roadway and proceeded southward to the south limits of the project.

The sequence of paving the continuously-reinforced sections and the conventional jointed control sections was as follows:

Section No.	Roadway	Direction of Paving
6	Southbound	N to S
5	Southbound	N to S
4	Southbound	N to S
8	Northbound	S to N
Control	Northbound	S to N
7	Northbound	S to N
	Gunpowder Falls Bridge	
9	Northbound	S to N
Control	Southbound	N to S
3	Southbound	N to S
2	Southbound	N to S
1	Southbound	N to S

The experimental pavement was constructed full roadway width. It has a sawed longitudinal joint with 48-in. long $\frac{5}{8}$-in. diameter tie bars spaced 48 in. apart. The concrete was placed in two lifts, the first being laid for a distance of 75 to 80 ft in advance of the second. A single dual-drum paver was used. Vibration was not employed, except for internal vibrators trailing the spreader close to the side forms.

The reinforcing steel was positioned at mid-depth in the 8-in. con-

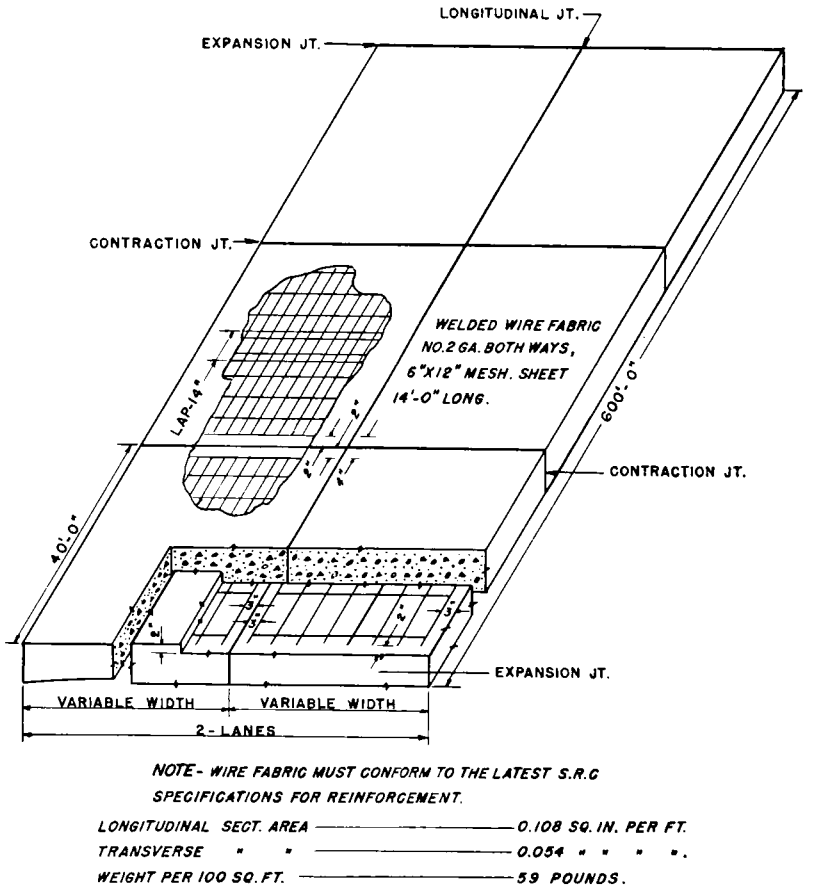

Figure 10. Details of welded wire fabric for conventional jointed pavement.

tinuously-reinforced sections and at 2 in. from the top surface in the conventionally reinforced control sections. Two workmen placed the sheets of welded wire fabric in the control sections; four workmen were required for placement of the heavier bar mats in the continuously-reinforced sections. Paving operations for both types of pavement were essentially similar and no unusual conditions were encountered.

The instrumented mats were positioned in exactly the same way as any other mat. Installation of the instrumented mats is described in detail in Part II.

The splice between bar mats was specified as 20 diameters, but in no case less than 12 in. Every splice was checked for length and four bars were securely wired together at each splice to prevent any possible creep in the mats due to finishing operations. No attempt was made to stagger the ends of the mats, but some stagger did occur as curves were traversed by the paving operations. The installation of the mats was well organized and no difficulty was encountered from placing the three different bar sizes and the three different steel percentages.

The three types of terminal joints

TABLE 2

DATA ON TEST SECTION MATERIALS

(*a*) SUBGRADE[1]

AASHO Soil Group and Index	Samples in Each Group (%)	% Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Geological Description	Agricultural Description
A–1–b (0)	20.0	8–23	31–36	5.6–5.7	Wissahickon schist; albite chlorite mica schist; northwestern facies	Chester loam; Chester stony loam; and Manor loam
A–2–5 (0)	3.3	33	42	7.3		
A–2–4 (0)	13.3	34–35	33–39	1–6		
A–4 (0 to 3)	36.7	37–50	30–40	1–7		
A–5 (0 to 6)	23.4	37–61	41–48	5–7		
A–7–5 (7)	3.3	52	50	15		

(*b*) SUBBASE[2]

Percent Passing							Liquid Limit	Plasticity Index
2-In. Sieve	1-In. Sieve	⅜-In. Sieve	No. 4 Sieve	No. 10 Sieve	No. 100 Sieve	No. 200 Sieve		
100	68	—	32	—[3]	—	2.5	—	N.P.
—	—	100	98	—[3]	17	—	—	N.P.

(*c*) CONCRETE[4]

Aggregate			Cement Factor	Slump (in.)		Age (days)	Compr. Str. (psi)		Flex. Str. (psi)	
Coarse		Fine		Range	Avg.		Range	Avg.	Range	Avg.
Type	Max. Size (in.)	Type								
Crushed stone (serpentine)	2	Siliceous sand		1.75–3.00	2.47	7	—	—	400–709	541
						14	3,035–3,750	3,380	460–716	614
						28	3,395–4,575	3,985	516–683	22

(*d*) LONGITUDITAL REINFORCING STEEL

Type	Grade	Bar Size	Tensile Strength (psi)				Elongation (%)	
			Yield Point		Ultimate		Range	Avg.
			Range	Avg.	Range	Avg.		
Billet	Hard (A–15)	No. 4	73,300–80,200	76,750	127,700–128,300	128,000	11.1–11.9	11.5
Billet	Hard (A–15)	No. 5	50,200–82,000	64,570	83,000–139,500	110,000	11.3–23.4	17.0
Billet	Hard (A–15)	No. 6	58,000–58,500	58,250	100,100–101,600	100,800	14.4–18.1	16.3

[1] Encountered in cut areas, and represents subgrade in cut and fill areas (no borrow required).
[2] Gradation specification.
[3] No. 10 screenings used as choke.
[4] Air-entrained.

were placed without incident. However, great care had to be exercised to insure that the concrete penetrated to all areas, particularly the under side of bearing plates.

A split timber bulkhead was installed at the end of the day or at any time it became necessary to cease paving operations. The bar mats were extended a minimum of 4 ft beyond the bulkhead. Extra deformed bars were inserted between the longitudinal bars of the mats to provide additional shear resistance at the construction joint. These extra bars were 3 ft long and of the same size as the longitudinal bars of the particular mat being used.

The pavement was cured with impermeable paper having a reflective surface. The paper remained in place a minimum of 72 hr after placement of the concrete.

TESTS AND OBSERVATIONS DURING AND AFTER CONSTRUCTION

HRB Correlation Service Circular No. 372 was followed very closely in connection with tests and observations leading to the ultimate evaluation of the project's performance. Some of the tests and observations have been made, others are in progress, and will continue for some years in the future. Circular No. 372, prepared by the Committee on Rigid Pavement Design and titled "Recommendations for Experimental Continuously-Reinforced Concrete Pavements," was published in November 1958.

Tests and observations during construction were as follows:

1. Maximum and minimum temperatures during the paving and curing periods.
2. Mid-depth temperatures of the concrete pavement at 200-ft intervals, just ahead of the final finishing operations.
3. Soil samples from both the subbase and subgrade materials at approximately 1,000-ft intervals.
4. Assembly of the details of all tests relating to concrete, aggregate, etc.
5. Pertinent notes pertaining to construction operations by construction and research personnel.
6. Complete photographic records.

Tests and observations since completion of construction have been as follows:

1. Annual range in air temperatures (obtained from U.S. Weather Bureau sources).
2. Semi-annual general surface condition inspection by District Engineer and Research Engineer.
3. Detailed crack surveys of central 500 ft of each continuously-reinforced section immediately following curing period; 2 to 3 weeks after curing; September 15, 1959; December 15, 1959; March 15, 1960; June 15, 1960; September 15, 1960. Future surveys are to be made at annual intervals.
4. Surface widths of representative pavement cracks in each test section located (a) in central 500 ft, (b) 500 ft from one end, and (c) immediately following a construction joint. Four cracks were plugged in each area, with zero readings by means of Whittemore gage and scale microscope correction.
5. Photographic record of selected cracks in each test section.
6. Cores from several apparently wider cracks (program not yet undertaken).
7. Absolute longitudinal movements at terminal ends of test sections.
8. Changes in width of the several types of terminal joints.
9. Determination of roughness by BPR-type roughometer; original observation just before opening to traffic, repetitions at three-year intervals.
10. Compilation of axle weights and frequencies three months after opening to traffic (see Table 1); rep-

TABLE
DATA ON CONSTRUCTION CONDITIONS AND AVERAGE

Sect. No.	Steel Reinf.			Pouring Conditions			
	%	Bar Size	Combination	Date (1959)	Air Temp. (°F)	Humidity (%)	Weather
1	0.5	No. 5	A	7/8	60–87	78–32	Sunny
				7/9	63–90	79–39	Sunny
				7/10	74–75	95–86	Sunny, showers
2	0.5	No. 4	B	7/3	65–86	66–35	Sunny
				7/6	68–80	90–61	Ptly. cloudy, showers
				7/7	67–84	75–32	Sunny
3	0.5	No. 6	C	7/1	88–94	61–50	Sunny
				7/2	76–90	75–36	Sunny
4	0.6	No. 5	D	6/9	95	30	Sunny
				6/10	73–93	71–34	Sunny
				6/11	70–97	77–42	Sunny
				6/12	66–80	95–73	Cloudy, showers
5	0.5	No. 5	A	6/5	66–70	85–77	Cloudy, showers
				6/8	65–93	85–30	Sunny
				6/9	73–95	66–35	Sunny
6	0.7	No. 5	E	6/3	68–77	71–61	Partly cloudy
				6/4	59–83	88–41	Sunny
				6/5	64–80	89–54	Cloudy, showers
7	0.6	No. 5	D	6/19	68–84	49–33	Ptly. cloudy, showers
				6/22	71–90	56–43	Sunny
				6/23	72–85	73–43	Partly cloudy
				6/24	61–83	63–39	Sunny
8	0.5	No. 5	A	6/12	84–86	59–53	Cloudy, showers
				6/15	56–75	50–21	Sunny, windy
				6/16	57–75	47–24	Cloudy, showers
				6/17	57–61	59–38	Cloudy, showers
9	0.7	No. 5	E	6/25	70–88	81–50	Partly cloudy
				6/26	73–96	78–47	Sunny
				6/29	80–94	67–43	Sunny

etitions annually for three years, then at three-year intervals.

11. Annual condition surveys of conventional-type control sections, including corner breaks, spalls, cracked areas, end failures, and restraint cracks.

OBSERVATIONS AND TEST RESULTS DURING EARLY LIFE OF PAVEMENT

The series of field observations made on this project about September 15, 1960, gives a record of approximately the first 15 months life of the pavement. At that time, however, the facility had not yet been subjected to traffic for a full year, as it was opened to traffic on October 28, 1959. The following summarizes the results of these observations.

Crack Spacing, Patterns, and Widths

In general, crack formation occurred about as expected and was quite similar to that of other projects in this region. Table 3 gives an aver-

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CRACK SPACING IN CENTRAL 500 FEET

Curing Conditions					Cracks in Central 500 Ft	
Period (1959)	Air Temp. (°F)	Humidity (%)	Weather		Approx. Age (mo)	Average Spacing (ft)
			Days	Type		
7/8–7/20	60–90	95–32	3	Sunny	3	8.62
			5	Cloudy	6	7.25
			5	Cloudy, showers	9	7.25
					12	7.25
					15	7.14
7/3–7/13	60–90	95–32	8	Sunny	3	8.33
			2	Ptly. cloudy, showers	6	6.58
			1	Partly cloudy	9	6.33
					12	6.33
					15	6.17
7/1–7/7	65–94	90–32	6	Sunny	3	8.47
			1	Ptly. cloudy, showers	6	7.69
					9	7 46
					12	7 46
					15	7.35
6/9–6/15	56–97	95–21	3	Sunny	3	6.25
			2	Partly cloudy	6	5.15
			1	Sunny, windy	9	5.00
					12	5.00
					15	5.00
6/5–6/12	65–97	95–30	4	Sunny	3	7.94
			2	Cloudy	6	7.35
			1	Cloudy, showers	9	7.35
					12	7.25
					15	7.25
6/3–6/9	59–95	89–30	3	Sunny	3	6.10
			2	Cloudy, showers	6	4.67
			1	Partly cloudy	9	4.20
					12	3 97
					15	3.85
6/19–6/29	60–96	81–33	8	Sunny	3	7.35
			3	Cloudy	6	5.95
			1	Cloudy, showers	9	5.75
					12	5.21
					15	5.15
6/12–6/22	56–90	95–21	5	Cloudy, showers	3	8.77
			2	Cloudy	6	6.41
			1	Sunny, windy	9	6.10
					12	5.90
					15	5.78
6/25–7/2	65–100	82–36	6	Sunny	3	5.68
			1	Partly cloudy	6	4.67
			1	Sunny, showers	9	4.13
					12	4.00
					15	3.93

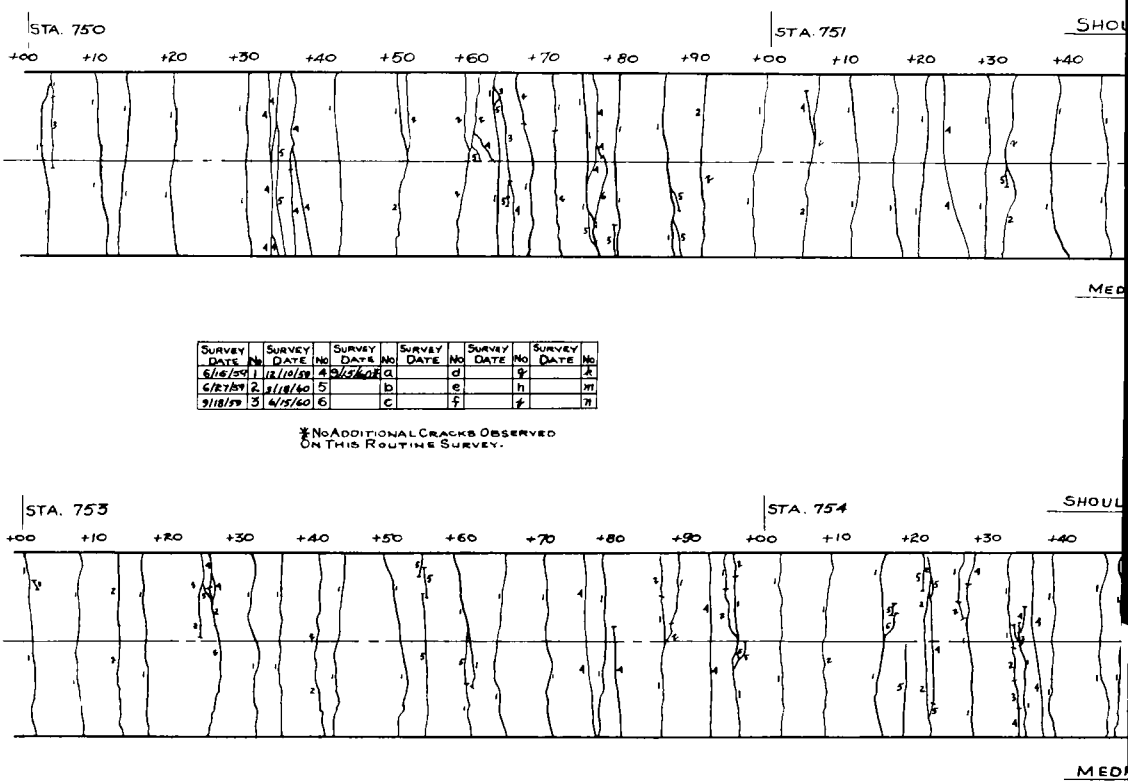
age crack spacing for each of the nine sections, together with other pertinent data. Although called an "average" spacing, the term is somewhat of a misnomer, as there is no regularity in the crack spacing. For the purpose of this report a crack is considered as a single one that extends at least three-fourths of the transverse distance between pavement edges. Also counted is the occurrence of two closely spaced transverse cracks which join to extend entirely across the pavement. Table 3 indicates the increase in crack frequency during the first 15 months at three-month intervals. These readings seem to indicate that the crack frequency is stabilizing, but the observations have not been for a sufficiently long period to so report definitely.

A detailed record was kept of the build-up of the crack pattern in the central restrained portion of each test section. After each crack survey, a carefully plotted layout of each

of the nine sections was made. In all cases an initial survey was made immediately after removal of the curing paper, a second one about two weeks later, a third in mid-September 1959, and at three-month intervals until mid-September 1960, after which annual surveys will be made. Because it would be quite bulky and hardly necessary at this time to present the details of all seven surveys for each of the nine test sections, the record for section 4 (Fig. 11) has been selected as generally representative of the manner of crack formation on the project.

U. S. Weather Bureau records will be assembled, and some details of these data will be included in later reports. Conditions prevailing during this early life of the pavement were not extreme. Lowest air temperatures observed during the winter months when gage readings, etc., were being taken were between 15 and 20 F. Snow was not excessive during the first winter, a total of about 48 in. falling in the area. Individual falls ranged from about 3 in. to about 15 in. Snow removal operations were carried on during nine snowfalls.

In the use of continuously-reinforced concrete pavement, it is hoped that the transverse cracks which develop will be held to such small widths by the reinforcing steel that infiltration of surface water and fines will not occur, and that significant spalling at the surface will be prevented. It was previously noted that a close check would be kept of crack widths in several regions of each test section. It is reasonable to expect that



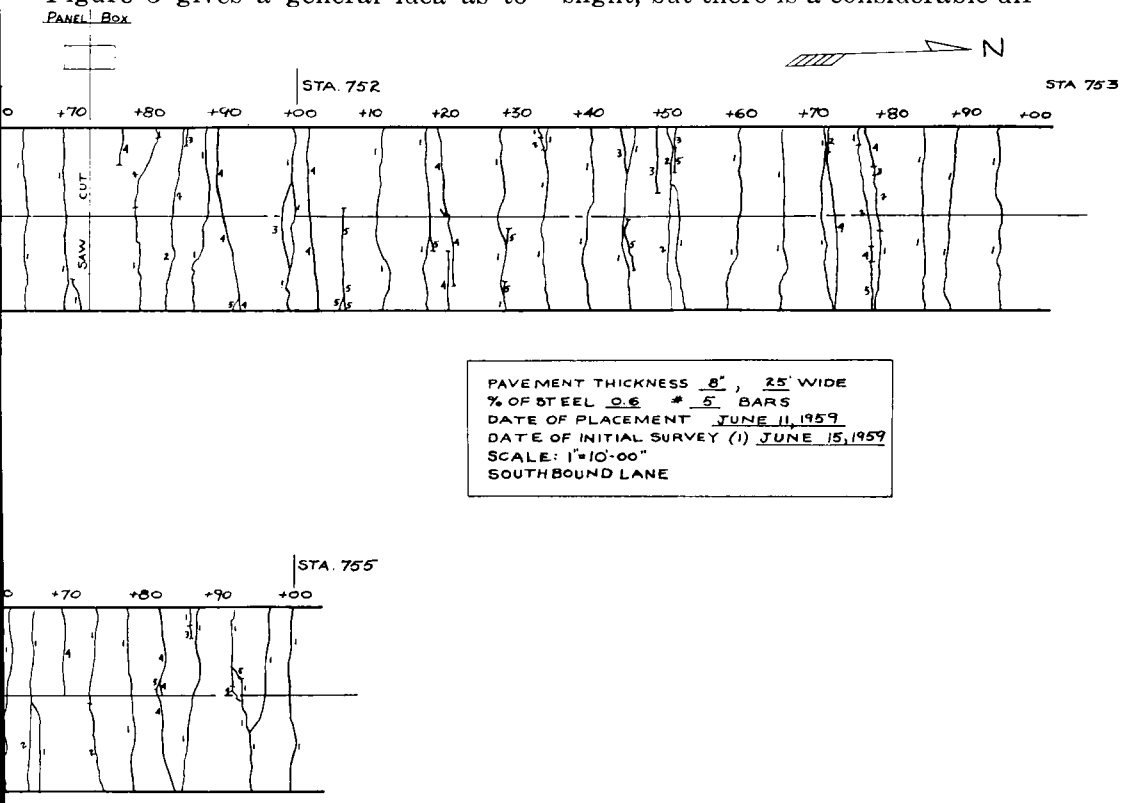
Survey Date	No	Survey Date	No	Survey Date	No	Survey Date	No	Survey Date	No	Survey Date	No
6/16/59	1	12/10/59	4	9/15/60*	a		d		g		j
6/27/59	2	3/18/60	5		b		e		h		m
9/18/59	3	4/15/60	6		c		f		i		n

Figure 11. Intensive pavement crack survey data for 5

there will be some differences in observed crack widths in (a) areas close to the free ends of long continuous sections, (b) the central restrained portion of the sections, and (c) near transverse construction joints where the continuity of paving was interrupted at the end of a day's operation or due to other causes. A record of width of four cracks in each of these regions of each test section has been obtained at approximately three-month intervals. The data are given in Table 4. Inasmuch as test sections 4, 5 and 6 were built as one long continuous stretch of pavement without transverse terminal joints, no "end area" crack widths are given for section 5. Stations indicated are only approximate, but Figure 3 gives a general idea as to their locations. Figure 12 shows a typical crack.

Measurements of crack widths were obtained for a total of 100 cracks. Of these, one opened more than 0.100 in. during the winter, another more than 0.060 in., 12 others more than 0.030 in., while 65 had an opening of less than 0.020 in.

Table 4 shows that the average width of observed cracks in the restrained portions of the 0.5 percent reinforcing opened to 0.0277 in. in December 1959. The average crack width in the 0.6 percent sections in the central restrained area was 0.0143 in. at the same period, while the average crack width for the 0.7 percent sections was 0.0122 in. The difference in average crack width between the 0.6 and 0.7 percent sections is slight, but there is a considerable dif-



Sta. 750+00 to Sta. 755+00) in central region of section 4.

TABLE 4

OBSERVED WIDTHS OF FOUR CRACKS IN EACH OF THREE AREAS OF EACH TEST SECTION

Sect. No.	Crack Width (in.)																
	Central 500-Ft Area						End Area 500 Ft from Free End						Following Transverse Constr. Joint					
	Sta.	Sept. 1959	Dec. 1959	Mar. 1960	June 1960	Sept. 1960	Sta.	Sept. 1959	Dec. 1959	Mar. 1960	June 1960	Sept. 1960	Sta.	Sept. 1959	Dec. 1959	Mar. 1960	June 1960	Sept. 1960
1	627+10	0.0152	0.0198	0.0173	0.0131	0.0115	632+99	0.0159	0.0191	0.0146	0.0117	0.0145	624+11	0.0190	0.0279	0.0213	0.0180	0.0204
	+30	0.0171	0.0233	0.0183	0.0141	0.0168	633+07	0.0139	0.0173	0.0130	0.0088	0.0142	+20	0.0296	0.0662	0.0436	0.0373	0.0410
	+49	0.0313	0.0471	0.0330	0.0108	0.0132	+18	0.0153	0.0206	0.0169	0.0216	0.0165	+39	0.0175	0.0252	0.0210	0.0166	0.0190
	+64	0.0133	0.0178	0.0129	0.0096	0.0137	+35	0.0107	0.0196	0.0105	0.0081	0.0100	+77	0.0162	0.0265	0.0185	0.0151	0.0190
2	666+35	0.0190	0.0278	0.0199	0.0153	0.0177	678+99	0.0192	0.0271	0.0211	0.0166	0.0209	660+06	0.0137	0.0176	0.0161	0.0094	0.0194
	+48	0.0279	0.0383	0.0297	0.0199	0.0233	679+08	0.0130	0.0177	0.0141	0.0106	0.0129	+19	0.0110	0.0118	0.0109	0.0052	0.0140
	+61	0.0136	0.0145	0.0117	0.0066	0.0146	+15	0.0191	0.0308	0.0249	0.0196	0.0227	+30	0.0122	0.0273	0.0129	0.0053	0.0095
	+98	0.0212	0.0300	0.0238	0.0207	0.0265	+21	0.0233	0.0270	0.0226	0.0200	0.0217	+38	0.0039	0.0079	0.0078	0.0050	0.0087
3	693+48	0.0085	0.0344	0.0174	0.0124	0.0164	702+18	0.0145	0.0389	0.0250	0.0140	0.0170	699+38	0.0077	0.0123	0.0119	0.0102	0.0108
	+66	0.0085	0.0252	0.0175	0.0040	0.0015	+39	0.0129	0.0259	0.0224	0.0166	0.0203	+41	0.0072	0.0339	0.0236	0.0062	0.0045
	+82	0.0401	0.1025	0.0619	0.0195	0.0194	+50	0.0161	0.0442	0.0341	0.0183	0.0144	+43	0.0128	0.0346	0.0297	0.0159	0.0205
	694+08	0.0054	0.0198	0.0118	0.0011	0.0007	+60	0.0110	0.0290	0.0236	0.0098	0.0087	+80	0.0162	0.0363	0.0320	0.0219	0.0246
4	749+01	0.0188	0.0185	0.0190	0.0165	0.0154	739+01	0.0062	0.0075	0.0070	0.0060	0.0073	755+51	0.0108	0.0123	0.0142	0.0106	0.0110
	+06	0.0092	0.0122	0.0124	0.0080	0.0081	+21	0.0079	0.0117	0.0130	0.0083	0.0106	+57	0.0087	0.0088	0.0097	0.0081	0.0103
	+33	0.0086	0.0095	0.0097	0.0053	0.0047	+28	0.0046	0.0072	0.0072	0.0027	0.0041	+62	0.0142	0.0144	0.0146	0.0122	0.0103
	+51	0.0149	0.0168	0.0170	0.0132	0.0143	+35	0.0057	0.0087	0.0088	0.0036	0.0055	+72	0.0153	0.0216	0.0220	0.0153	0.0161
5	782+75	0.0222	0.0271	0.0293	0.0223	0.0249	No free end in Section 5						797+76	0.0117	0.0137	0.0140	0.0097	0.0100
	+82	0.0129	0.0138	0.0143	0.0090	0.0092							798+15	0.0075	0.0203	0.0206	0.0074	0.0087
	+92	0.0148	0.0156	0.0165	0.0117	0.0125							+30	0.0210	0.0233	0.0280	0.0177	0.0184
	783+04	0.0106	0.0111	0.0116	0.0077	0.0090							+42	0.0101	0.0361	0.0384	0.0135	0.0175
6	812+63	0.0094	0.0097	0.0099	0.0048	0.0053	826+27	0.0209	0.0204	0.0199	0.0130	0.0149	Same as for "end area"; for Section 6, one set of four cracks is common to "end area" and "following construction joint."					
	+93	0.0114	0.0128	0.0135	0.0092	0.0100	+37	0.0068	0.0027	0.0031	0.0003	0.0032						
	813+28	0.0145	0.0136	0.0137	0.0087	0.0094	+79	0.0156	0.0162	0.0172	0.0123	0.0123						
	+44	0.0134	0.0138	0.0139	0.0102	0.0116	827+61	0.0206	0.0198	0.0202	0.0144	0.0167						
7	808+08	0.0117	0.0171	0.0134	0.0085	0.0077	825+81	0.0076	0.0143	0.0113	0.0095	0.0092	799+94	0.0131	0.0195	0.0232	0.0023	0.0009
	+30	0.0041	0.0076	0.0074	0.0051	0.0040	+93	0.0090	0.0170	0.0116	0.0078	0.0081	800+05	0.0126	0.0150	0.0151	0.0128	0.0147
	+44	0.0086	0.0147	0.0120	0.0098	0.0087	826+31	0.0077	0.0098	0.0081	0.0059	0.0070	+13	0.0151	0.0216	0.0222	0.0182	0.0200
	+59	0.0080	0.0179	0.0132	0.0077	0.0048	+76	0.0091	0.0137	0.0105	0.0075	0.0068	+23	0.0118	0.0181	0.0161	0.0130	0.0167
8	747+06	0.0084	0.0189	0.0148	0.0093	0.0117	739+89	0.0089	0.0135	0.0111	0.0088	0.0113	760+27	0.0009	0.0010	0.0010	0.0031	0.0058
	+80	0.0117	0.0219	0.0195	0.0156	0.0182	740+59	0.0086	0.0115	0.0113	0.0092	0.0115	+47	0.0015	0.0016	0.0016	0.0016	0.0019
	748+87	0.0168	0.0330	0.0297	0.0223	0.0222	+70	0.0113	0.0155	0.0138	0.0100	0.0119	+53	0.0005	0.0013	0.0013	0.0012	0.0000
	749+06	0.0080	0.0115	0.0109	0.0087	0.0079	741+38	0.0095	0.0144	0.0137	0.0129	0.0116	+56	0.0001	0.0007	0.0009	0.0004	0.0011
9	710+96	0.0129	0.0200	0.0156	0.0126	0.0131	705+53	0.0046	0.0091	0.0083	0.0047	0.0055	723+70	0.0058	0.0090	0.0072	0.0028	0.0057
	711+40	0.0057	0.0118	0.0086	0.0047	0.0028	+86	0.0045	0.0086	0.0072	0.0051	0.0048	+83	0.0052	0.0068	0.0053	0.0034	0.0057
	+84	0.0035	0.0083	0.0046	0.0014	0.0034	706+35	0.0057	0.0106	0.0062	0.0033	0.0066	724+10	0.0079	0.0116	0.0082	0.0044	0.0061
	712+16	0.0052	0.0069	0.0055	0.0035	0.0044	+54	0.0056	0.0092	0.0062	0.0025	0.0054	+27	0.0165	0.0283	0.0172	0.0100	0.1310
Average:																		
0.5% sections		—	0.0277	—	—	0.0145												
0.6% sections		—	0.0143	—	—	0.0085												
0.7% sections		—	0.0122	—	—	0.0075												

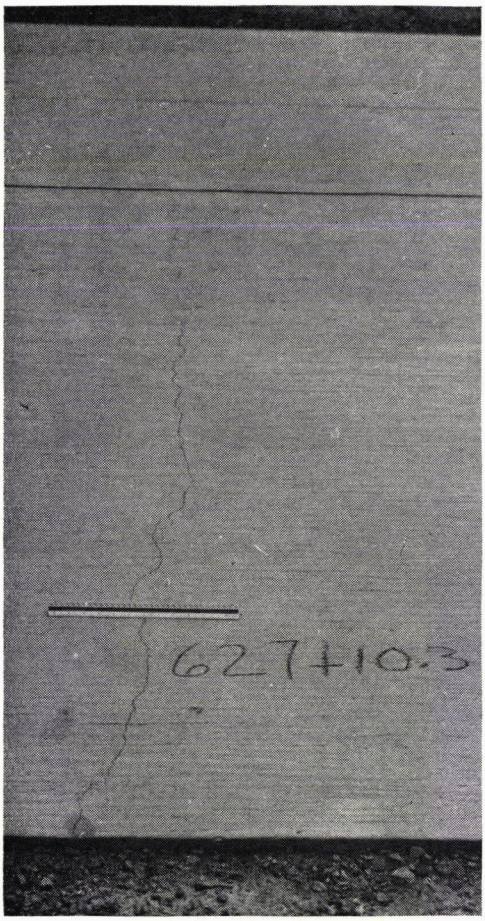


Figure 12. Typical crack as observed.

ference in the crack widths observed for the 0.5 and 0.6 percent sections. This might indicate that some of the main reinforcing steel in the 0.5 percent sections has been strained beyond its elastic limit.

Absolute End Movements and Terminal Joint Movements

Most experimental continuously-reinforced concrete pavement projects have provided for observation and study of free end movements and transverse terminal joint movements. Some of the transverse terminal joints so far used in this type of construction are very costly. If very few are necessary for a project the equivalent square yard cost will not be significant, but if there are many structures with decks at grade, many terminal joints will accordingly be necessary and equivalent square yard costs could become prohibitive. Therefore, careful observation of end movements of continuous sections, and total longitudinal movements of transverse terminal joints, becomes of utmost importance. If total end movements are only nominal over a long period of years, and if a relatively simple joint functions as well as one of more costly and complex design, the definite establishment of these facts should eventually redound to the economic advantage of this type of paving. The several types of terminal joints have previously been pictured and described. Tables 5 and 6 present the data pertaining to end movements of continuously-reinforced sections, and total movements across the various types of transverse terminal joints.

Pavement Conditions in Vicinity of Transverse Construction Joints

Transverse construction joints form zones of discontinuity in the attained concrete strength either side of the joint. Previous literature has indicated that such areas might give a poor performance compared to the remainder of the paving. It has been recorded that crack frequency is usually greater on the last-poured side of the joint. Some thought has been given to the use of a retarder in the mix for the last 100 ft or so on the first-poured side. It has also been suggested that the curing method might have some influence on the pavement performance in the area.

As previously mentioned, it was decided to use four different methods of curing at the transverse construction joints of this project. Elapsed time between first- and second-poured

TABLE 5
ABSOLUTE END MOVEMENT OF CONTINUOUSLY-REINFORCED SECTIONS

Sect. No.	Corner	Movement[a] from Initial Reading[b] (1/64 in.)							
		North End				South End			
		Sept. 1959	Dec. 1959	Mar. 1960	June 1960	Sept. 1959	Dec. 1959	Mar. 1960	June 1960
1	Shoulder	+2	−20	−4	+14	−7	−32	−30	0
	Median	−2	−10	−2	+10	−9	−37	−31	−4
2	Shoulder	−7	−27	−29	+6	+14	−20	−36	+24
	Median	−4	−17	−23	+1	+13	−16	−38	+24
3	Shoulder	−18	−49	−48	−5	−8	−18	−16	+8
	Median	−11	−38	−34	0	−8	−20	−21	+6
4	Shoulder	—[c]	—[c]	—[c]	—[c]	+4	−17	−4	+11
	Median	—[c]	—[c]	—[c]	—[c]	−2	−21	−14	+6
5	—	—	—	—	—	—	—	—	—
6	Shoulder	−4	−21	−18	+8	—[c]	—[c]	—[c]	—[c]
	Median	−3	−17	−14	+9	—[c]	—[c]	—[c]	—[c]
7	Shoulder	−11	−36	−32	−4	−4	−22	−24	+12
	Median	−17	−31	−26	−4	−5	−25	−24	+10
8	Shoulder	+3	−17	−6	+29	+14	−4	−9	+26
	Median	−2	−27	−13	+22	+2	−14	−19	+15
9	Shoulder	−5	−16	−14	+16	+5	−17	−21	+18
	Median	−4	−14	−13	+23	+9	−11	−12	+25

[a] + indicates lengthening, − indicates shortening; see Figure 3 for details of location.
[b] At age one to two weeks.
[c] Not free; continuous with section 5.
[d] Neither end free; continuous with sections 4 and 6.

sides ranged from overnight to over a weekend.

Detailed crack patterns for a distance of 75 ft either side of the construction joints were plotted during the early life of the pavement, and all such areas have been kept under close observation. A wide crack developed on the first poured side of one joint. It is not felt that this crack was caused primarily by shrinkage or temperature variations. It was repaired and is described later herein.

A detailed inspection was made of each joint in early October 1960. All areas in the vicinity of the joints appear to be in very good condition. First cracks on the second-poured side are much closer than first cracks on the first-poured side of a joint. On the second-poured side first cracks averaged about 4 ft from the joint, although in two instances a first crack appeared about 1 ft distant. On the first-poured side the average distance from joint to first crack was about 9 ft.

At this time it is not apparent that the curing method influenced the performance of the pavement in the vicinity of transverse construction joints. It is intended to keep these areas under observation, and to describe the results in detail in a subsequent report.

Transverse Terminal Joints

The three types of transverse terminal joints used on this project have previously been mentioned and pictured.

After nearly one year's service under traffic the terminal joint performance has been generally satisfactory. The pavement on either side of the joints is likewise in satisfactory condition, and apparently has not been adversely affected by the joints.

The rubber joint, which is well liked by maintenance personnel, has performed very well, although several were slightly damaged by snow plows during winter operations. It apparently will be necessary to carefully mark their locations so that blades can be raised to avoid them.

The steel-plate joint has also been satisfactory. Some "banging" occurs when truck traffic passes. A moderate amount of re-welding will be necessary for these joints.

TABLE 6

TOTAL MOVEMENT ACROSS TRANSVERSE TERMINAL JOINTS

Roadway Location	Pavement Abutting Terminal Joints	Type of Joint	Corner	Total Movement[a] from Initial Reading[b] (1/64 in.) Sept. 1959	Dec. 1959	Mar. 1960	June 1960
		(a) Southbound Roadway					
South end	Two sections of 9-in. jointed and existing 9-in. pavement						
		Rubber	Shoulder	+5	+36	+34	−8
			Median	+9	+34	+31	−10
	Section 1						
		Fabr. steel	Shoulder	+2	+20	+7	−18
			Median	+2	+20	+9	−17
	Existing 9-in. and two sections of 9-in. jointed pavement						
		Bituminous	Shoulder	+10	+37	+30	−12
			Median	+11	+38	+30	−11
	Section 2						
		Rubber	Shoulder	+15	+42	+38	+6
			Median	+13	+39	+35	+3
	Structure approach						
	Structure approach						
		Bituminous	Shoulder	+11	+33	+28	−1
			Median	+10	+32	+28	−1
	Section 3						
		Rubber	Shoulder	+2	+17	+13	−14
			Median	0	+24	+19	−9
	Control section; 9-in. jointed pvt.						
	Structure approach						
		Fabr. steel	Shoulder	+8	+26	+23	−6
			Median	+9	+28	+21	−7
	Section 4						
	Section 6						
		Rubber	Shoulder	+11	+23	+18	−14
			Median	+16	+32	+23	−11
North end	400-ft section of 9-in. jointed pavement						
		(b) Northbound Roadway					
North end	400-ft section of 9-in. jointed pavement						
		Bituminous	Shoulder	+12	+40	+36	−6
			Median	+10	+40	+36	−6
	Section 7						
		Rubber	Shoulder	−1	+28	+18	−29
			Median	−3	+24	+12	−32
	Control section; 9-in. jointed pvt.						
		Fabr. steel	Shoulder	−11	+13	+5	+11
			Median	−12	+14	+8	+13
	Section 8						
		Fabr. steel	Shoulder	+4	+28	+24	−14
			Median	+4	+29	+26	−13
	Structure approach						
	Structure approach						
		Bituminous	Shoulder	+10	+28	+26	−4
			Median	+10	+30	+26	−5
	Section 9						
		Fabr. steel	Shoulder	+7	+32	+31	−7
			Median	+6	+32	+31	−7
South end	Two sections of 9-in. jointed and existing 9-in. pavement						

[a] + indicates opening of joint, − indicates closing; see Figure 3 for details of location.
[b] At age one to two weeks.

The bituminous joint has been satisfactory, but is not as popular as the others with maintenance personnel. They have experienced some difficulty in keeping the face between the steel angle and the bituminous fill well sealed.

If absolute end movements, and total movements at the various terminal joints, do not increase significantly beyond the ranges indicated in Tables 5 and 6, some simplified type of terminal joint should suffice.

Road Roughness

An initial record of pavement roughness was made on October 20, 1959, shortly before the facility was opened to traffic. These data were obtained with a Bureau of Public Roads roughometer and a Maryland roughometer built from the same plans and specifications.

The results of this roughness survey are given in Table 7. Comparisons between performance of conventional and continuous sections as indicated by roughness readings will be fully reported in subsequent years.

GENERAL APPEARANCE AND FIRST-YEAR PERFORMANCE

A close general inspection of this entire pavement in late summer and early fall of 1960 leads to the following observations:

1. Most (probably 99 percent) of the continuously-reinforced pavement is in very good condition and gives an excellent appearance.
2. The conventional control sections are in an equally excellent condition.
3. Eight patched areas appear in the continuously-reinforced sections. These are discussed in the following.

The patches were made in order to correct unusually wide cracks which developed at certain locations and gave indications that bond and continuous pavement action had been broken. One such area developed before opening to traffic, and was re-

TABLE 7
ROAD ROUGHNESS SURVEY DATA

Roadway Portion	Long. Reinf. Steel[a] (%)	Relative Roughness (units/mi)			
		Outside Lane[b]		Inside Lane[c]	
		BPR Indicator	Md. SRC Indicator	BPR Indicator	Md. SRC Indicator
(*a*) Southbound Roadway					
Test section 1	0.5	100	102	105	106
Test section 2	0.5	94	91	95	92
Bunker Hill Br.		149	150	160	140
Test section 3	0.5	89	85	88	86
Control section[d]		91	90	90	88
Gunpowder Falls Br.		136	128	141	136
Test section 4	0.6	107	106	118	119
Test section 5	0.5	104	98	113	113
Test section 6	0.7	105	100	110	106
(*b*) Northbound Roadway					
Test section 7	0.6	93	94	94	93
Control section[d]		96	91	105	102
Test section 8	0.5	110	110	118	118
Gunpowder Falls Br.		121	116	125	130
Test section 9	0.7	102	97	91	91

[a] Noted for continuous sections only.
[b] Shoulder lane.
[c] Median lane.
[d] Conventional jointed pavement.

paired on a force account basis by the contractor. The remaining faults developed during the winter period, and by early spring it was decided that repairs should be made. It is possible that these cracked areas may not have disintegrated further. This has been the case in some continuously-reinforced pavements. However, it was felt that the original conditions of continuity should be restored.

Several of the cracks were near the low points of long grades, and on the low side of superelevated curves. It is possible that these low areas collected excess moisture that resulted in a delay of satisfactory bond strength development. At least one seriously cracked area developed where a breakdown of the paving equipment occurred. The first lift of concrete was exposed for a long period before the second was placed. Another serious crack occurred on the first-poured side of a transverse construction joint. It is believed that the additional short bars were disturbed while the concrete was still "green" and that loss of bond or actual damage to the concrete occurred.

All excessive cracks save two developed at laps in the bar mats. One of those which did not develop at splice laps is the crack described in the preceding paragraph. Saw cuts were made each side of the faults, spalled or deteriorated concrete was removed, the subbase was re-tamped, splices were welded; and the area was repaved with concrete similar to the original job mix.

During repairs it was found that all splices had been perfectly made. However, there were some indications of lack of bond between the first and second lifts of pavement, and also some lack of bond between the reinforcing steel bars and the concrete. This concrete pavement was not vibrated except along the forms, and the reinforcing bar mats were placed with the transverse bars down.

OBSERVATIONS ON CONSTRUCTION AND DESIGN

Observations during the construction and early life of this project indicate that:

1. Vibration is apparently necessary to promote bonding between the two pavement lifts, and to insure full flow of low-slump concrete around the reinforcing steel.

2. Staggering the ends of reinforcing steel mats is desirable, so that any late developing areas of bond strength will not be concentrated in one line across the pavement.

3. Better encasement of the steel with concrete can be accomplished if the longitudinal steel of the mats is placed down.

4. A splice of 20 diameters is hardly sufficient to prevent all failures at laps; 25 or even 30 diameters would be much better if bond alone is relied on at laps.

5. A simple mechanical anchorage between adjacent mats would be desirable. One way of accomplishing this might be to weld end transverse members securely to their longitudinal members, then bring these end transverse members into bearing, so that this bearing value would be in addition to bond strength developed on the longitudinal bars.

CONCLUSIONS

No attempt is made to draw final conclusions from the performance of a pavement subjected to traffic use for only one year. Certain trends are apparent, however, as follows:

1. An inverse proportion exists between crack spacing and percentage of steel reinforcing.

2. An inverse proportion is also apparent between crack spacing and bond area for the 0.5 percent steel used in this investigation.

3. In the restrained central portions of the test sections the average width of observed cracks is notably greater in the 0.5 percent sections than in the 0.6 and 0.7 percent sections. Crack widths are somewhat greater in the 0.6 than in the 0.7 percent sections, but the difference is slight as compared to the 0.5 to 0.6 percent differential.

4. The several methods of curing near transverse construction joints do not appear to have influenced crack widths or crack patterns in these areas.

5. The maximum total movements so far recorded at the various types of terminal joints are: rubber, 7/8 in.; steel plate, 5/8 in.; bituminous, 3/4 in. Maximum absolute end movements of the continuous sections abutting these particular terminal joints were: rubber, 5/8 in.; steel plate, 1/2 in.; bituminous, 5/8 in.

6. The areas of open cracks which were later repaired could not be associated with any particular percentage of reinforcing or size of bars.

Based entirely on the first year's history of this project, it is seen that rather extensive maintenance has been necessary for certain specific locations of the continuous pavement. Practically no maintenance has been necessary on the conventional pavement. It is believed a better picture of relative maintenance costs will develop as the pavement attains an age of some ten years. It will also be interesting to observe an adjoining project where the concrete is being vibrated, mats are staggered, and splices are 25 diameters long.

II. First-Year Strain Observations

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• CONTINUOUSLY-REINFORCED concrete pavement is constructed in unjointed lengths generally ranging upwards from 1,000 ft. Because of the restraints offered by the subgrade friction in the end portions, there are no gross movements of expansion and contraction within the central portion of such lengths. The normal tendencies to expand or contract are prevented and associated compressive or tensile stresses are induced in the pavement because of the restraint. The compressive stresses that may be developed are small, relative to the strength of the concrete, and do not cause any deterioration of the pavement. On the other hand, the tensile stresses will develop to sufficient magnitude to cause cracking of the concrete and the reinforcement serves the function of preventing cracks so formed from opening to any detrimental extent.

The longitudinal reinforcement must be of sufficient amount and strength to serve this purpose. Continuously-reinforced pavement has been constructed with longitudinal reinforcement ranging in amounts from 0.3 to 1.8 percent of the cross-sectional area of the concrete. High yield point reinforcement is preferred over structural grade to provide a greater range of strain over which there will be elastic spring recovery to close the cracks after they have been open in periods of low temperatures.

The studies described in this first progress report are part of the Maryland State Roads Commission's in-

TABLE 8
DETAILS OF INSTRUMENTED SITES

(a) Design Features

Site	Reinforcing Bars				Continuous Section			Grade at Instr. Panel (%)	Alignment
	(%)	Size	No.	Gaged[a]	Length (ft)	South (ft)	North (ft)		
1	0.506	No. 5	38	3, 8, 12, 17	1792	825	967	−2.6	Tangent
2	0.503	No. 4	59	5, 13, 18, 26	3777	1768	2009	0.0	Inside 2°36′
3	0.499	No. 6	26	2, 5, 8, 11	2018	839	1179	+1.3	Inside 2°05′
4	0.601	No. 5	47	4, 10, 15, 20	3748	1770	1978	−4.5	Tangent
5	0.499	No. 5	39	3, 8, 12, 17	3000	1502	1498	−1.0	Tangent
6	0.703	No. 5	55	4, 11, 16, 24	3000	1653	1347	+4.0	Outside 2°00′

(b) Construction Features

Site	Inst. Mat Placed		Temp. (°F)		Joint Sawed		Induced Crack Formed		
							Before		Hours After Placing
	Date (1959)	Time	Air	Conc.	Time, P.M.	Age (hr)	Time	Date	
1	8 July	3:50 P.M.	88	92	10:00	6:10	8 A.M.	July 10	26 to 40
2	6 July	3:00 P.M.	78	79	9:40	6:40	8 A.M.	July 8	31 to 39
3	2 July	10:30 A.M.	82	86	4:00	5:30	6 A.M.	July 3	14 to 20
4	11 June	10:00 A.M.	85	79	6:00	8:00	8 A.M.	June 12	12 to 23
5	8 June	3:30 P.M.	89	89	10:00	6:30	8 A.M.	June 10	30 to 39
6	4 June	2:20 P.M.	80	76	11:00	5:25	6 A.M.	June 6	32 to 40

[a] From shoulder.

a vertical direction. Furthermore, the strain gage readings revealed that the crack must have developed as desired. After the concrete had cured and dried, the saw cut was sealed with a protective bituminous compound.

The reinforcement is deformed bar hard grade billet steel. Yield points are 71,300, 67,800, and 55,000 psi for the No. 4, 5, and 6 bar sizes, respectively. Bar mats, made up of 16-ft long reinforcing bars, and 6 ft wide, were fabricated for this job. The mats are held together by seven transverse No. 3 bars, to which the longitudinal steel is attached. Four of these mats cover the width of the pavement. Bar mats were placed with an overlap at each end to provide continuity through the entire pavement length. The overlap is 12 in. for No. 4, 13 in. for No. 5, and 15 in. for No. 6 bar sizes. Transverse tie bars of No. 5 size, structural grade reinforcing, 4 ft long and spaced at 4-ft intervals, span the longitudinal joint between the lanes of the pavement. All of the reinforcing steel is located at the mid-depth of the pavement.

Instrumented Mats

The strain- and temperature-measuring instruments at each test panel consist of the following:

1. Temperature-compensating test gages.
2. Strain gages on reinforcing steel.
3. Concrete strainometers.
4. Temperature-measuring coils in slab, subbase, and subgrade.
5. Whittemore strain gage lines on surface of concrete.

These instruments all are located within the length of one bar mat in the shoulder lane of the pavement near the center of each continuously-reinforced section as previously described.

The locations of the strain- and temperature-measuring instruments are shown in Figure 14. All the gage panels are similar in arrangement.

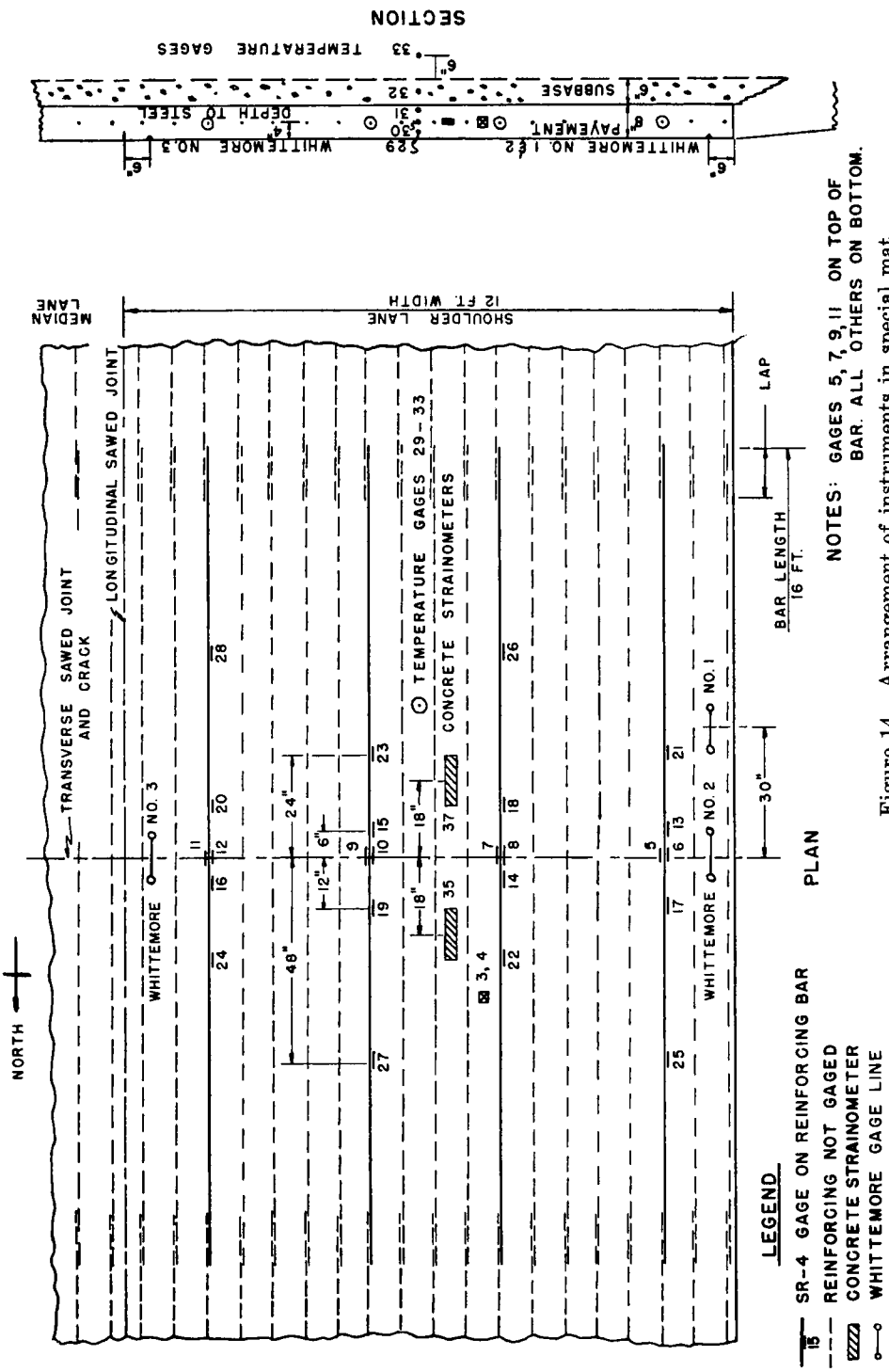

Figure 14. Arrangement of instruments in special mat.

vestigation of continuously-reinforced concrete pavement. The objective of the entire investigation is to make a thorough study of the behavior of continuously-reinforced pavement, with particular emphasis on the effects of amount and size of reinforcing steel. The site of the field investigation and the details of the test sections have been described in Part I.

In addition to observations of overall performance, instrument installations were planned to investigate the behavior in both the concrete and the reinforcing steel in the vicinity of induced cracks. Instrumented bar mats have been constructed in each of the locations indicated for the different amounts and sizes of reinforcing steel. These instrumented panels contain strain- and temperature-measuring instruments. They have been designed to determine the tensile strains in the main reinforcing steel at and near induced cracks. It is expected that data will be obtained on the bond between the steel and the concrete and on the crack width in the continuously-reinforced pavement. These observations, taken since the start of construction and to be continued for a considerable period of time, are expected to show in detail the performance of the pavement as affected by environmental changes, age, and the cumulative effects of traffic loads.

FIELD INSTALLATIONS

The field test sections that contain instruments are located in the outside, or shoulder, lane of the southbound roadway. Figure 13 shows the salient features of the roadway containing the six instrumented sections. The continuously-reinforced sections are identified as Sites 1 through 6. There is a section of conventional jointed 9-in. pavement between Sites 3 and 4.

The continuously-reinforced pavement is 8 in. thick, supported by a 6-in. layer of selected subbase material. The subbase material is crusher-run gradation of a crushed local serpentine rock. The pavement is 24 ft wide south of Gunpowder Falls Bridge and 25 ft wide north of the bridge.

The chief variable in the field experimental pavement is the amount and size of the reinforcing steel. Sites 1, 2, and 3 contain nominally 0.5 percent of steel made up of No. 5 bars in Site 1, No. 4 bars in Site 2, and No. 6 bars in Site 3. In Sites 4 through 6 all reinforcement consists of No. 5 bars. Site 4 has 0.6, Site 5 has 0.5, and Site 6 has 0.7 percent steel.

Details of the design and construction features of the individual experimental lengths in which instrumented bar mats are located are given in Table 8. As indicated, gaged bars replaced regular reinforcing bars in the shoulder lane of the pavement. Temperature-measuring coils and concrete strainometers were placed within the length of these instrumented panels.

Sections 4 through 6, north of Gunpowder Falls Bridge, are continuously reinforced for the entire length. Within this total length of 9,748 ft, the transition from one section to the next is accomplished by merely changing the bar mat and the percentage of reinforcement.

Conditions at the time of construction also are given. A saw cut 2 in. deep was made across the full pavement width to induce a crack at the location of the strain gages. It was decided that a notch made by sawing would produce a more nearly natural crack than could be obtained by means of a parting strip. During either the first or second night following placement of the concrete, a natural crack formed directly below the saw cut through the entire depth of the pavement. Observation of the edges of the pavement showed that in each case this crack ran very true in

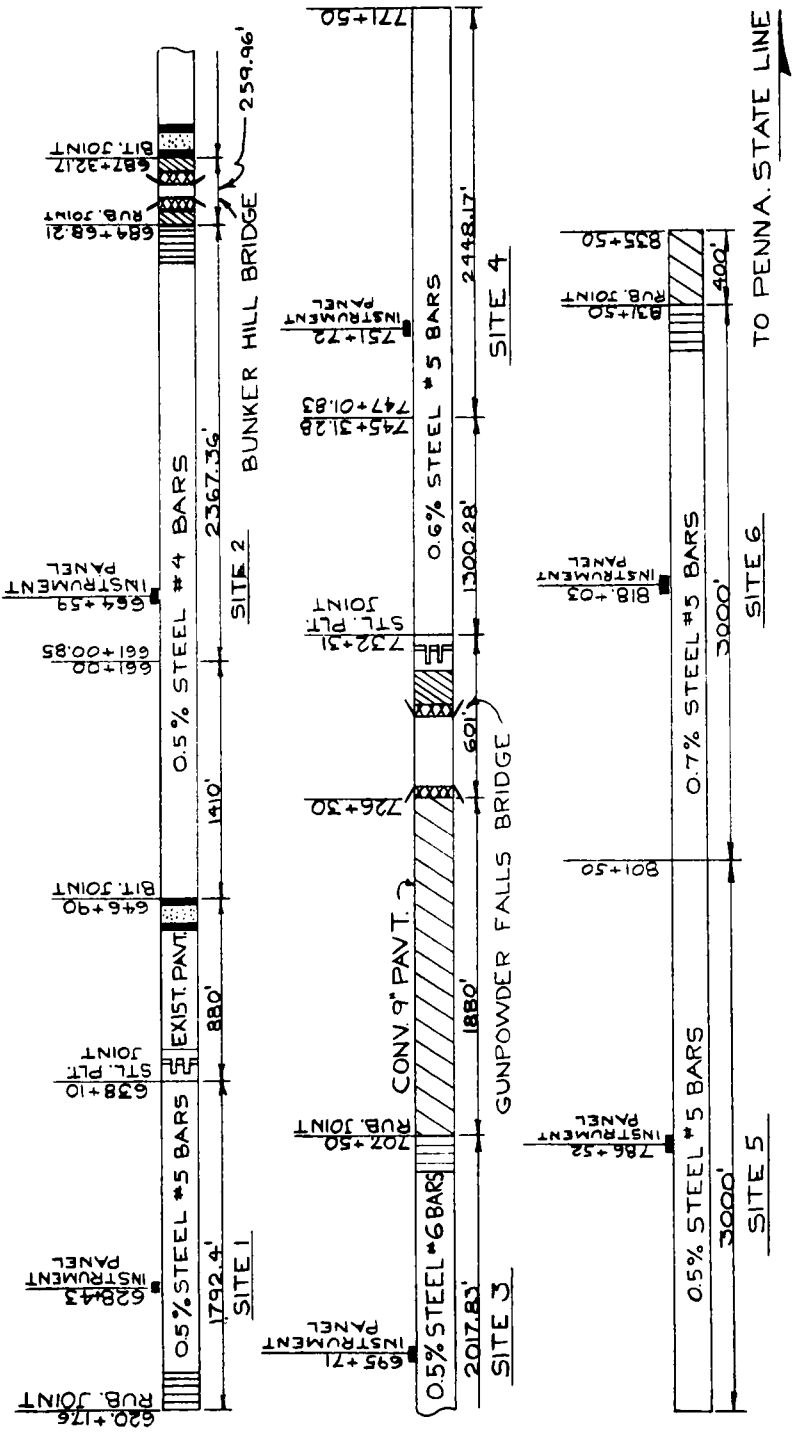

Figure 13. Experimental sections in southbound roadway of Baltimore-Harrisburg Expressway.

The gage spacing along reinforcing bars is identical in each case, but the locations of the bars differ slightly for each panel because of the different numbers of bars in the lane for the different bar sizes and steel percentages.

The temperature - compensating gages (Nos. 3 and 4) are SR-4, type AB-7, strain gages mounted on opposite sides of a short (1½-in. long) piece of reinforcing steel covered with a heavy coating of soft rubber waterproofing and protective tape. This assembly is located at mid-depth of the pavement between reinforcing bars 4 to 5 ft in from the shoulder edge. Readings from these gages are intended to provide data for temperature compensation of the other strain readings on the reinforcing bars.

Strain gages Nos. 5 through 28 on the reinforcement are the source of major data in the field investigation. These gages give strains at several locations along the bars and on four bars spaced equally across the lane width. The gages are SR-4, type AB-7, resistance wire strain gages (nominal gage length ¼ in.)

Gages Nos. 5 through 12 are located at the induced crack, which extends through the pavement depth. These gages are paired, diametrically opposite each other, on each bar. The bars were placed so that the odd-numbered gages (5, 7, 9, 11) are on top of the bar and the even-numbered ones (6, 8, 10, 12) on the bottom. Each of the four gaged bars has gages at 6, 12, 24, and 48 in. from the paired gages at the center of the bar. The bars were so placed in the mat that these gages are on the bottom of the bar and the relative positions of the gages alternate from bar to bar across the lane width. Gages 13 through 16 are 6 in. from the induced crack, Nos. 17 through 20 are at 12 in., Nos. 21 through 24 are at 24 in., and Nos. 25 through 28 are at 48 in. from the induced crack.

Two concrete strainometers, gages 35 and 37, are located near the middle of the instrumented mat and centered 18 in. to either side of the induced crack. These strainometers were developed at Lehigh University and consist essentially of an aluminum bar on which a strain gage bridge of SR-4 gages is mounted. The ends of the aluminum bar are bonded in the concrete to subject the gages to a strain equivalent to that in the 10-in. length of concrete between the ends of the strainometer.

Temperature - measuring coils (gages No. 29 through 33) are located in the pavement slab, the subbase, and 6 in. below the subbase. They are near the middle of the instrumented mat and 3 to 4 ft from the induced crack. Gage No. 29 is 1 in. below the pavement surface, No. 30 is at mid-depth of the concrete (at the reinforcing steel level), No. 31 is 1 in. above the bottom of the concrete. Gage No. 32 is at the mid-depth of the crushed-stone subbase and Gage No. 33 is in the subgrade 6 in. below the subbase.

Gage lines for measurement of surface strains by the hand-held Whittemore strain gage span the induced crack at positions 2 and 3, each located 6 in. from the outer edge and longitudinal joint, respectively, of the shoulder lane. Position 1, centered 30 in. from the induced crack and about 6 in. in from the edge of the pavement, is used for measurement of surface strains in the concrete segments between cracks. Strains on these gage lines are measured between brass plugs embedded in the concrete pavement.

Electrical lead wires from all strain gages were carried through a conduit beneath the shoulder of the road to a terminal box well off the road shoulder. The terminal box contains resistance coils for balancing the electric circuits and a plug-jack board to which connection is made with the strain indicator (Fig. 15). It is made of heavy-gage steel

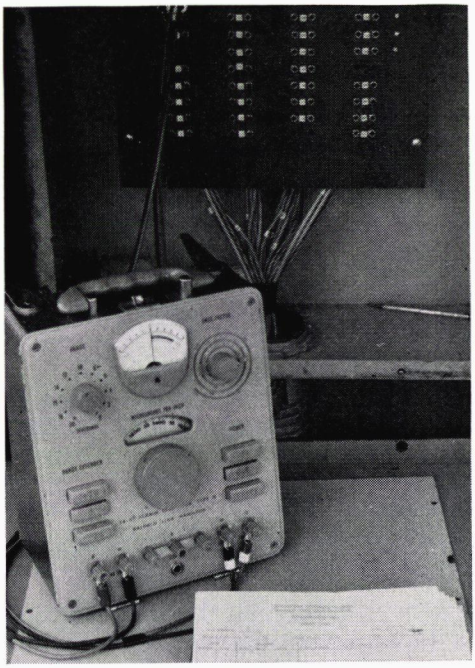

Figure 15. Terminal box for gage leads.

mounted on a concrete base and equipped with a hinged door that opens down to provide a shelf on which the instruments may be placed while taking readings. The terminal board is enclosed in an inner box within the outside box structure. The inner box is moistureproof, the covering being provided with a sponge rubber seal. Performance of the entire installation thus far has been completely satisfactory.

Preparation of the gaged reinforcing bars and other instruments, together with their installation in the roadway, was accomplished by I. J. Taylor, Instruments Associate, Lehigh University. The deformations were removed from the reinforcing steel with a high-speed grinder, and care was taken to remove only enough of the bar to provide a smooth area for attaching the small gage. The SR-4 gages were attached to the metal with an epoxy resin adhesive. A resilient sheath of vulcanized rubber covered the cemented gages and lead wire connections to complete waterproofing of the assembly. An outer cover of tape provided mechanical protection. About 2 in. of bar length are covered by the gage protection; thus, this length is not available for bond to the concrete. Temperature-measuring coils consist of a length of nickel wire formed into a small loop and covered with insulation and mechanical protection. Resistance change caused by temperature change acting on these gages was read on the SR-4 indicators, and laboratory calibration tests were made to convert such readings to temperatures.

Construction of Instrumented Panels

At an early stage in the road construction, the junction boxes were mounted well back from the shoulder on substantial concrete pedestals. A length of 3-in. flexible conduit was run from the bottom of the box and left loose until it could be placed in position to terminate under the edge of the pavement. The junction boxes were delivered with terminals installed; but, of course, lead wires from the gages could not be connected until the placement operation.

To provide time for installation of the gaged bars, the contractor obligingly extended his forms well ahead of the concreting operations so as to pass the next instrument box. One 10-ft length of form was cut with a ¼-in. vertical slot through the head to mid-depth of the form web. In the center of the web a 2-in. diameter hole was burned out to permit passage of the gage leads. As it happened, it was possible to use this same section of form for all six installations.

After the form was placed, a trench was dug for the flexible conduit, which was brought under the form

and to the surface of the subgrade some 4 to 6 in. inside the form. To install the instrumented bars, the proper reinforcing mats were selected and placed on the subbase at the position they would occupy in the pavement. Bars were removed from the standard mat and replaced by instrumented bars, which were then wired to the transverse No. 3 tie bars to hold them in position relative to the mats. The central transverse bar was removed to avoid interference with gages 5 through 12. The strain gage leads were laid out individually across the road toward the median strip and straightened out. The ends of the leads were then collected and the entire bundle of wires was pulled through the conduit and into the junction box. Each gage lead was traced individually and pulled tight, leaving just enough slack to permit handling the mats and to accommodate the 4-in. greater elevation when the mats would be placed in the concrete. At this stage the bar mats were lifted from the subbase and placed on the shoulder where they were out of the way of the paving operation, the leads being threaded through the slot and hole in the form. Final finishing on the subgrade could then be completed while the gage leads were being connected in the junction box.

An index temperature reading for the temperature-measuring coils was obtained by inserting the five temperature coils in a thermos of water, together with a laboratory thermometer. After temperature equilibrium was reached, the gages were read on the strain indicator and the water temperature noted. All subsequent temperature measurements are referred to this initialization.

The method of placing the instrumented mats in the pavement was identical with the general construction. Concrete was placed and spread in a 4-in. layer. After this layer of concrete was placed past the instrument site, the instrumented mats were positioned on the fresh concrete. A string line between marks on both forms was used to locate the center of the mat and the positions of gages 5 through 12. These form marks served later to locate and guide the saw cut over these gages. While the construction proceeded beyond the instrumented mat and the second layer of concrete was being placed up to it, the lead wires were tied to the reinforcing bars to prevent their riding across the sawed-joint location or into the upper layer of concrete. The concrete strainometers, temperature-compensating gages (Nos. 3 and 4), and temperature coil No. 30 were placed at the level of the bar mats and wired to hold them in position. A hole was made by trowel to place the temperature coil (No. 31) at 1 in. above the bottom of the fresh concrete. The coil to be located 1 in. below the top surface of the pavement (No. 29) was positioned by wiring the lead to a vertical spike. These operations were completed well in advance of the placing of the second layer of concrete. Table 8 gives construction data.

Because the instrumented mats had to be aligned with the hole in the form, it was necessary to break a standard mat length to the north of the instrumented panel, paving being from north to south. Cut lengths of bar mats were used to make the required closures. In every case the short length of closing bar mat is one full mat length removed from the instrumented panel.

The brass plugs for the Whittemore strain gage lines were placed in the fresh concrete immediately following the finishing operation. By placing these in the wet concrete, there was no need for special cementing. After the concrete hardened, 6 to 8 hr after placing, the zero readings were taken.

PROCEDURE FOR OBSERVATIONS

Readings of the strains and temperatures were taken from the time of construction to the present and are being continued at regular intervals. The zero readings were taken after the instrumented mat had been placed in the pavement and mechanical concrete finishing operations had passed over and beyond its length.

In all panels the first significant set of readings was taken about 6 hr after paving was completed, at which time the concrete was quite firmly set. These readings coincided very nearly with the sawing of the transverse joint. An additional set of readings was taken between 11 and 12 o'clock of the night after paving. The next set of readings was taken in the early morning following the day of paving. If the crack had not formed at the transverse saw cut, readings were taken during the first day and into the evening and then resumed with early morning readings the second day. In any event, readings were taken at frequent intervals until the formation of the crack at the sawed joint.

During the construction period (June and July 1959) it was feasible to take frequent readings on the panels that were then built. Following completion of construction with Panel 1, readings were taken during the summer of 1959 at approximately weekly intervals over 24-hr cycles each week. Readings over a daily cycle were taken at two- to three-week intervals through the first year.

The regular readings were taken over a 24-hr cycle to get the full effect of the daily temperature fluctuations. It was learned quite early in the investigation that the temperatures within the pavement slab lag behind the atmospheric temperatures by some 2 or 3 hr. The reading schedule, therefore, began with a reading in the afternoon, a second reading between 10 o'clock and midnight, a third reading early the following morning, and the fourth reading in the afternoon of the second day, approximately 24 hr after the first reading. Although these times do not coincide necessarily with times of extreme air temperature during the daily cycle, it is felt that they are reasonably close to the extremes of pavement temperature in this cycle.

FORMATION OF INDUCED CRACK

Development of the transverse crack at the saw cut, located over gages 5 through 12, occurred within 14 to 40 hr after placing of the concrete. At this time, other natural cracks formed in the pavement adjacent to the induced crack. Later, still other cracks appeared between the initial ones.

Data concerning the formation of the induced crack are given in Table 9. In the two panels placed in the morning (Nos. 3 and 4) the crack formed between midnight and early morning of the night following construction. In all other panels formation of the crack occurred during the second night after construction.

Examination of the strain-time curves (for both Whittemore readings and steel strains) shows that the formation of the crack was coincident with a marked decrease in slab temperature. The observed values of temperature drop are given in Table 9. The increases in reinforcement strain at the crack, and crack opening, over this same time interval are given. Crack spacing at about one-week age also is given.

There appear to be relations between the steel strains at the crack and crack opening with temperature drop and crack spacing, as shown in Figures 16 and 17. The strain in the reinforcing steel at the crack (Fig. 16) follows closely to a linear relation with temperature drop for the four panels that cracked during the second night. Crack opening does not conform as well to this linear rela-

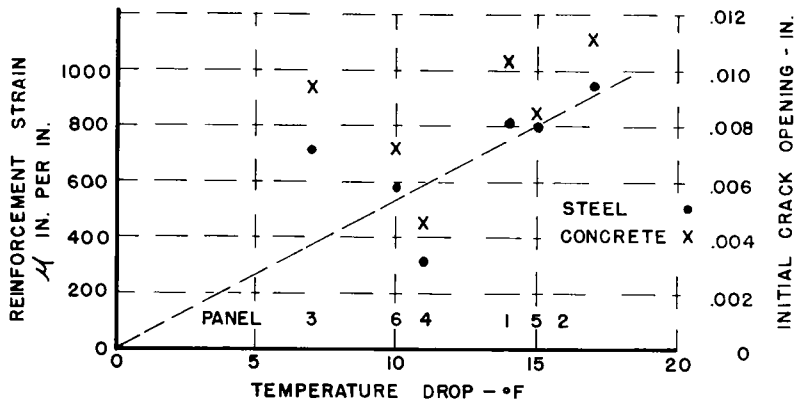

Figure 16. Strain at first opening of crack as function of drop in temperature.

tionship. Panels 3 and 4, which cracked during the first night, do not conform with the others. There is more scatter in the data when plotted as a function of crack spacing (Fig. 17), but a trend toward larger values with greater spacing is indicated.

Initial crack formation appears to be the result, at least in part, of cooling from the high temperatures caused by the heat of hydration of the cement. There is probably some drying shrinkage also taking place at this time. The concrete, then, is attempting to contract. Tensile stresses are developed because the concrete is continuous and anchored by the subgrade friction, and the strain caused by stress must be equal and opposite to the strains which would be caused by temperature drop and shrinkage if these latter strains were free to occur. When the tensile stress becomes equal to the concrete tensile strength, the crack develops.

If the concrete were unreinforced,

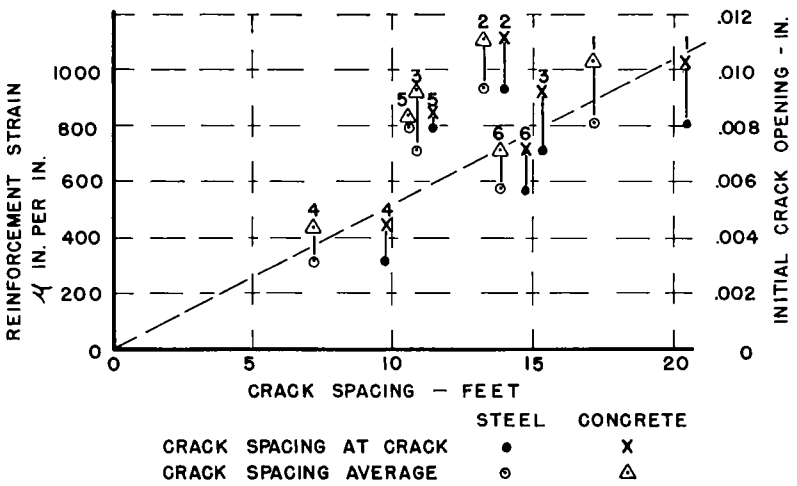

Figure 17. Strain at first opening of crack in terms of crack spacing.

TABLE 9

DATA ON DEVELOPMENT OF TRANSVERSE CRACK

Test Panel No.	Steel		Concrete Placed			Crack Formed After Concreting (hr)	Temperature (°F)			Strain Change in Steel at Crack (μ-in.)	Initial Crack Opening (in.)	Crack Spacing (ft)	
	%	Bar No.	Time	Temp. (°F) Air	Temp. (°F) Conc.		Before Crack	After Crack	Drop			Nearest[a]	Avg.[b]
1	0.5	5	3:50 P.M.	88	92	26–40	111	97	14	820	0.0104	20.8	17.2
2	0.5	4	3:00 P.M.	78	79	31–39	100	83	17	936	0.0112	14.0	13.2
3	0.5	6	10:30 A.M.	82	86	14–20	113	106	7	720	0.0094	15.3	10.0
4	0.6	5	10:00 A.M.	85	79	12–23	108	97	11	320	0.0045	9.8	7.2
5	0.5	5	3:30 P.M.	89	89	30–39	104	89	15	800	0.0085	11.5	10.6
6	0.7	5	2:20 P.M.	80	76	32–40	91	81	10	580	0.0072	14.8	13.9

[a] Total length from induced crack to adjacent cracks at approximately one-week age.
[b] Average crack spacing over 500-ft length at approximately one-week age.

there would be little restraint on the segments between the cracks. Cracks would open to such an amount that the shrinkage strain plus strain due to temperature drop over the length between cracks would nearly equal the crack width. Reinforcement across the crack prevents the crack from opening to this width, because it develops tensile forces in pulling against the anchorage effect of subgrade friction that is developed by the weight and length of the pavement. The stiffness of the reinforcement across the crack in resisting deformation causes some tensile stress to remain in the concrete and prevents the crack from opening to the full width required by the contraction effects.

CRACK OPENING OBSERVATIONS

Strain gage lines on which measurements of crack opening have been taken are located at the outer and inner edges of the shoulder lane of the pavement. These gage lines, defined by brass plugs embedded in the surface of the pavement, are located at Whittemore positions No. 2 and No. 3 in Figure 14. A third surface gage line, noted as position No. 1 in Figure 14, is located near the shoulder of the pavement with its center 30 in. from the induced crack.

All Whittemore gage readings were accompanied by standard bar observations to provide corrections for changes in instrument setting and temperature. Zero readings were taken on these gage lines when the transverse saw cut was made (see Table 8). Strains were calculated as the changes in length from the zero readings.

A sample of the data for crack openings is shown in Figure 18. Panel 4 is selected as being representative of observations near the center of the range found for all six panels. The upper diagram is a plot of slab temperatures against time of

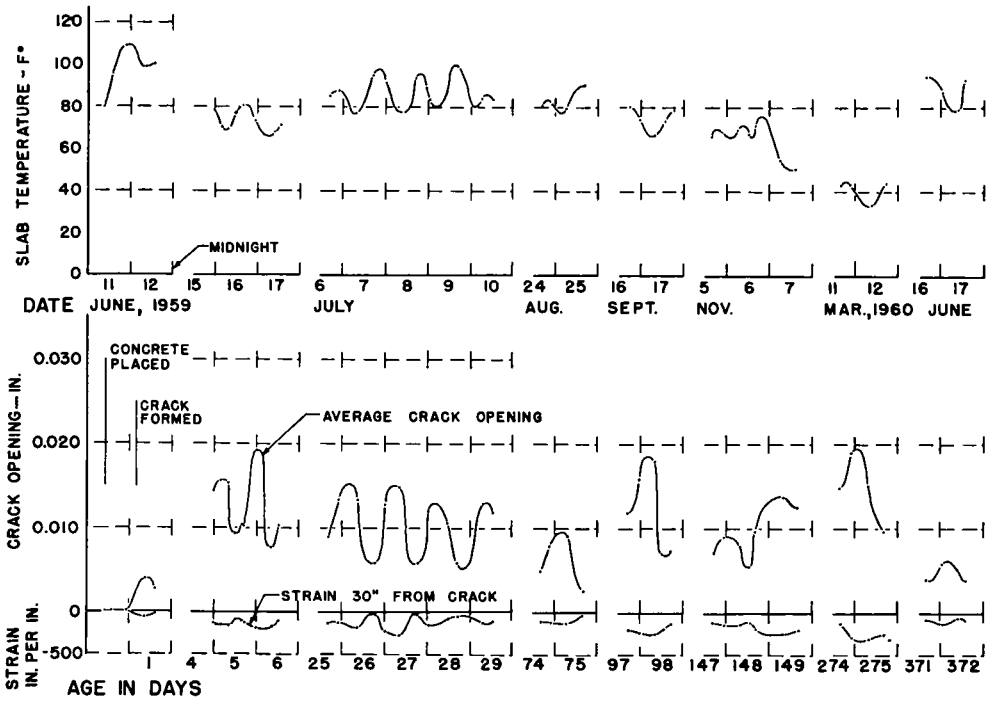

Figure 18. Crack openings and concrete surface strains, panel 4, 0.6% steel, No. 5 bars.

observation. The two curves of the lower diagram are plotted from the Whittemore gage readings. The upper curve shows the crack opening in inches. The lower curve, generally showing compressive strain, is a plot of the unit strains at position 1. Observed points are shown, together with the estimated variation between times of observation. The dates shown illustrate the more significant happenings during the first-year performance.

The slab temperature plotted is the average of the three temperature readings in the concrete (gages 29, 30 and 31). Points defining the curve of crack openings are average values of crack opening calculated from the readings at positions 2 and 3. These values on this diagram are not corrected for the effect of temperature on the concrete within the gage length.

The fact that crack opening is strongly dependent on temperature is clearly revealed by these strain-time curves. The pattern of opening and closing of the crack with the daily fluctuations in temperature is most marked. As would be expected, the crack opening increases with a drop in temperature, while the strain at position 1 away from the crack generally shows a slight decrease. The opposite changes occur with rising temperature.

There are a few exceptions to this general pattern of behavior. Such discrepancies may be due to moisture effects, or to warping caused by marked differences in temperature through the slab thickness.

Because it is difficult to make comparisons from these detailed curves, summary crack opening-time curves have been plotted in Figure 19 for panels 1, 4, and 6. The curves rep-

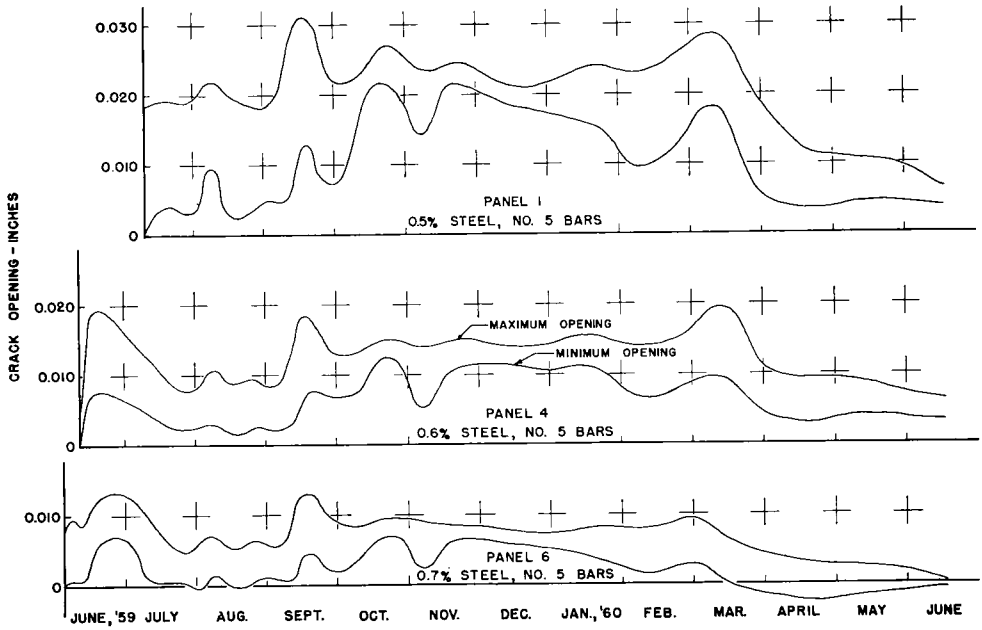

Figure 19. Variation in crack opening during first year.

resent the maximum and minimum crack openings for each of the daily observation periods. Smooth curves are drawn to show the trend between observations. Panel 1 data show the largest openings, panel 6 data are generally the smallest; panel 4 is closely representative of the trends for panels 2, 3, and 5 as well. These curves show more clearly the behavior over the entire year.

Rather consistent trends may be seen from these crack opening-time curves. Relatively large openings, in the neighborhood of 0.01 to 0.02 in., were observed during the first summer despite the fact that the slab temperatures were rarely below 70 F. There is no particular indication of a gradually increasing crack opening, as might be expected as the concrete dried out. In general, the largest openings occurred during the first observed cold snap in September 1959. The openings at this time were larger, in most cases, than those observed subsequently during the winter months, even though the slab temperatures dropped into the thirties.

With the advent of warmer weather in the spring of 1960, the crack openings were smaller than they were at comparable temperatures during the previous warm season.

Variation of Crack Opening With Temperature

The strain-time diagrams show a strong dependence on temperature of both crack opening and strain at a distance from the crack. To illustrate this dependence, diagrams similar to those shown in Figure 20 have been plotted for each panel.

Figure 20 shows all of the data taken by the Whittemore strain gage on panel 4 over the year from the time of construction through the middle of June 1960. This represents, essentially, the first year's performance in regard to the crack openings. The lowest temperatures of the sea-

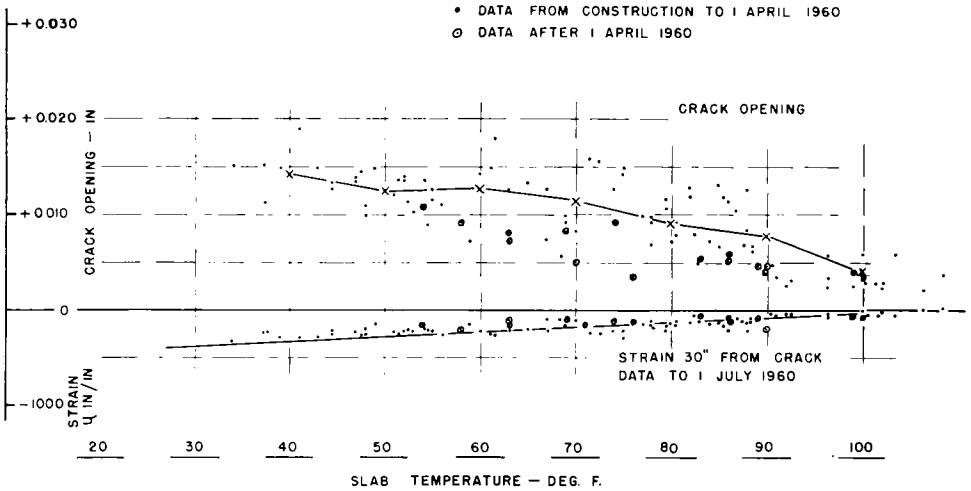

Figure 20. Variation of crack opening and concrete surface strain with slab temperature, panel 4, 0.6% steel, No. 5 bars.

son occurred in early March 1960. The observations made in April, May, and June 1960, the warm weather after the first winter, as mentioned earlier, did not produce as large crack openings as had been produced by comparable temperatures in the previous summer and autumn.

The strains at position 1 conform reasonably well to a trend line that has a slope equal to the coefficient of expansion of concrete (0.000005 per °F) and passes through zero strain at the temperature the concrete had just prior to development of the crack. This trend line is shown in Figure 20 as the dashed line on the negative side of the zero axis.

It is apparent that the readings for crack openings do not fall on a single straight line. With the large amount of scatter present in these data the best averaging that can be done is a grouping at different temperature levels. Therefore, the crack opening over each 10-degree temperature interval was computed. For example: the point plotted at 60 F represents the average of the observed openings between 55 and 64.9 F. The averaging line is plotted through these points of crack opening at each 10-degree interval over the range. The data after the end of cold weather in March are not included in this average trend line.

Diagrams similar to Figure 20 revealed that these trend lines have the same general shape for each panel. The crack openings increased sharply at lower temperatures during the early life of the pavement. However, at later ages and as the slab temperatures fell in the range below 60 F during the winter, the opening did not increase in the same proportions. It also may be noted that the observations for the warmer temperatures in the spring of 1960 in general follow the trend of the points observed for the 30 to 50 F temperature range during the winter, when this trend is extrapolated to the higher temperatures.

Comparison of Crack Openings

The opening of the induced crack for the trend lines for each of the panels is shown in Figure 21. The curves include data to 1 April 1960, only, and do not show the effects of the warmer temperatures after the first winter. The trend lines have a

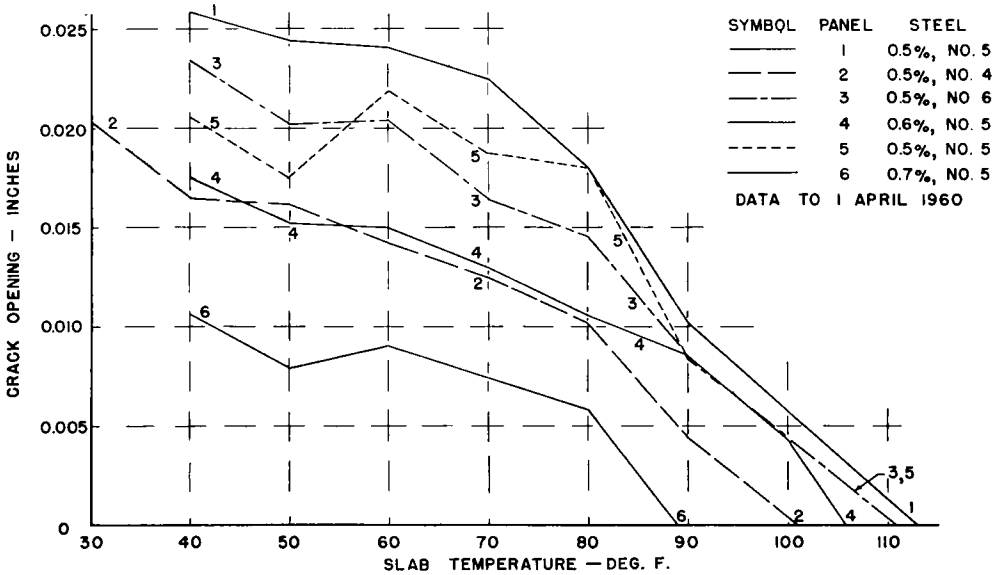

Figure 21. Variation in crack opening with temperature.

somewhat different shape than that in Figure 20 because the temperature corrections for the concrete within the gage length have been applied. These temperature corrections were made using a thermal coefficient for the concrete of 0.000005 per °F, the average value found from laboratory tests of concrete samples taken during construction. The most notable difference in shape is that the dip in the trend curves between 60 F and 50 F is lessened, if not removed altogether.

The crack opening curves in Figure 21 have been extended to the right from the trend curves of the earlier figure, to include zero crack opening at the pavement temperature observed at the last reading prior to the formation of the crack. These temperatures are relatively high, because they include the effects of heat of hydration of the cement.

The crack openings for the first 10 months of pavement life appear to vary between 0.01 and 0.025 in. Further, the trend in the average crack width appears to be a function of temperature. The curves for the several panels have the same general shape and slopes. The differences in ordinates between the several curves appear to come principally from the differences in temperature prior to the formation of the crack. It is readily apparent that if all six curves of Figure 21 were plotted from the same starting temperature, they would lie close together throughout the entire temperature range. This suggests that the temperature attained in the concrete during its early life is a highly significant factor in controlling crack opening.

STRAINS IN REINFORCEMENT

Strain in the reinforcing steel is caused by both stress and change in temperature. The steel is stress free when it is placed in the fresh concrete, and has the initial temperature which is observed at that time. The development of strain in the reinforcement at any time subsequent to placing can be computed, therefore, by

$$\epsilon_s = \frac{\sigma_s}{E_s} + c_s(t - t_0) \qquad (1)$$

in which

ϵ_s=total strain, in inches per inch;
σ_s=unit stress, in lb per sq in. (positive for tension, negative for compression);
E_s=modulus of elasticity, in lb per sq in.;
c_s=thermal coefficient of expansion or contraction, in in. per in. per ° F;
t=temperature, in °F, at time of interest; and
t_0=temperature, in °F, at time of zero readings.

This relationship applies as long as the stresses in the steel do not exceed the elastic strength of the material.

That portion of the total strain caused by stress alone is the first term in the equation. This is the "temperature-compensated" strain. Two SR-4 strain gages (Nos. 3 and 4) were placed in the pavement at the level of the reinforcing steel in each of the test panels. Temperature-compensated readings of strain are obtained by subtracting the reading for gages 3 and 4 (average) from the reading for the active gage on the reinforcing bar. In the discussion of the data which follows, the term "strain" is used to mean temperature-compensated strain.

Strain at Induced Crack

The strains in the reinforcing steel at the induced crack in panel 4 are shown in Figure 22, which presents in detail the slab temperature, the strain at the induced crack, and the strain at 6 in. from the induced crack, all as functions of time. The

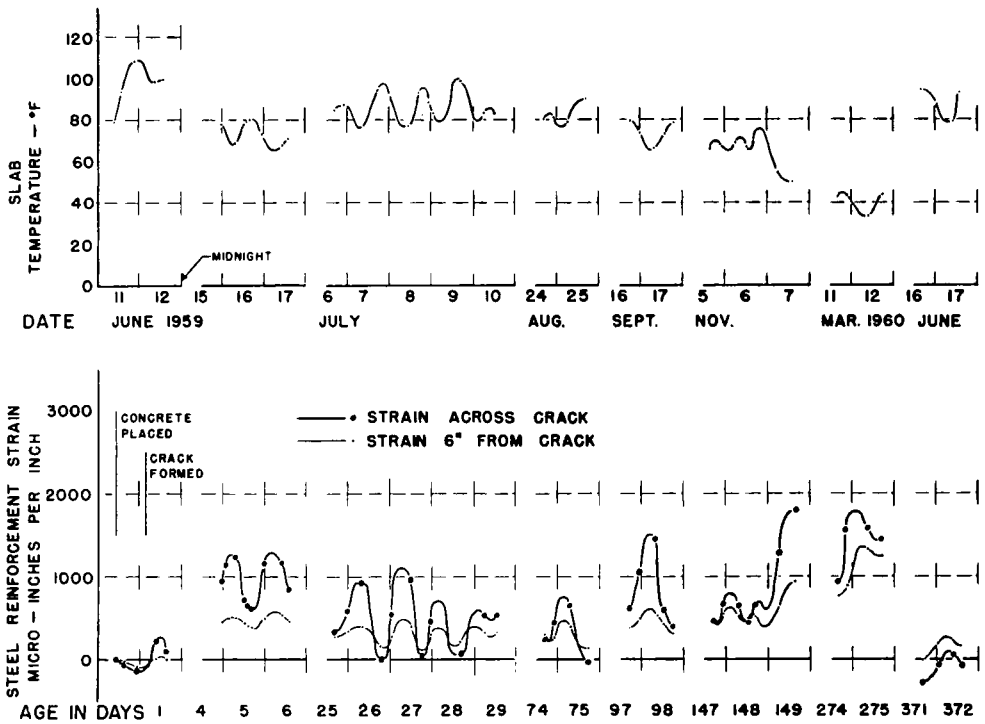

Figure 22. Reinforcement strains, panel 4, 0.6% steel, No. 5 bars.

upper diagram represents the average slab temperature. This average of the three temperature readings in the concrete is felt to be more reliable than the single reading for the temperature coil (No. 30) embedded in the concrete at the level of the steel. The time scale is sufficiently expanded to show the daily fluctuation in temperature.

The lower diagram of Figure 22 shows the strain at the induced crack and the strain in the reinforcing steel at 6 in. from the induced crack. Strain at the crack is taken as the average of the eight strain gages (Nos. 5 through 12) at this location. Strain 6 in. from the crack is the average of four gages (Nos. 13 through 16), two on either side of the crack. These are temperature-compensated strains taken from zero strain at the initial reading when the concrete paving had been completed at the test panel. Figure 22 is a representative sample showing the variation in strain over daily temperature cycles. All panels behaved similarly in this respect.

Comparison of Strain-Time Curves

Figure 23 shows the average temperature-compensated steel strains across the crack for all six panels for the entire year. The variation in slab temperature through the year is given for reference. The upper and lower curves in each case represent maximum and minimum strains observed at each 24-hr cycle of observations. Variation in slab temperature is shown similarly with maximum and minimum lines. The high initial temperatures caused by hydration of the cement during the first two days are shown only for box 6, the first one installed. However, the individual strain-time curves show the effects of this initial temperature rise in each case.

Figure 23 shows a similarity in strain-time behavior for all panels throughout the year. Maximum observed reinforcement strains ranged from 1,200 to 1,550 μ-in. per in. during the first summer. With falling temperatures from September through November, the maximum strains range from 1,400 to 2,000 units, except for panel 1, and remain within this range through the winter months. With the advent of warmer weather following the first winter the strains decreased, generally to smaller values than they had at comparable temperatures during the previous summer. Although the strains differ in magnitude, the same pattern of behavior is seen to apply to all panels except No. 1.

In panel 1 a maximum strain of 2,350 μ-in. per in. was developed in September 1959. Again in October, maximum strains rose above the yield point value of 2,260 units, followed by a gradual increase during the winter months. Further contraction of the concrete during the cold winter months operated against the reduced stiffness of the steel in the yield point range, with consequent increase in the strains at the crack. The resisting force at the crack does not increase proportionally with strains during this range.

Strain-Temperature Relations

Temperature - compensated steel strains across the crack are plotted as functions of slab temperature in Figures 24 and 25. Although there is scatter in the data, a definite trend toward increasing strains with decreasing temperatures is shown. The trend line has been plotted through average points at the even 10-degree intervals. Separate trend lines are plotted for the observations made after 1 April 1960.

With the return of higher temperatures in the spring of 1960, the reinforcement strains shown in Figure 25 for panel 4 are considerably less than they were for corresponding tem-

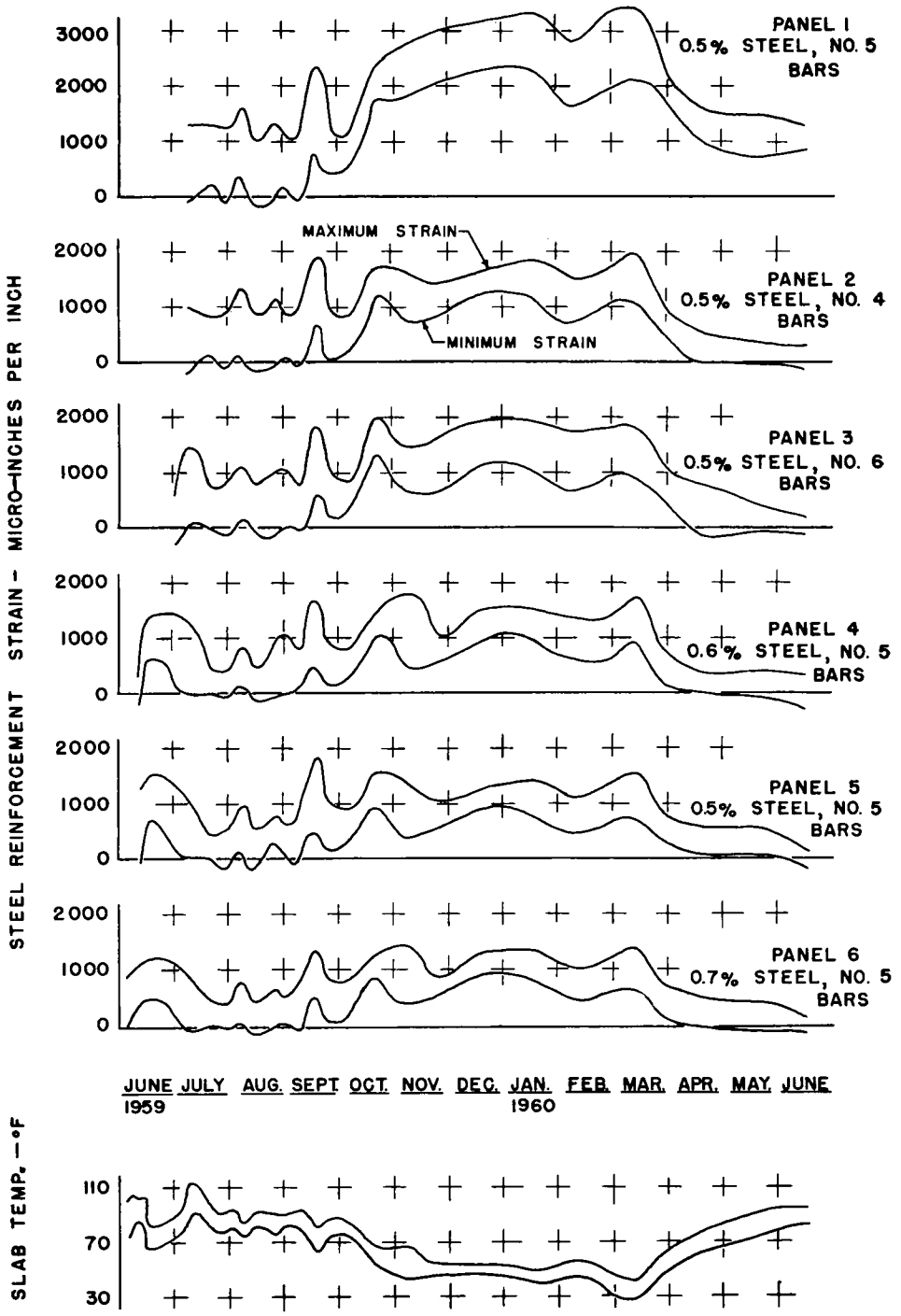

Figure 23. Variation in steel strain across crack in first year.

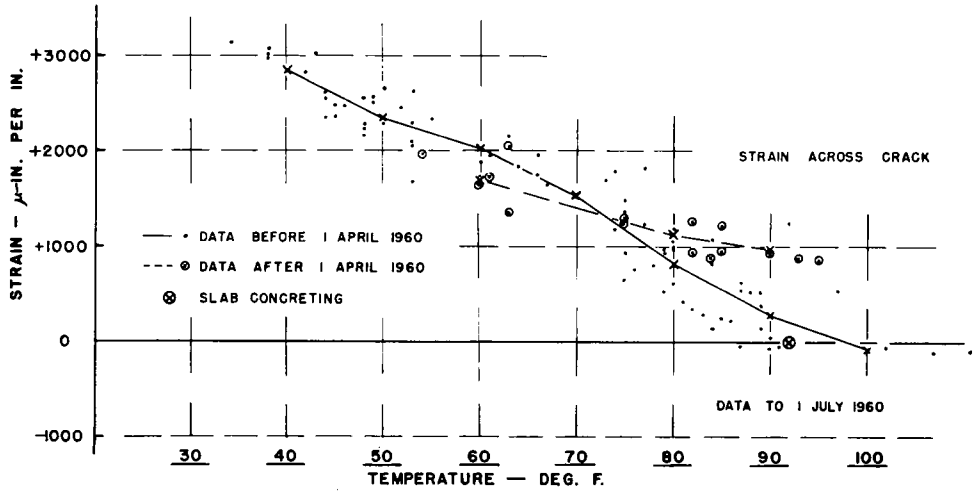

Figure 24. Temperature-compensated steel strains *vs* slab temperature, panel 1, 0.5% steel, No. 5 bars.

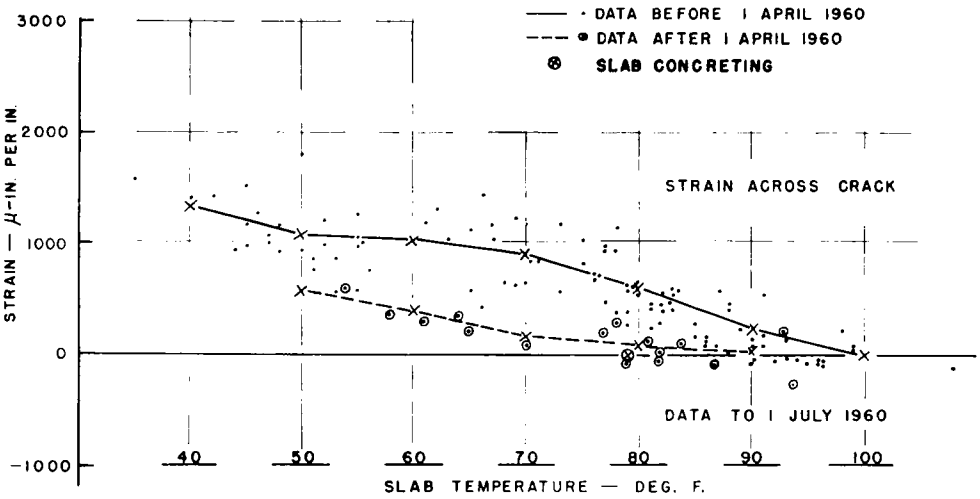

Figure 25. Temperature-compensated steel strains *vs* slab temperature, panel 4, 0.6% steel, No. 5 bars.

peratures earlier in the life of the pavement. In all cases except panel 1, these later strains are small, and the trend line through these points has a lower slope than the original curves. In Figure 24 it may be noted that the trend line for panel 1 has a slope quite similar to that in the other panels, but is displaced upward about 1,000 micro-inches. This similarity of slope to the other panels and the displacement are indications of the permanent set that has been produced in the reinforcement in panel 1.

The trend lines from the strain-temperature diagrams for the six panels (Fig. 26) represent the behavior from construction through March

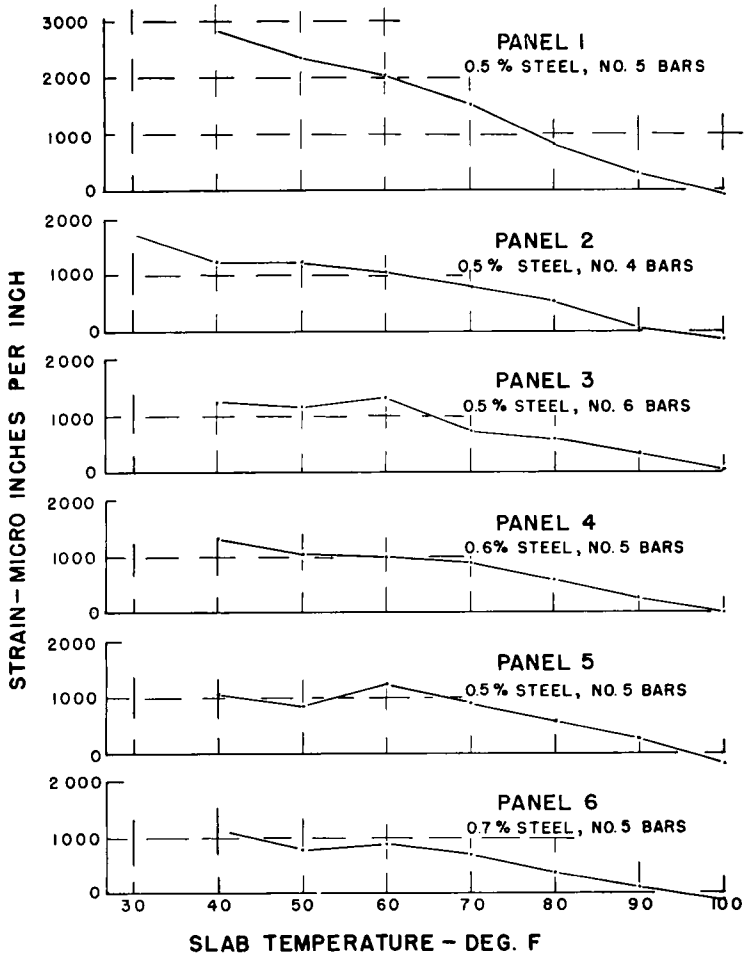

Figure 26. Temperature-compensated steel strains across crack *vs* slab temperature.

1960. There is a marked similarity in the magnitudes of strains at the same temperatures for all panels except No. 1. The strain in the reinforcement at the crack in panel 1 is roughly twice as great as it is for the other panels at all temperatures. The strains in panels 4 and 6, which contain the higher steel percentages, are somewhat less than in panels 2, 3, and 5. The average strain-temperature relation is nearly linear for panels 1 and 4. In panel 2, a slight dip is shown at the 40° temperature. The curves for panels 3, 5, and 6 show a trend that is essentially linear down to 60 F, but then drops below this line at lower temperatures. This break is similar to that noted on the corresponding diagrams for crack openings (Fig. 21).

Strain Variation Along Reinforcement

Strains in the reinforcement at 12, 24, and 48 in. from the induced crack are relatively small. Complete diagrams showing the variation in strain at these locations as a function of

time are difficult to interpret. It is necessary to show the variation in strain with respect to both distance along the bar and time in order to present a complete picture.

Isometric drawings seem to offer the best method for representing the strain variation along the reinforcing bars. Selected diagrams of this type are shown in Figure 27, in which strain is plotted vertically with tensile strain upward and compressive strain downward from the coordinate axes, time is plotted along the axis running down and to the right, and distance along the bar is plotted on the coordinate axis running downward and to the left. Slab temperature for each observation time is indicated above the strain distribution diagram. Observations over four 24-hr periods through the first year are shown. Panel 5 is typical of the behavior.

As shown in Figure 27a, higher temperatures during the afternoon following development of the crack caused the strain at the crack to decrease to very small values. Compressive strains along the bar increased at the same time. The behavior during the first summer (Fig. 27b) is characteristic of a daily cycle at that time.

The cold weather readings in March (Fig. 27c) show relatively high tensile strains at the crack and along the entire length of the reinforcement. The warmer temperatures in June 1960 (Fig. 27d) caused the very small strains which are typical of this time. Small tensile strains exist along the entire bar at the lower temperatures, whereas strains at the crack and along most of the bar length are compressive at the higher temperatures. As noted in the first summer, strain again exists at 6 in. from the crack at the higher temperatures.

Strain Performance

The behavior of all six test panels in this investigation is remarkably similar. Although there are differences in the magnitudes of the effects, the performance as indicated by both the Whittemore strain gage readings on the concrete surface and the SR-4 readings on the reinforcement showed the same general pattern.

The most striking factor affecting the performance at the crack is temperature change. The concrete segments between cracks attempt to expand and contract with fluctuations in temperature. Upon contraction, the cracks open and tensile stresses in the reinforcement increase. The amount of such changes in tensile stress over a daily cycle of temperature fluctuation is between 20,000 and 30,000 psi through the first summer and winter. These daily fluctuations in stress seem to be somewhat less in the warmer weather of the spring and early summer following the first winter.

Crack openings and reinforcement strains vary in response to seasonal temperature changes over the first year, as well as with the daily temperature cycles. Peak strains and crack openings are associated with low values of minimum slab temperature. The scatter in the data, however, indicates the influence of factors other than temperature.

There is no particular indication in these data of a build-up in strain as the concrete became drier during the weeks after construction. Equilibrium between moisture in the pavement and the summer atmospheric conditions must have been established relatively soon after the curing paper was removed.

For the range of slab temperatures during the first summer the slopes of the strain-temperature diagrams are relatively steep. If these slopes were extrapolated downward to temperatures in the thirties, very large strains would be indicated for winter conditions. The strains actually measured during the winter tempera-

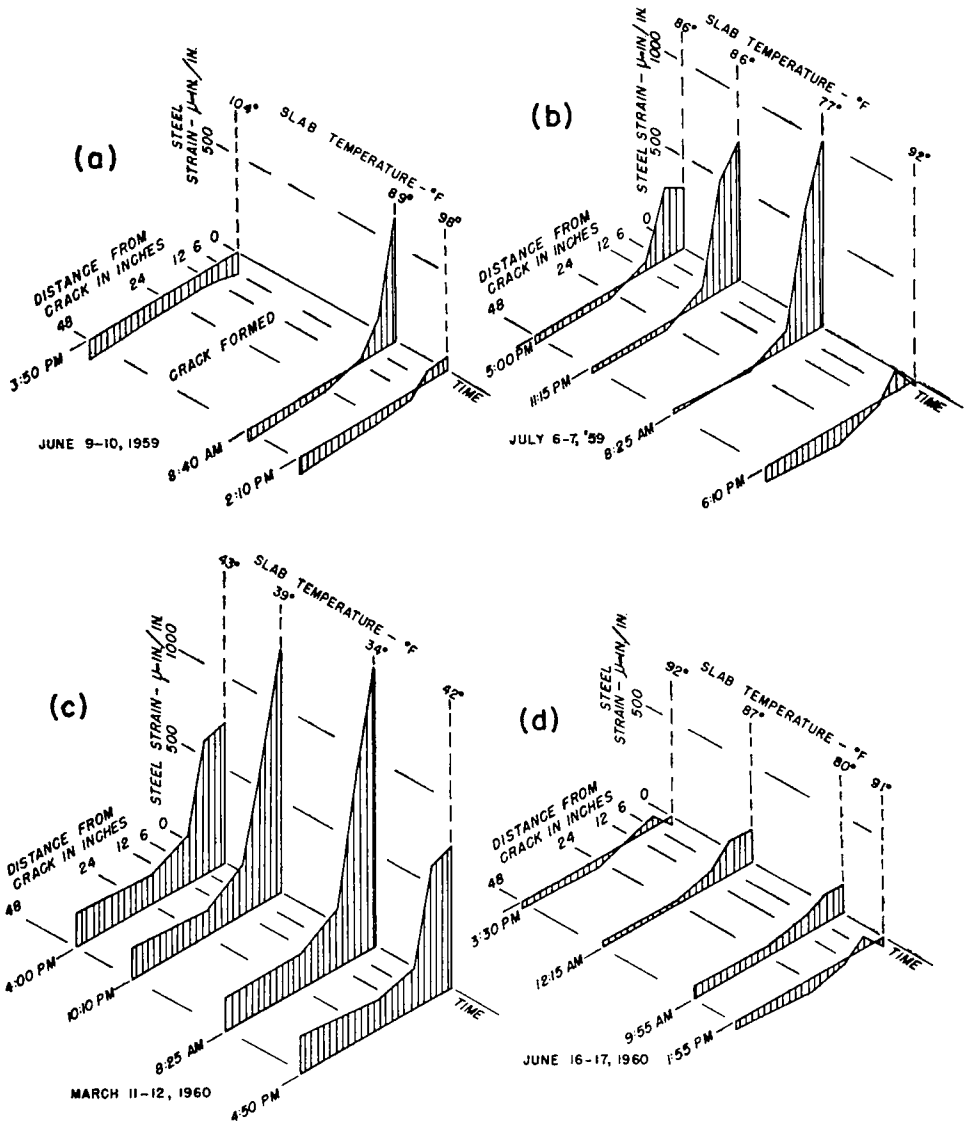

Figure 27. Strain variation along reinforcement, panel 5, 0.5% steel, No. 5 bars.

tures are considerably less than might have been expected from the early observations.

The largest strains that were observed, in most cases, occurred with the cool weather in the autumn. In a few instances these strains were exceeded, but only by small amounts, at the much lower slab temperatures during the winter months. It may be that the autumn rains and the lower slab temperatures during the winter combine to increase the moisture content of the pavement in the winter months. Such increase in moisture could produce an expansion of the

concrete, which would offset, in part, the contraction caused by lower temperatures.

The formation of additional cracks is another factor that may be responsible for the fact that strain in the reinforcing steel did not increase greatly during the first winter. The restraint offered by the reinforcement as the temperature became lower produced greater tensile stresses in the concrete. The diagrams of steel strain distribution along the bars for the winter months show tension to exist throughout the pavement length. This tension is undoubtedly the cause of additional cracking in the segments between the original cracks. The formation of additional cracks would tend to relieve strain in the existing segments and to reduce both the reinforcement strain and crack opening at previously existing cracks.

The total tensile force in the reinforcing steel at transverse cracks is essentially constant from one crack to the next through the central portions of a continuously-reinforced length. In going from one crack to the next the total force can be changed only by the amount of the subgrade friction. In the short lengths between adjacent cracks this frictional force is small when compared to the total force at a crack. Thus, when additional cracks form the segments between cracks must move as a whole to adjust to this condition of statics. Vibration from traffic undoubtedly hastens these movements.

Reinforcing steel was strained beyond the yield point only in panel 1. There is no obvious explanation for this in the conditions at the time of construction. Panels 1 and 5 are identical in design, except for the difference of 1 ft in pavement width. Although the concrete temperature before cracking in panel 1 was somewhat higher than it was in panel 5, it is not the highest temperature of any panels. The initial strains when the crack opened are similar in the two cases. The only marked difference between panel 1 and the other panels is a greater spacing between cracks during the early life of the pavement. This greater spacing is consistent with larger crack width and higher steel strain.

CONCLUSIONS

The following conclusions may be drawn from the first-year performance of this experimental pavement:

1. Maximum crack openings observed during the first year at the six instrumented panels ranged from 0.01 to 0.025 in. at the pavement surface. Although the panels which had the higher percentages of reinforcement showed smaller crack openings, the temperature of the fresh concrete prior to cracking appears to be a more important factor affecting crack openings during the first year than the range in amount of reinforcing steel used in this investigation.

2. Daily fluctuations in temperature produced changes in crack opening as great as 0.01 in. Stress in the reinforcement varied by as much as 20,000 to 30,000 psi over daily temperature cycles.

3. In this experimental pavement, which was constructed during the early summer, maximum crack openings and steel strains occurred, in most instances, with moderately cool autumn temperatures and did not increase proportionately with falling temperature during the winter months of the first year.

4. Straining of the steel beyond the yield point occurred at only one of the six test panels. This panel was reinforced with 0.5 percent of steel. Permanent set amounted to about 1,000 μ-in. per in. at the crack.

5. Warmer weather following the first winter caused smaller values of

crack opening and steel stress than were caused by comparable temperatures during the first summer. In fact, the reinforcement at the crack was in slight compression rather than tension for warmer temperatures of the second summer.

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