Maryland Investigation of Continuously-Reinforced Concrete Pavement: 1959-1964 Strain Observations

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The Maryland experimental continuously-reinforced concrete highway pavement was constructed on the Baltimore-Harrisburg Expressway, Interstate 83, during the summer of 1959. The pavement and subbase thicknesses are constant; steel percentages of 0.5, 0.6, and 0.7 percent and bar size Nos. 4, 5, and 6 are the variables.

Strain and temperature measuring instruments were installed at six sites to investigate the behavior of the reinforcing steel in the vicinity of induced cracks. Instrumented bar mat panels containing different amounts and sizes of reinforcing steel were constructed in each of the sections.

Observations of steel strains, crack openings, and pavement temperatures have been made over a $4\frac{1}{2}$-yr period. The instruments and gages, with a few exceptions, have given satisfactory readings since construction.

Reinforcement strains and crack openings vary with the daily range in pavement temperatures. Strain at the crack per degree temperature change is generally greater in winter than in summer. The highest steel strains at the crack and the largest crack openings occurred in the sections containing the smallest amount of reinforcing (0.5 percent). The magnitude of the reinforcing strain is not appreciably affected by the bar size and is controlled primarily by the percentage of reinforcement. The width of the crack opening, however, is governed by both the percentage of steel and bar diameter. Construction conditions also have an apparent influence on pavement stresses.

In all sections of experimental pavement, certain trends occur with age. The crack openings have shown a slight increase and the maximum steel strains at the crack have decreased as the pavement aged. The rates of increase or decrease are greater in sections with smaller percentages of steel.

•CONTINUOUSLY-REINFORCED concrete highway pavement is reinforced longitudinally by continuous steel over the entire length of the pavement, eliminating the need for joints. Restraint imposed by the weight of the slab and subgrade friction in the end portions prevents large movements due to expansion or contraction of the concrete within the central portion of the pavement. Since the central portion of the pavement is restrained, there are no changes in length due to expansion or contraction, and therefore, compressive or tensile stresses are induced in the pavement as temperature and moisture variations occur. The compressive stresses which develop are small in relation to the strength of the concrete and do not cause any structural damage to the pavement. The tensile stresses, however, develop to sufficient magnitude to cause transverse cracking of the concrete. The function of the reinforcement is to prevent these cracks from opening to an extent detrimental to the pavement. The longitudinal reinforcement must be of sufficient amount and strength to serve this purpose.

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DESCRIPTION OF PROJECT

Objectives

The primary objectives of the Maryland continuously-reinforced concrete pavement field tests (3) were to make a thorough study of the behavior of the pavement with particular emphasis on the following:

1. The relative effects of different amounts (0.5, 0.6 and 0.7 percent) of longitudinal reinforcement of constant bar size; and
2. The relative effects of different bar sizes (Nos. 4, 5 and 6) with the amount of reinforcement constant at 0.5 percent.

Scope

This paper discusses data gathered with instrumentation embedded in the concrete in each of six test sections of continuously-reinforced pavement. The instrumentation consists of SR-4 strain gages attached to the bars, temperature coils, and Whittemore gage lines on the surface of the concrete pavement. Data were gathered on steel strains, crack openings, and pavement temperatures at and near induced cracks near the center of each continuously-reinforced length. In addition to the instrumented panels, information concerning crack spacing and crack openings was gathered (4).

Test Sites and Properties

During the summer of 1959, six continuously-reinforced sections of highway pavement were constructed as part of the Baltimore-Harrisburg Expressway I-83. These test sections are situated between Mt. Carmel and Middletown Road interchanges in the vicinity of Parkton, about 25 mi north of Baltimore. The continuously-reinforced lengths ranged from 1,800 ft for a single section to 9,800 ft consisting of 3 sections. Detailed descriptions of the field test sites have been given previously (3, 5).

The test sites, numbered one through six (Fig. 1) from south to north, in the southbound roadway, contain an instrumented panel near the center of each. The instrumented panel consists of two bar mats in the shoulder lane, in which eight reinforcing bars have been replaced by instrumented bars.

The continuously-reinforced pavement is 8 in. thick, placed on a 6-in. layer of selected subbase material of crushed local rock. The controlled variables in the experimental pavement are the size and the amount of the reinforcing steel.

Sites 1, 2, and 3 all contain, nominally, 0.5 percent steel with bar size Nos. 5, 4 and 6 in each of the three sites, respectively. There are no intermediate joints between sites 4, 5, and 6; the transition from one section to the next is accomplished by changing the bar mats and percentages of reinforcement. The reinforcement in sites 4, 5, and 6 is composed of No. 5 bars in 0.6, 0.5, and 0.7 percent, respectively.

The reinforcement is deformed bar hard-grade billet steel. The mats are made in 16-ft lengths. Each mat is approximately 6 ft wide. The longitudinal reinforcement is tied to seven No. 3 transverse bars per mat. The specified longitudinal overlap is 12 in. for the No. 4 bars, 13 in. for the No. 5 bars and 15 in. for the No. 6 bars. All reinforcement is located at middepth of the concrete. The following average values of reinforcement yield point were found: bar size No. 4, 71,300 psi; bar size No. 5, 61,500 psi; and bar size No. 6, 55,000 psi.

The instrumented panel near the center of the length of each test site included strain and temperature measuring devices (Fig. 2). The regular reinforcing bars in each mat were replaced by instrumented bars. The instruments which furnished data for this study were as follows: (a) temperature compensating gages; (b) strain gages on the reinforcing steel; (c) temperature measuring coils in the slab; and (d) Whittemore strain gage lines on the surface of the concrete.

The instruments are located within the length of one bar mat, in the shoulder lane near the middle of the length of each test section.

The temperature compensating gages are SR-4 strain gages, type AB-7, which are mounted on 1½-in. long pieces of reinforcing steel and covered with waterproofing and

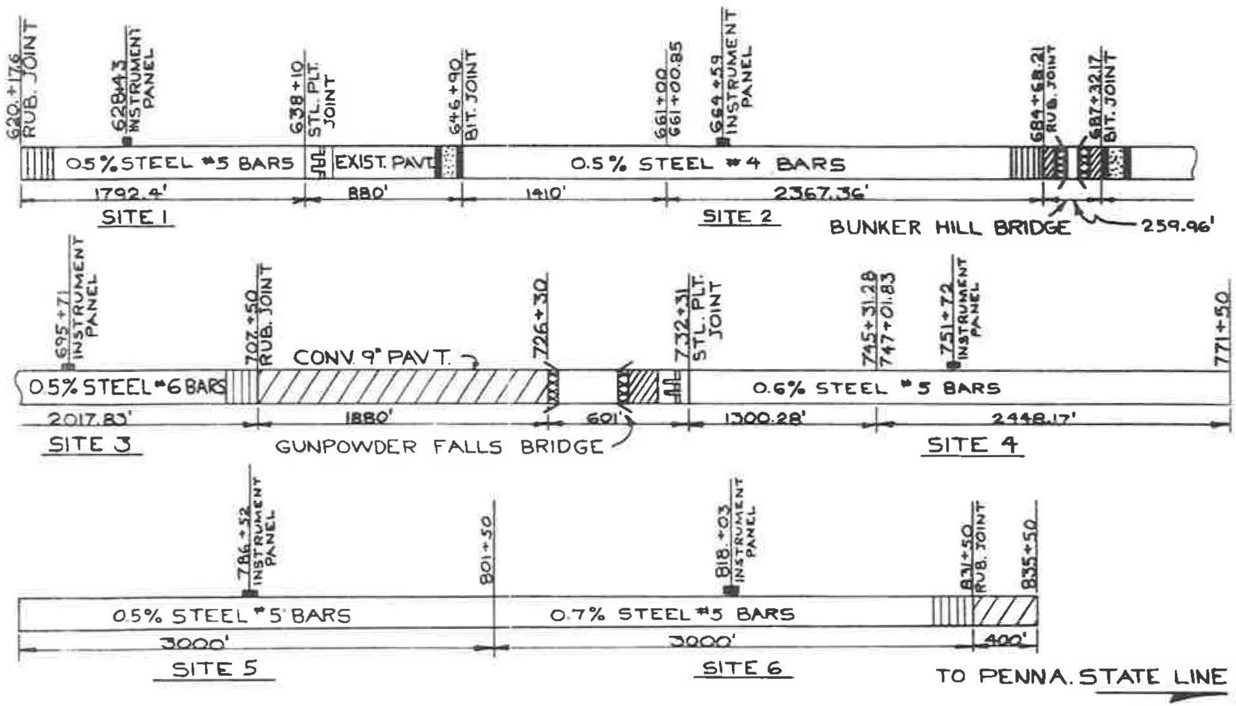

Figure 1. Experimental sections, Baltimore-Harrisburg Expressway.

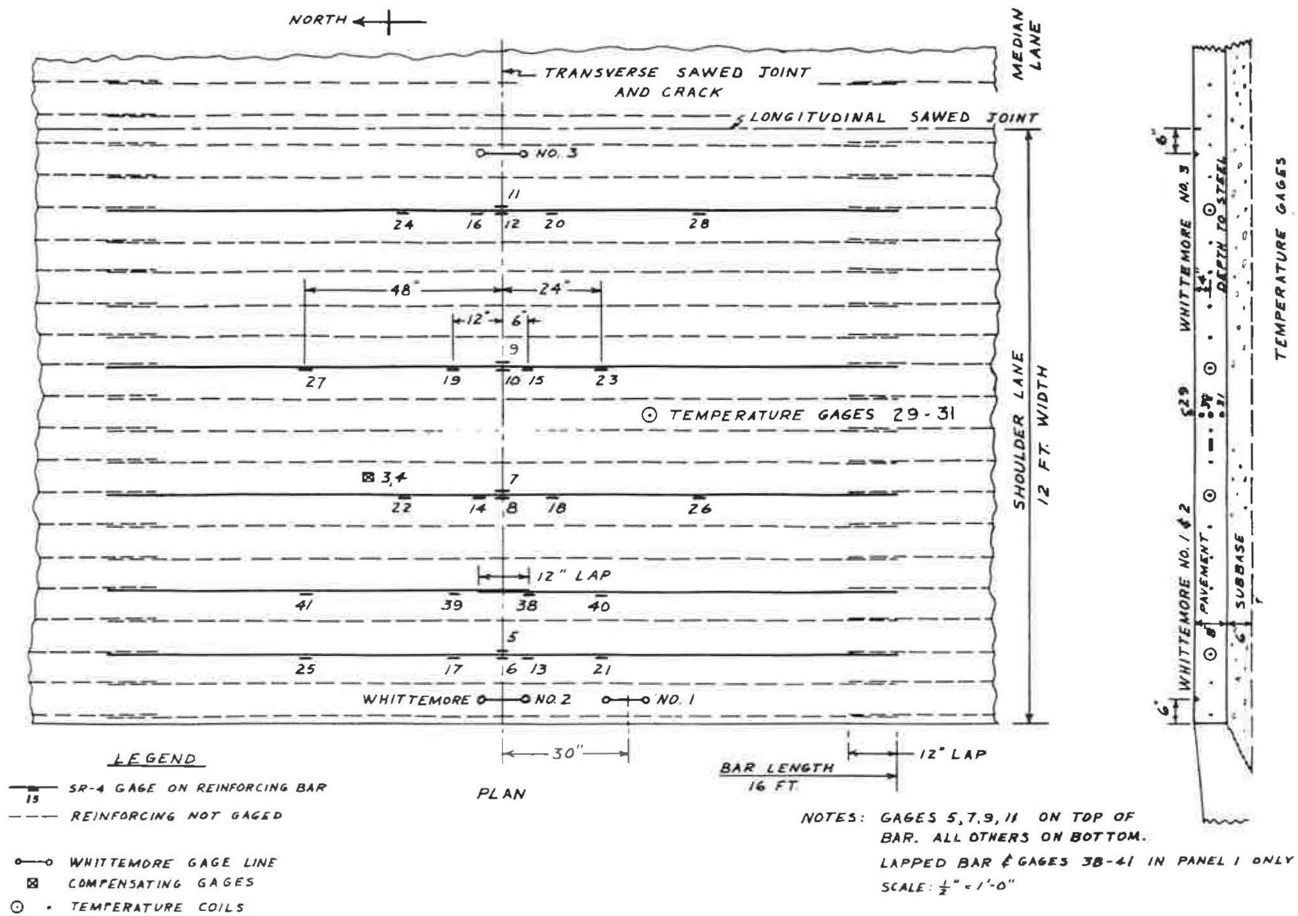

Figure 2. Arrangement of instruments in pavement.

protective tape. The temperature compensating gages are located at middepth in the pavement, 4 to 5 ft from the shoulder edge. The readings from these gages provide temperature compensation for the strain readings on the reinforcement.

The strain gages on the bars are SR-4, type AB-7. There are eight gages at the induced crack (Nos. 5 through 12) placed diametrically opposite each other (to compensate for the effects of bending) on four bars equally spaced across the lane width. Gages on the underside of the bar are located at 6, 12, 24, and 48 in. from the induced crack.

The temperature measuring coils are located in the concrete slab near the middle of the lane and 3 to 4 ft from the induced crack. In each panel, there are 3 temperature measuring coils located 1 in. below the pavement surface, at pavement middepth, and 1 in. above the bottom of the slab.

The electrical leads from the strain gages are carried through a conduit beneath the shoulder to a terminal box containing resistance coils for balancing the electric circuits, and a plug-jack board to which connections are made with the strain indicator.

The brass plugs for the Whittemore gage lines were placed in the fresh concrete immediately following the finishing operation. No. 1 is 30 in. from the induced crack. Gage lines 2 and 3 span the induced crack. The average of readings on gage lines 2 and 3 yields the average crack opening.

The object of the strain readings is to determine the reinforcement behavior in the vicinity of a natural crack. To induce a crack as nearly natural as possible, a 2-in. deep transverse saw cut was made over the center of the instrumented mat 6 to 8 hr after placing of the concrete. In all six cases the cut was made without incident or damage to the concrete surface. After the concrete had cured and dried, the saw cut was sealed with a bituminous compound.

Observation Procedure

The zero readings were taken after the mat had been placed in the concrete and the mechanical finishing operations had passed over and beyond its length. Throughout the summer of 1959, while construction was occurring, measurements were taken over 24-hr periods at weekly or more frequent intervals. The weekly readings were continued until January 1960, at which time biweekly readings were taken. Between April 1960 and August 1961, readings were taken monthly. After August 1961, readings were taken every six months during the winter, in January or February, and during the summer, in August, to provide the minimum data necessary to study the pavement behavior.

At each date, four separate readings for each panel were taken over a 24-hr period. These four readings make up the daily cycle. The first was usually taken in the afternoon, when the pavement temperatures were at their maximum; the second, around midnight, when the pavement temperatures had started to drop; the third, early in the morning, when pavement temperatures were at a minimum; and the fourth, during the afternoon of the second day.

Of the original 226 gages installed, after 4½ yr only 25 are either completely inoperable or yield unusable readings.

INDUCED CRACK OPENING, STEEL STRAIN AT CRACK AND SLAB TEMPERATURE OBSERVATIONS

Variables Affecting Behavior

The behavior of continuously-reinforced pavement is affected by several natural variables including: (a) temperature in the concrete; (b) average transverse crack spacing; and (c) changes occurring with age (i.e., creep, changing concrete properties, and effects of traffic loadings).

The effects of these variables on the crack opening and on the reinforcement strains are examined in the data from the field observations. These variables are, of course, uncontrollable in a field investigation such as this. Furthermore, the moisture content of the pavement fluctuates (9, 10) and undoubtedly affects the strains, but this cannot be studied as a variable since there was no provision made for moisture measurement. No data were taken for the stresses induced by wheel loads, but the cumulative effect of traffic loads is inseparable from the other long-range influences.

Line Graphs

Variations of induced crack opening, steel strain at the crack, and slab temperature with pavement age are shown in Figures 3 and 4. Each vertical line is representative of the readings taken over a 24-hr cycle. The extremities of the lines, that is, the upper and lower ends, show the maximum and minimum readings taken during each daily cycle. This difference is the daily range. Throughout this report, the steel strains are compensated for temperature changes and hence represent strain caused by stress alone.

The daily range of both the steel strain and the crack opening, in general, is larger during the winter daily cycles than during the summer daily cycles, even though the slab temperature range is smaller during the winter than during the summer. This may indicate that certain of the variables, probably moisture, have more effect on the pavement during the winter months than during the summer months, since the coefficients of expansion and contraction at 40 F in water are greater than those at 40 F in air (8). The coefficients of expansion and contraction for concrete were slightly higher at the lower temperature range, 0 to 40 F (5). Compression during the summer months may be a contributing factor to this condition. The available expansion space may be used up before the pavement is subjected to the peak daily temperature.

The line graphs are composed of a series of undulations or cyclic variations. Each cycle occurs over a 1-yr period and is defined as a yearly cycle (as opposed to a daily cycle of 24 hr). Both the steel strain at the crack (average of gages 5 through 12) and the induced crack opening (average of Whittemore readings 2 and 3) vary inversely with the average slab temperature. The yearly cycles of the steel strains have shown a tendency to decrease in amplitude; that is, the variation or difference between the summer and winter readings for the steel strains has become less as the pavement age increases. The crack opening variation, however, is either increasing or remaining essentially constant as age increases. In all panels, except panel 1, the maximum crack opening has increased and the minimum crack opening has increased also, causing an upward trend in the crack opening cycles. This will be discussed in more detail as other curves are introduced.

Crack Openings

The graphs for sites 1, 2, and 3 show that the induced crack opening of panels 1 and 3, with No. 5 and No. 6 bars, respectively, has exceeded 0.03 in. over several winter periods. The induced crack opening in panel 2, with No. 4 bars, has been recorded as opening above 0.03 in. only once. Since each of these panels contains 0.5 percent steel, the size of the bar seems to be a significant factor. The average of the maximum crack openings observed during the five winter periods of observation is plotted against bar size in Figure 5a. The indication seems to be that the smaller bar size, with its greater ratio of surface to cross-sectional area and, consequently, relatively greater bond capacity, is effective in keeping the crack opening as small as possible under conditions of constant steel percentage.

However, panel 5, containing 0.5 percent steel composed of No. 5 bars, has only two recorded crack openings greater than 0.025 in. This indicates that variables other than bar size influence crack opening. In this case, the temperature at which the pavement was placed and cured is considered the controlling variable. Panel 5, along with panels 4 and 6, was concreted during the first two weeks in June and cured under nearly ideal conditions. Panels 1, 2, and 3 were concreted during the first two weeks of July and cured during July under hot, dry conditions.

Panels 4 and 6 with 0.6 and 0.7 percent steel, respectively, have the smallest maximum crack openings of the six test panels. The crack openings of panel 4 remained below 0.020 in. while those of panel 6 were not recorded as exceeding 0.013 in. Figure 5b shows the effect of steel percentage on the maximum crack openings. In

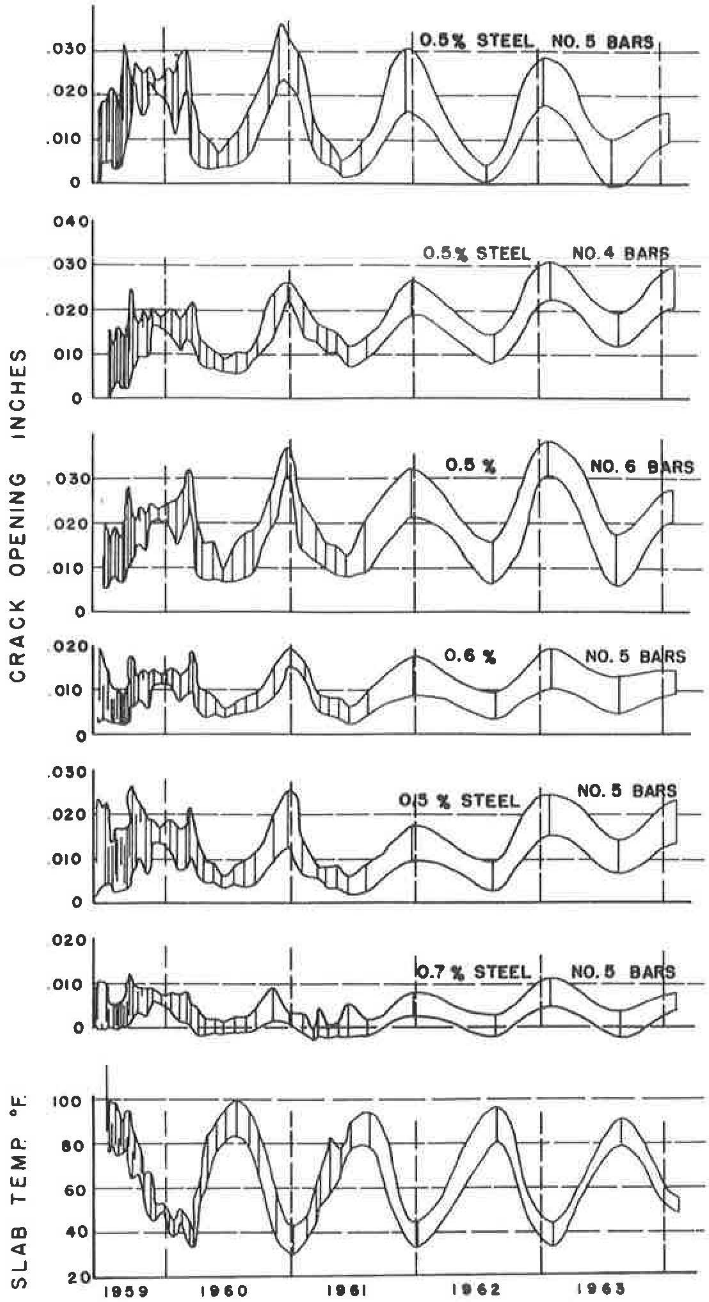

Figure 3. Variation of induced crack opening with pavement age.

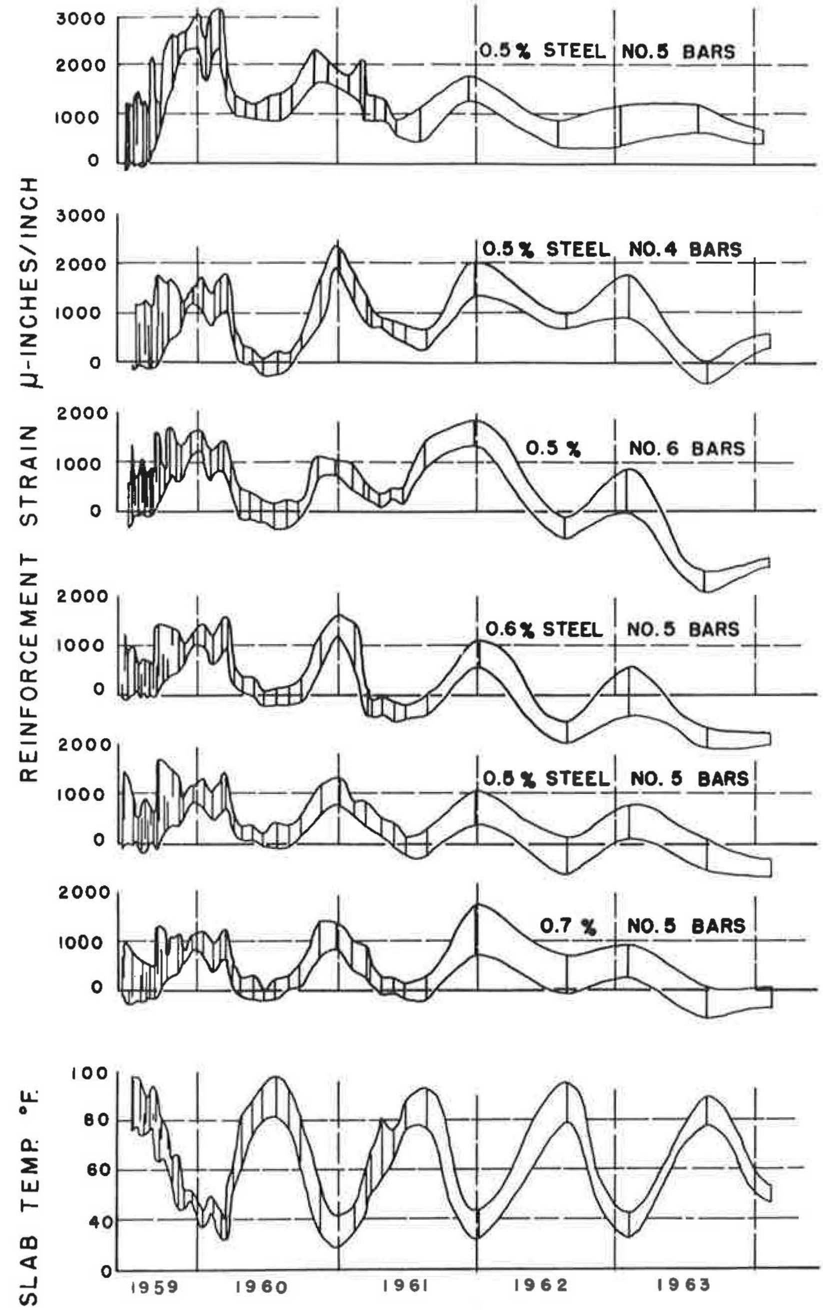

Figure 4. Variation of steel strain at induced crack with pavement age.

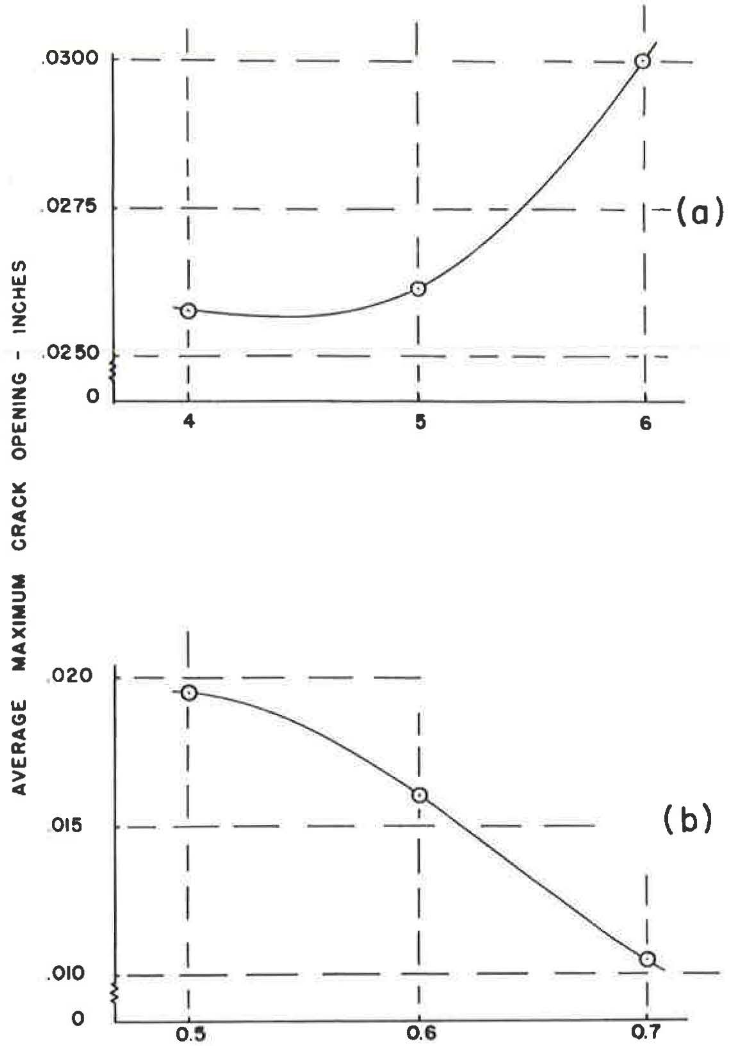

Figure 5. Effect of (a) bar size and (b) steel percent on average maximum crack opening.

addition to bar size and concreting conditions, therefore, the percent of reinforcement is the dominant factor controlling the size of crack opening.

Strains at Induced Crack

The line graphs (Fig. 4) of the steel strains at the induced crack of panels 1, 2 and 3 differ somewhat from the graphs of the crack opening, in that the largest steel strains at the crack do not necessarily coincide with the largest crack opening in any one panel. The steel in panel 1 was strained beyond its yield point during the first winter of exposure—October and November 1959. After the summer of 1960, the seasonal undulations in steel strain at the crack level off, but crack openings have large seasonal changes. The daily range, however, appears unaffected. In panel 2, the yield point of the steel may have been exceeded during the winter of 1960-1961. The steel strains at the crack for the next four readings show signs of leveling off. The steel of panel 3 also shows signs of having been strained beyond its yield point.

The steel strain readings for both summer and winter show a steadily decreasing trend and are going into compression for readings of August 1963 and February 1964.

The pavement temperatures recorded during the winter of 1963-1964 were 10 to 15 deg higher than those of preceding years. These high temperature values together with the high winter moisture content of the pavement may result in the summer-like strains that appear in all the panels.

The steel in panels 4, 5 and 6 has no recorded strains above the yield point and extreme variation in the yearly cycle does not appear. The percentage of steel has very little effect on the maximum strains in these three panels. In each panel the maximum recorded strain is below 2, 000 μ-in./in. The temperature and conditions of placing and curing seem to be the factors responsible for the nearly equal strains in these three panels. Each of these panels seems to be following a general trend of decreasing yearly strain variation, with little effect shown on the range of the daily cycle.

In sites 1, 3 and 5 it has been necessary to repair portions of the pavement near the instrumented panels, as described in detail by Lee (4). A failure occurred in site 1 27 ft south of the induced crack during the summer of 1963 at a reinforcement overlap. Extreme cracking was noted in an area 8 ft wide in the shoulder lane of the southbound roadway. Repairs were made in May 1964. Two failures occurred near the induced crack in site 3. One was 39 ft north of the induced crack, the other 60 ft south of the induced crack. Both were noticed during the winter of 1961-1962 and repaired during the fall of 1962. Two failures in site 5 occurred near the induced crack. One was 47 ft south of the induced crack, and 10 ft wide and covered the shoulder lane of the southbound roadway. The other was 70 ft north of the induced crack and extended the entire width of the southbound roadway. Repairs were made in the fall of 1962. The effects of the pavement failures are indicated in the line graphs. In each case, the loss of continuity had a greater effect on the steel strains than on the crack opening widths.

Although these line diagrams show the complete history of crack openings and steel strains at the crack for the $4\frac{1}{2}$-yr observation, they can lead only to general conclusions as to magnitudes of these quantities. In the following sections, an attempt is made to separate some effects to permit better understanding of the behavior shown in these diagrams.

CRACK SPACING

The average crack spacing is equal to a distance divided by the number of cracks contained within this distance. Only the central 500 ft of each test site was studied for crack spacing (3). Records of average crack spacing were started from 1 to 3 mo. after the paving was completed. These records were kept by the Maryland State Roads Commission. The dates of observation of crack spacing do not necessarily coincide with those at which strain and crack opening readings at the induced crack were taken. The average crack spacing decreases as the age of the pavement increases. The curve ranges from the initial crack spacing in August 1959 to an essentially constant value 2 to 3 yr after placing of the concrete. In all the sites, the largest decrease in crack spacing took place during the first year after the pavement was placed. The largest amount of cracking occurred during the time required for the pavement to cure (Fig. 6).

After the first winter, with its low temperatures, the formation of new cracks decreased quite rapidly and the curves began to level off.

The 1963 values indicate that the higher percentages of steel produce the more closely-spaced cracks, and it also appears that the size of the bar contributed to the number of cracks formed (Fig. 7). Site 2 with the smallest bar size, No. 4, for 0.5 percent steel, has the smallest average crack spacing and consequently the largest number of cracks. Site 3 contains No. 6 bars and has the largest average crack spacing. The crack spacing varies directly with the bar size of the reinforcement or inversely with the surface area of the reinforcement. The crack spacing varies inversely with the percentage of reinforcement. The cracks are more evenly distributed throughout the sites containing the higher percentages of steel. The cracks of sites having 0.5 percent reinforcement appear closely spaced in some sections of the sites and widely spaced in others (noted from unpublished crack survey data, similar to those in Figure 11, Ref. 3, taken by Maryland State Roads Commission).

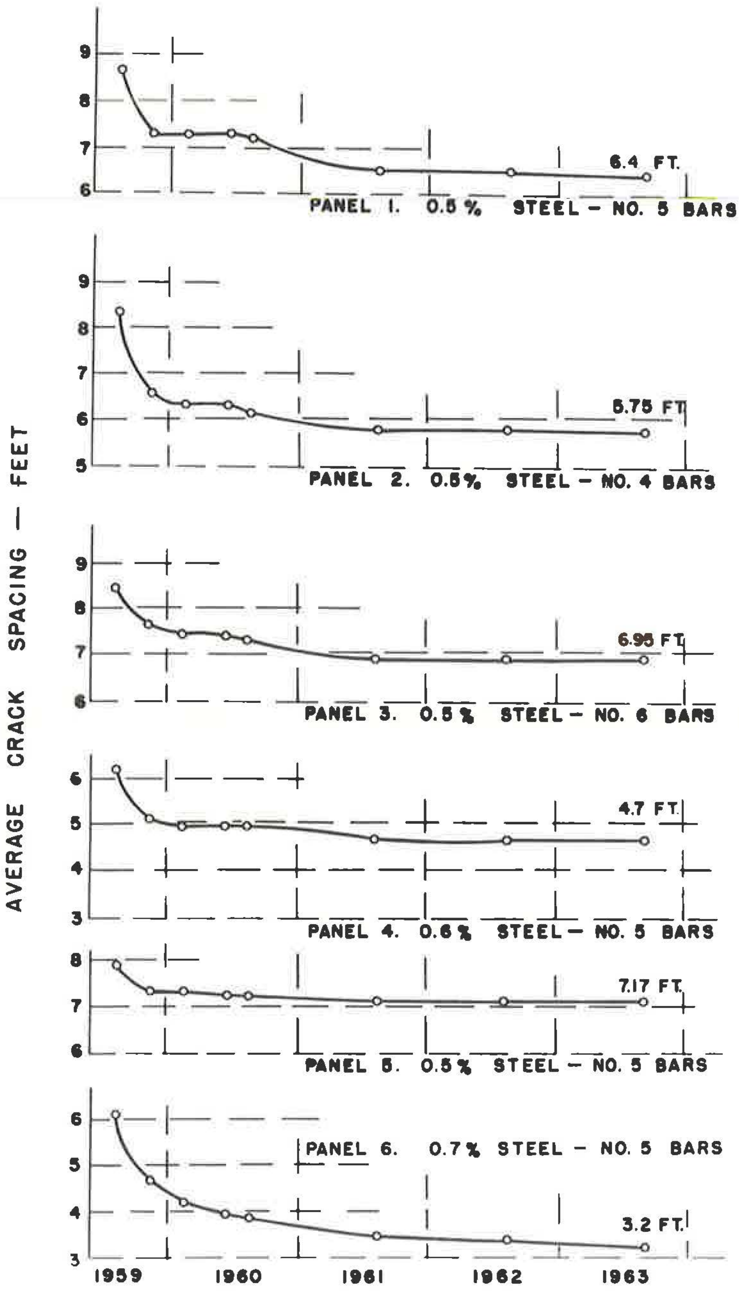

Figure 6. Variation in average crack spacing with pavement age.

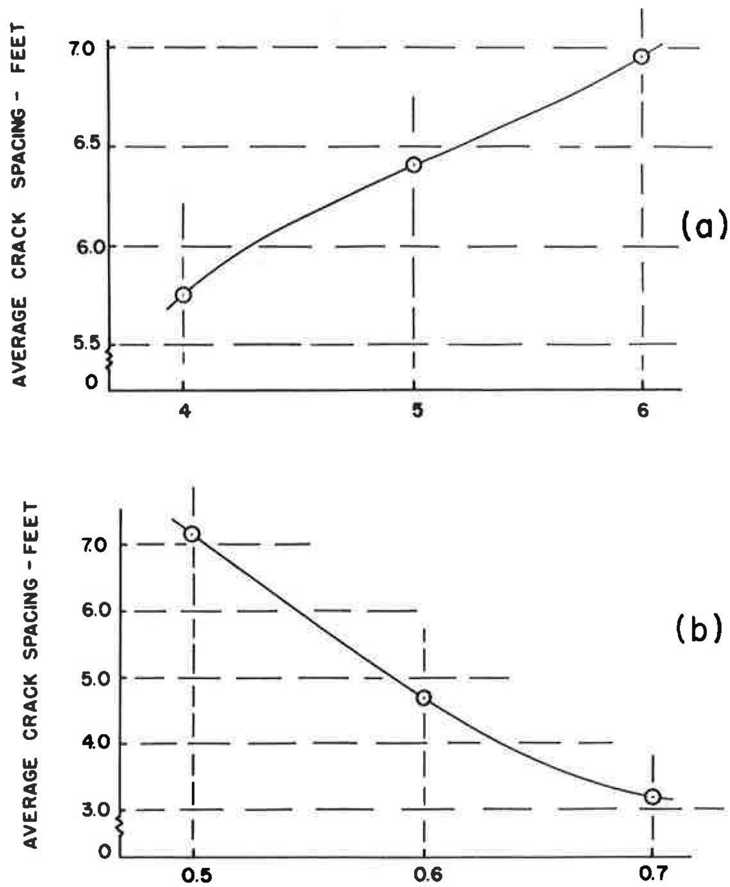

Figure 7. Effect of (a) bar size and (b) percent steel reinforcement on average crack spacing.

DAILY STEEL STRAIN CYCLES

The daily cycles are defined by four readings for each panel taken over a 24-hr period. During this period, it seems logical to assume that the steel strains will be affected only by temperature change since any change in the moisture content of the pavement would be small. Ideally, then, if steel strains were plotted against pavement temperature for a 24-hr period, the points should form a straight line.

Average steel strain (gages 5 through 12) at the induced crack was plotted as a function of average slab temperature (Figs. 8, 9, 10) for panels 4, 5, and 6. These three panels were chosen because of their relatively consistent behavior and their variation in steel content. In most cases the four points observed in the daily cycle fall very close to a straight line, fitted to the points with a small amount of error. In cases where the points were widely scattered, two methods were used. If three points formed a straight line and a fourth fell wide of this, then the line was drawn through these three points, omitting the fourth. If the four points were widely scattered with no apparent order, then a straight line was averaged through the points with a slope that seemed consistent with the slopes of preceding and following daily cycles. The observed points fit well to a straight line in seven of ten cases.

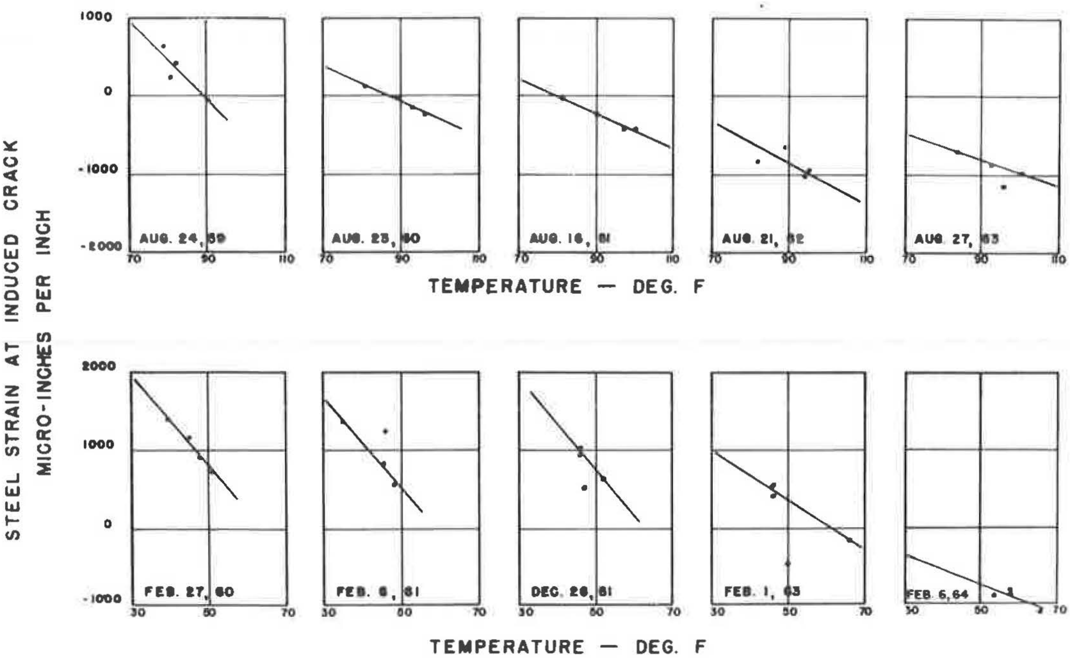

Figure 8. Daily strain-temperature cycle at 6-mo. intervals, panel 4.

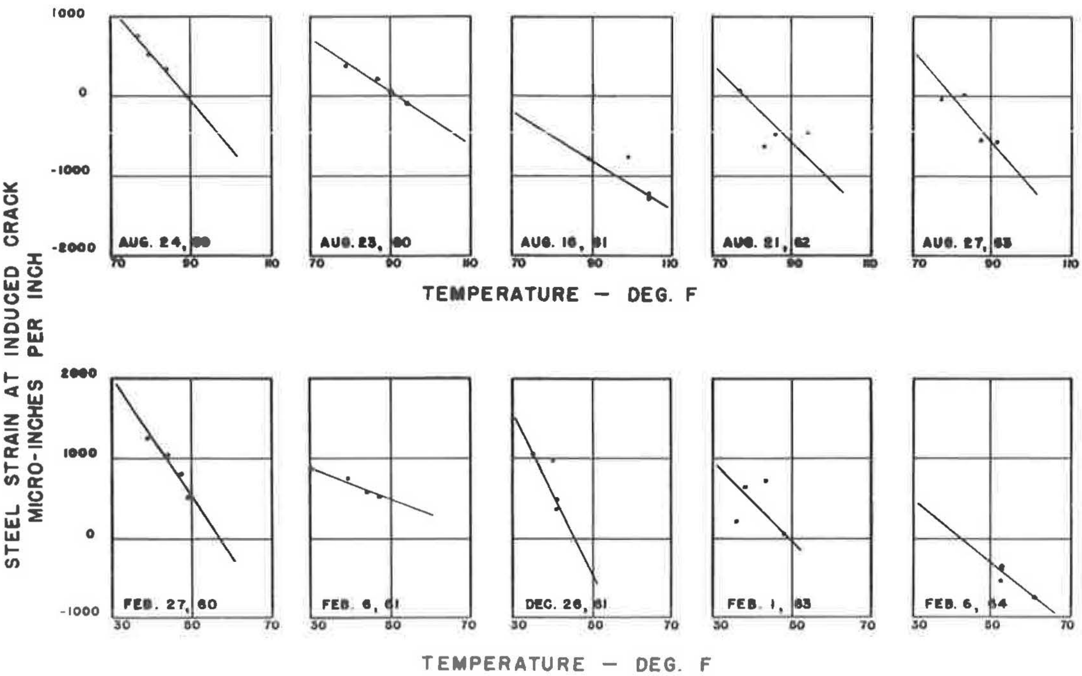

Figure 9. Daily strain-temperature cycle at 6-mo. intervals, panel 5.

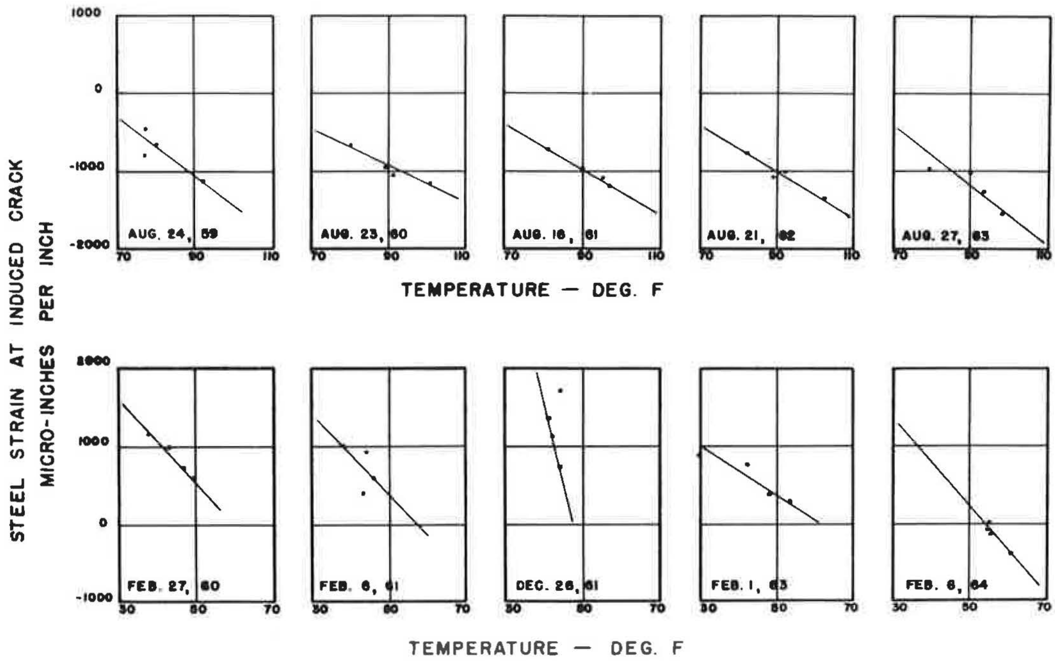

Figure 10. Daily strain-temperature cycle at 6-mo. intervals, panel 6.

Panel 5, with 0.5 percent reinforcement, No. 5 bars, has the largest scattering of points of the three panels (Fig. 9). The steel strain values of the first winter daily cycle, taken February 27 and 28, 1960, fall in a very nearly straight line. The second set of winter readings, taken February 6 and 7, 1961, do not fall on a straight line; however, an average straight line can be drawn through the points without much difficulty. After this time, the points become increasingly widely scattered. In panel 5 the variation of steel strain with temperature goes progressively from a straight-line variation to a four-point variation having no apparent sequential order. These erratic results probably are due to pavement failures that occurred or were in the process of occurring during the winter of 1961-1962.

Panel 4 contains 0.6 percent reinforcement composed of No. 5 bars. The steel strains at the induced crack, when plotted against the pavement temperatures (Fig. 8), fall very close to a straight line, with three exceptions. As in panel 5, the readings of the first winter daily cycle fall very close to a straight line. Three of the readings of the 1961, 1962, and 1963 winters fall on a straight line in each case. One point falls out of line with the others and was not considered when the straight line average was drawn. The slopes of the winter strain-temperature lines (Table 1) do not decrease for the first two winters. Only after the third winter does the slope of the daily cycle begin to decrease. Slopes of the daily cycle graphs became nearly constant for summer readings after 1-yr age.

The winter readings for panel 6, containing 0.7 percent steel, are perhaps the most consistent of the three panels under discussion. As in panels 4 and 5, the February 1960 readings fall on a straight line. The 1961 winter readings are widely scattered. The 1962 and 1963 readings, unlike those of panels 4 and 5, fall quite close to a straight line. The slope of the strain-temperature line decreases from winter 1960 to winter 1961. After this, the slope shows no apparent consistency. The sharp increase in slope of the winter 1961-1962 occurs for no apparent reason, since no breaks or extreme cracking appear in the test section, and this value must be considered as instrumental error.

TABLE 1

OBSERVED VALUES OF STEEL STRAIN AT CRACK PER DEGREE F AND OBSERVED CRACK SPACING

Date	Panel 5[a]		Panel 4[b]		Panel 6[c]	
	μ-in./in./Deg F	Crack Spacing (ft)	μ-in./in./Deg F	Crack Spacing (ft)	μ-in./in./Deg F	Crack Spacing (ft)
Aug. 1959	60	7.94	52	6.25	39	6.10
Feb. 1960	78	7.35	59	5.00	54	4.20
Aug. 1960	33	7.27	23	5.00	24	3.85
Feb. 1961	19	7.20	59	4.92	51	3.60
Aug. 1961	31	7.14	23	4.76	29	3.45
Dec. 1961	104	7.14	59	4.70	200	3.40
Aug. 1962	49	7.14	26	4.70	30	3.35
Feb. 1963	50	7.14	32	4.70	32	3.30
Aug. 1963	58	7.14	17	4.70	39	3.20
Feb. 1964	41	7.14	20	4.70	58	3.15

[a]0.5 percent No. 5 bars.
[b]0.6 percent No. 5 bars.
[c]0.7 percent No. 5 bars.

The observed slopes of the diagrams of steel strain at the crack vs temperature are given in Table 1. These strains, in micro-inches per inch per degree Fahrenheit, are shown, together with average crack spacing in the central 500 ft of each of the three sites. Steel strain at the crack per degree Fahrenheit varies from 17 to 78 μ-in./in./deg F, if the widely inconsistent values for December 1961 are discarded as erroneous readings. Change in stress per degree change in temperature, found by multiplying these strains by modulus of elasticity of the steel, ranges between 510 and 2, 340 psi/deg F.

In all three panels, the slopes of the winter strain-temperature lines are greater than the slopes of the summer strain-temperature lines in almost every case. This may indicate that the greater moisture content of the pavement during the winter affects the coefficients of expansion and contraction of the concrete (9).

There does not appear to be any regular change with age in these values of strain per degree temperature change. The irregular data in many instances of daily cycles and the lack of any long-term consistency or trends indicate that many factors affect the forces in the pavement. A simple relation between strain and temperature change does not hold at all times even over the period of a day. Long-term effects such as might be caused by changes in crack spacing, concrete properties and moisture content of the pavement are not great enough to affect the range of values of strain per degree temperature change in the daily cycle.

VARIATION OF CRACK OPENING AND STEEL STRAIN AT CRACK WITH AGE

Trend Lines

Crack openings and strains vary with temperature and with age (Figs. 3 and 4). To study the changes which occur with age, it is necessary to establish comparable strain and crack opening values that are independent of temperature of observation. This was done by fitting straight lines (called trend lines) to plots of strain at crack and crack opening as functions of temperature. The intercepts of these trend lines at 40 and 80 F were the normalized values of the two quantities.

Diagrams were plotted using only the February and August readings because the pavement contains maximum and minimum moisture, respectively, and minimum and maximum temperature, respectively, at these times in the yearly cycle. Although it

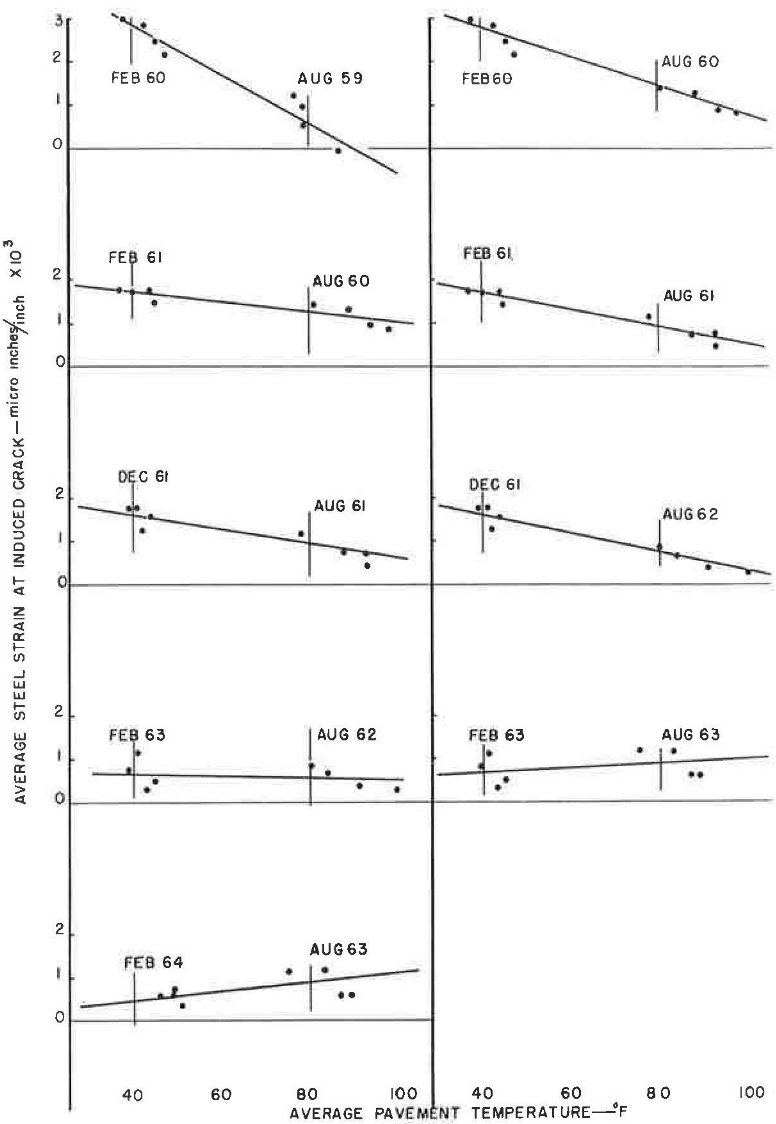

Figure 11. Steel strains at induced crack vs average pavement temperature, panel 1.

was not possible to eliminate the effects of the moisture in the pavement, these times were chosen so that any effects of moisture from one yearly cycle to the next would be approximately the same.

The data were plotted in two sets, from August to February, which will be called warm to cool trends, and from February to August, the cool to warm trends. From the fitted lines steel strain at the induced crack and induced crack opening values were read at slab temperature values of 40 and 80 F, thereby bringing the trend lines to a position where they may be studied for age and crack spacing effects at the same temperature values.

Four readings of induced crack opening or steel strain at the crack were taken over a daily cycle for each August or February. These values were then plotted against the pavement temperature at the time of each reading. One August and one February daily cycle, in each case, make up one trend line. For example, Figure 11 shows the steel

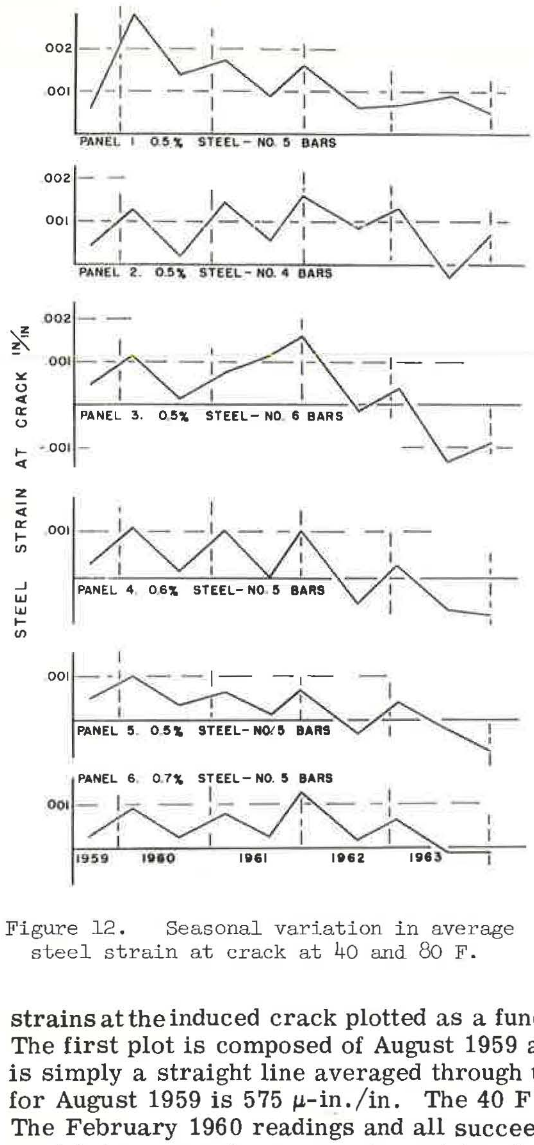

Figure 12. Seasonal variation in average steel strain at crack at 40 and 80 F.

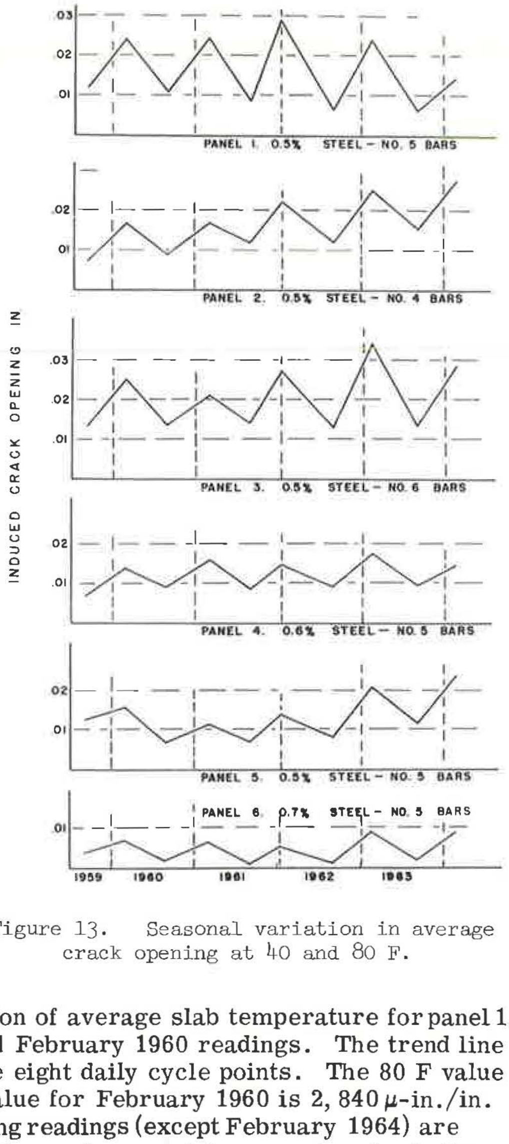

Figure 13. Seasonal variation in average crack opening at 40 and 80 F.

strains at the induced crack plotted as a function of average slab temperature for panel 1. The first plot is composed of August 1959 and February 1960 readings. The trend line is simply a straight line averaged through the eight daily cycle points. The 80 F value for August 1959 is 575 μ-in./in. The 40 F value for February 1960 is 2, 840 μ-in./in. The February 1960 readings and all succeeding readings (except February 1964) are used twice, once for a warm to cool trend and once for a cool to warm trend. In the next plot (February 1960 to August 1960) the 40 F value for February 1960 is 2, 750 μ-in./in. These two 40 F values were averaged to give one February 1960 reading of 2, 795 μ-in./in. The succeeding 40 and 80 F values are all averaged to yield one winter or summer value. This procedure was used for all panels.

Annual Cycles

The steel strain and crack opening values at 40 and 80 F are plotted against pavement age in Figures 12 and 13. The resulting steel strain and crack opening curves are saw-toothed with peaks and valleys at 40 and 80 F, respectively.

The steel in panel 1, Figure 12, was strained beyond the yield point during the first winter. Since then both the summer and winter steel strains at the crack have been decreasing. The difference in strains between the 40 and 80 F values from one February to August or August to February to the next is decreasing. The slightly downward slope of the August 1962 to February 1963 trend line and the upward slope of the February to August 1963 trend line are contrary to what might be expected.

During July 1963, a failure in the pavement at a reinforcement overlap approximately 27 ft south of the induced crack was repaired. There was noted extreme cracking with a hole in the pavement. This shattered area was approximately 1 ft wide and extended about 6 ft in toward the centerline from the shoulder edge of the southbound lane. The depth of the hole extended below the reinforcement so that both ends of the lap were plainly visible. A break of this sort could very well result in loss of continuity in the steel and its strain readings and account for the two odd trend lines. The beginning effects of the break may be noted in the February 1963 strain readings, since it appears that continuity was lost during the autumn of 1962. The strain of about 650 μ-in./in. shown in August 1962 and February 1963 is, most likely, the permanent set remaining in the unstressed steel at this time. This set is consistent with the amount of strain beyond the yield point observed during the 1959-1960 winter.

The break in the pavement of panel 1 did not appear to affect the crack openings as much as it did the steel strain. Its effects, however, may be seen on careful examination of Figure 13. The 40 F crack opening appeared to be on an upward trend until February 1963, at which time it dropped to a value equivalent to earlier values. The 80 F values seemed to be decreasing steadily until August 1963, when the value was equal to the previous summer reading and did not follow the foregoing trend.

Site 2 has no repaired places near the induced crack. The steel strains at the crack at 40 F in this panel remain essentially constant with strain differences, becoming steadily less as the age of the pavement increases (Fig. 12). The August 1963 and February 1964 values, however, show very marked decreases in strain. At this time, these decreases are unexplained; however, they may be the advanced effects of a break near the induced crack that is not yet visible on the surface. Both the 40 and the 80 F crack-opening values are increasing. The result is an upward trend with a slightly increasing seasonal difference in crack opening.

The steel strain in panel 3 is quite erratic, but, in general, is on a downward trend (Fig. 12). There have been two pavement repairs near the induced crack, as described previously. The patches are relatively small and no cracking has occurred in either patch. These breaks and the resulting loss of continuity may explain the upward slope of the February to August 1961 trend and the extreme plunge downward of the succeeding cool to warm trends.

Again, the break has had little effect on the opening of the induced crack (Fig. 13). The 40 F crack opening during the winter of 1960-1961 is much lower than previous and later readings. However, the 40 F crack opening and the crack opening seasonal difference are increasing, resulting in an upward trend.

The steel strains at the crack in panel 4 show a steady downward trend (Fig. 12). Site 4 has no repaired places or wide crack openings near the induced crack. There is, however, a natural crack 4 ft south of the induced crack. This crack occurred during the summer of 1959 and shows its presence in the readings of the two gages located 4 ft from the induced crack.

The crack openings for this panel are steadily increasing with a slightly increasing seasonal difference in the average opening (Fig. 13). The strains at the induced crack seem to be unaffected by the nearby crack.

Site 5 has two patches near the induced crack. These patches in the roadway of site 5 acted as an integral part of the continuously-reinforced pavement, in that transverse cracks have formed in them. The steel strains in panel 5 appear to be leveling off (Fig. 12). The low 40 and 80 F strains during the summer of 1962 and succeeding years may have been caused by the breaks in the pavement and the loss in initial tension. The crack opening values appear to be on an increasing trend with the crack opening seasonal difference increasing also (Fig. 13).

Panel 6, with 0.7 percent steel, is the best performing of the six panels. There are no repaired areas or evidence of breaks. The average crack spacing is quite close, yet the cracks are evenly spaced. The 40 F strains at the crack and the 80 F strains are both decreasing with age (Fig. 12). The seasonal difference in strains at the crack is decreasing also. The crack opening shows an upward trend (Fig. 13), with both 40 and 80 F crack-opening values increasing, but not by an excessive amount.

The steel compression strains and the increase in the crack openings are contrary to expectations. The compression strain readings may be due to creep under tensile stress of the adhesive used to fasten the SR-4 strain gages to the reinforcement. If this stress, which occurs during the summer, were removed, the gages could show compression, while the steel would actually be in tension.

Pavement age and accompanying influences have an effect on the steel strains at the induced crack. The annual cycles have continued but in general show a decreasing trend. This indicates that the longitudinal forces in the pavement are gradually decreasing from their high initial tension. The range, or difference between the summer and winter readings, is decreasing and the 40 F steel strain readings at the induced crack are also decreasing. This would indicate that the behavior of the pavement is becoming less dependent on stresses due to initial shrinkage and crack spacing and is controlled more by concrete moisture content and yearly temperature difference.

The 40 F crack openings in all the panels, except panel 1, are increasing, as indicated by the comparisons in Table 2.

Panels 4 and 6, with their greater amounts of reinforcing steel, are the best performing of the six panels. Panel 5 with 0.5 percent steel, though its crack openings are somewhat wider than those of the other two panels, also appears to be performing well.

As the pavement ages, a loss of bond between the steel and the concrete at the cracks would allow the same amount of deformation to occur in a greater length of steel and thereby resulting in smaller steel strains, while the crack opening remains essentially constant or increases.

STRAIN VARIATION ALONG REINFORCEMENT

In each panel, 24 gages are attached to the reinforcement. Eight of the gages measure the strain at the crack. The steel strains at 6, 12, 24, and 48 in. from the induced crack are measured by four gages at each distance from the crack (Fig. 2).

Figures 14 and 15 show stress variations along the reinforcement over typical daily cycles for summer and winter readings. The strain variation plots are similar in shape. In general, the strains at the induced crack in the daily cycle have maximum values at the early morning readings when pavement temperatures are at a minimum. At colder temperatures early in the pavement life, the entire pavement is in tension with sharp peak values of stress at the crack (Fig. 14a). This peak drops off within 6 to 12 in. from the crack, and stresses in the blocks of pavement between transverse cracks are relatively uniform. Later in the pavement life, the stresses are lower throughout (Fig. 14b), although the same pattern occurs through the daily cycle. The most probable cause of the reduced stresses is the larger number of cracks at the later age. At warmer periods in the daily cycle of temperatures, the peak stress at the crack is blunted, even though stresses in regions away from the crack are reduced very little. Summer temperatures cause stresses away from the crack to be greatly reduced from winter values, even to the extent of becoming compressive at high temperatures and later ages (Fig. 15). A slight tendency for peak stresses at the crack to occur is shown at low temperatures in each daily cycle. There were more irregularities in the data for the small readings in the summer.

The steel strains at the crack and at 6 in. from the crack are plotted in Figure 16. Values for the five winters of observation are shown, since the higher winter stresses are of greatest interest. Pavement temperature range for each date is stated. Despite a few obvious irregularities, these plots are indicative of behavior at the time of high stresses. These curves confirm the comments previously made in discussing the variations of steel strain at the crack with age. Strains at 6 in. from the crack do not, in general, show much change until the fourth or fifth winter, when decreasing values occur.

The pavement temperatures recorded during February 1964 were approximately 10 deg higher in all the panels than those recorded for previous winter readings. These relatively high winter temperatures when coupled with the high moisture content of the pavement, experienced during the winter, may explain the summer-like strain patterns for February 1964.

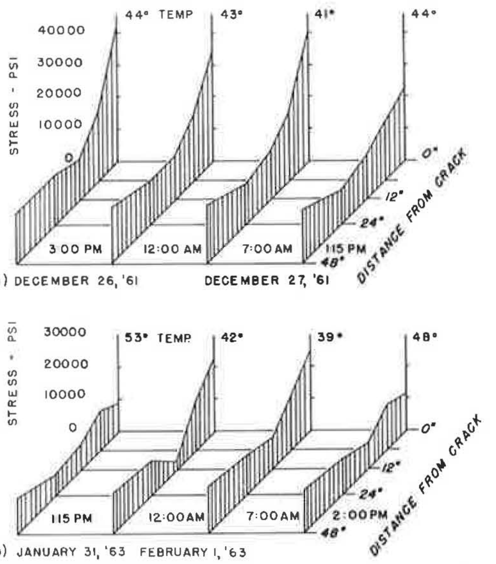

Figure 14. Stress variation along reinforcement over winter daily cycle, panel 6.

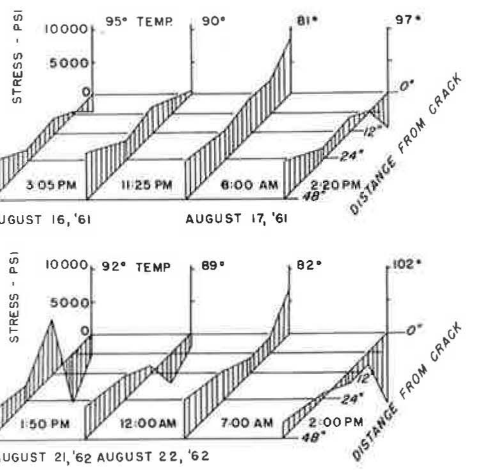

Figure 15. Stress variation along reinforcement over summer daily cycle, panel 6.

A study of the strains along the reinforcement during the winter readings will give some idea of the bond action at and away from the induced crack. During the early life of the pavement, the largest steel strain and consequently the largest stress takes place at the crack. The steel strains decrease for a short distance away from the crack because of the transfer of stress to the concrete by means of bond between the steel and the concrete. Beyond this distance of 6 to 12 in. the steel strains remain essentially constant.

The difference between stress at the crack and stress 6 in. from the crack, times bar area is the difference in force at these sections. Dividing by the surface area of bar in this length gives the average bond stress. Using the greatest strain differences from Figure 16, the highest observed bond stresses are 970 psi in panel 3, Dec. 1961; 860 psi in panel 1, Feb. 1960; around 500 psi in panels 2, 4, and 6; and 390 in panel 5.

Concrete creep may be responsible for the seemingly odd, yet relatively consistent, strain readings along the reinforcement. Some summer readings show a very small

TABLE 2

STEEL STRAINS AT INDUCED CRACK AT 40 F DURING 5 CONSECUTIVE WINTERS

Panel No.	Strains (μ-in./in.)				
	1959-1960	1960-1961	1961-1962[a]	1962-1963	1963-1964
1	2,795	1,700	1,600	668	470
2	1,300	1,450	1,600	1,338	750
3	1,168	720	1,625	400	-880
4	1,200	1,138	1,010	310	-880
5	1,005	695	710	425	-750
6	970	800	1,300	665	-100

[a]The readings taken for the winter of 1961-1962 were consistently high for all panels, possibly due, in part, to having been taken not in February but in December.

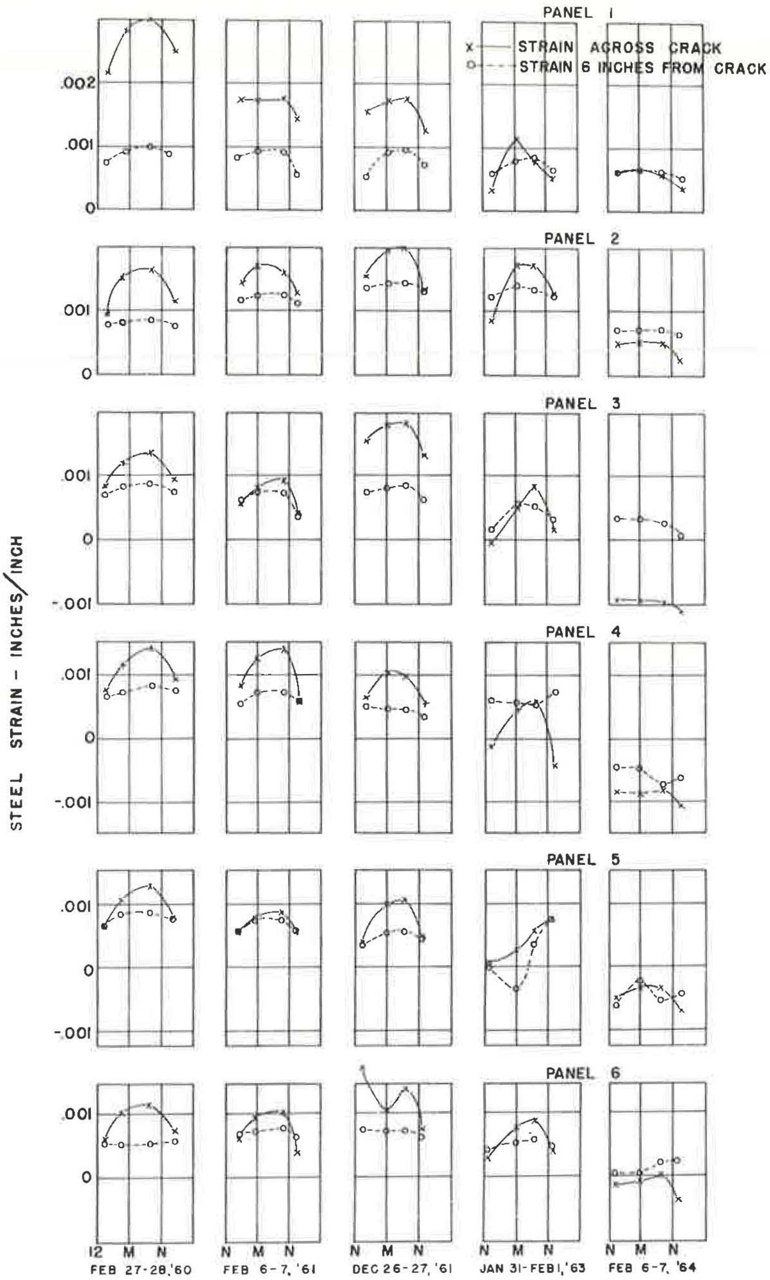

Figure 16. Steel strains at crack and at 6 in. from crack.

tensile strain or a compressive strain at the crack, whereas farther away from the crack the gages show a relatively high tensile strain. A possible explanation of this phenomenon is as follows.

During the winter months, the reinforcing steel and the concrete are subjected to high tensile stresses and would be deformed as shown in Figure 17. The stress values would vary over the daily cycle; however, all the stresses would be tensile. A core of concrete around the reinforcing bar would be subjected to relatively high tensile stresses, whereas farther away from the bar these stresses would be small. Creep in concrete

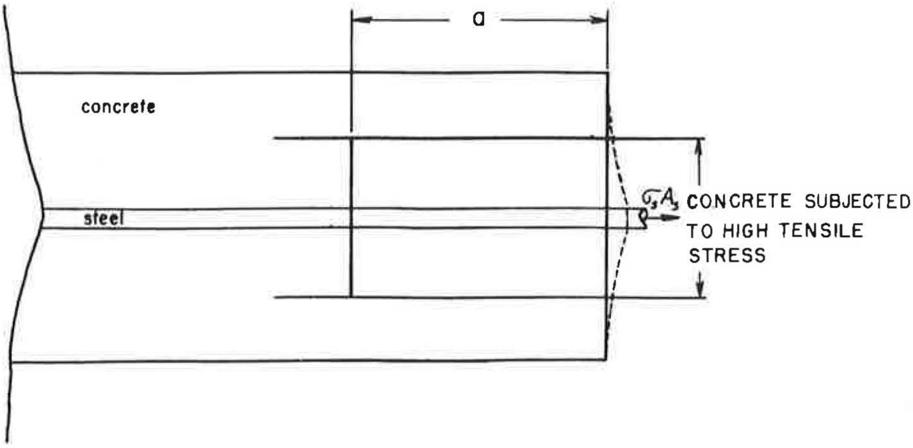

Figure 17. Deformed section.

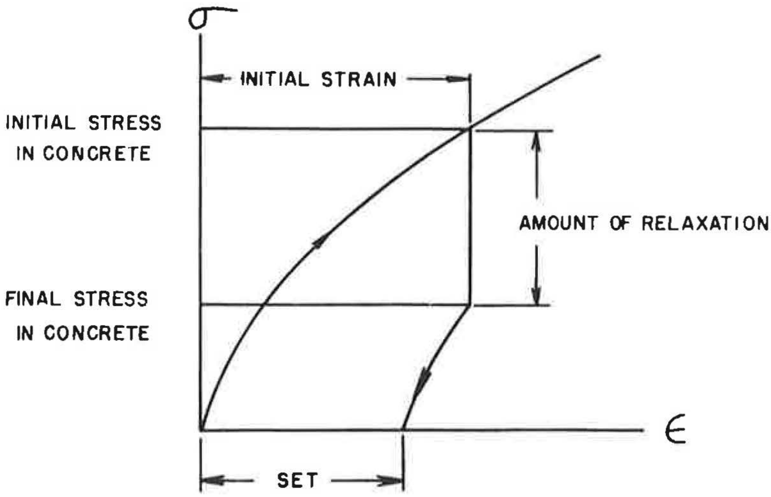

Figure 18. Relaxation of concrete stress.

is a relatively rapid phenomenon, in that approximately one-fourth to three-fourths of the ultimate creep under a load may be expected to take place within 1 to 6 months (2).

During the winter period, creep of the highly strained concrete around the reinforcement would occur, resulting in a permanent set with a corresponding relaxation of stress (Fig. 18).

As the temperature of the pavement increases during the spring and summer, the tensile stresses at the crack would decrease. The creep of the concrete while in tension would cause the concrete to act as a restraining collar on the reinforcing bars. Although the steel stress at the crack may be zero or compressive in nature, the concrete would hold the steel in tension at regions away from the crack. The process of creep would be reversed and the concrete would be in compression. The ultimate amount of creep for concrete in tension or compression is approximately the same. The rate of creep, however, is greater during the early stages if the concrete is in tension (1). If the concrete is considered to be subjected to high tensile stresses for three winter months of the year and to be subjected to relatively high compressive stresses during the three summer months of the year, it becomes evident that the tensile stresses caused by initial shrinkage of the concrete will eventually be eliminated and the stresses within the pavement will be due to temperature and moisture change only.

SUMMARY AND CONCLUSIONS

The Maryland experimental sections of continuously-reinforced concrete highway pavement have been under observation for $4\frac{1}{2}$ years. Several significant trends have been shown during this time and significant conclusions can be drawn.

The strain at the crack of the reinforcing steel and the opening of the transverse cracks vary inversely with the temperature of the pavement. The amount of steel contained within the continuously-reinforced section has a definite effect on the magnitude of the steel strains and the crack opening. The highest steel strains and the largest crack openings occurred in the sections of pavement containing the smallest amount (0.5 percent) of reinforcement. The size of the reinforcement (No. 4, 5, or 6) does not appear to have any significant effect on the steel strains, since the strains of all the sections containing 0.5 percent steel were quite high, regardless of bar size. The bar size, however, does seem to affect the width of the transverse crack opening. The section containing the smallest diameter bar experienced the smallest crack opening.

The uniformity of crack opening throughout the pavement is dependent on both percentage of reinforcement and reinforcing bar size. The crack openings throughout the central portion of the sections of continuously-reinforced pavement are more uniform in the sites containing 0.6 and 0.7 percent reinforcement. In sections containing the same amount of reinforcing steel, the crack opening throughout the central portion is most uniform in the section containing the smallest bar size, No. 4.

The spacing of transverse cracks affects the steel strains and the width of the crack opening. The smaller steel strains and smaller crack openings occur in the sites with the smaller average crack spacing. Average crack spacing is a function of the two controlled variables. It varies inversely with the percentage of reinforcement and directly with the bar size. The transverse cracks are more evenly distributed throughout the central portion of the pavement in the sections containing the higher percentages of steel. The size of the bar does not appear to affect the uniformity of crack spacing.

In all the sections of experimental pavement, certain trends occur as the age of the pavement increases. After the pavement has been exposed to an extended period of low temperature, the formation of new cracks is reduced quite drastically, and after a $4\frac{1}{2}$-yr period, the formation of new cracks has virtually ceased. The widths of the crack openings increased and the maximum steel strain decreased as the pavement aged. The rate of increase or decrease is greater in sections with the smaller percentages of reinforcement.

The decrease in temperature from the elevated value associated with hydration of the cement and the drying shrinkage attendant on curing cause initial cracking and high initial tensile forces in the pavement. In a climate such as Maryland's, additional cracking and high stresses occur at times of low temperatures. After two or three winters, there is little additional cracking, and stresses are less than they were at earlier ages. Stresses continue to decrease after the crack spacing has become steady. This is evidence of creep in concrete reducing the initial tension. As the tension is lost, compressive stresses at times of warm temperatures may be expected to increase with age.

In general, sites 4 and 6, containing 0.6 and 0.7 percent reinforcement, respectively, are the best performing sections. The steel strains and crack openings of site 5, containing 0.5 percent steel of No. 5 bars, however, were not excessive. This may very well be due to the fact that these three sites were placed and cured under nearly ideal weather conditions.

The experimental aspects of this construction led to the formation of some wide cracks requiring repair early in the pavement life. The need for repairs within the central 500 ft of the sites has been influenced by percent reinforcement and bar size. Three of the four sites reinforced by 0.5 percent steel have undergone some sort of repair work. Of these four sites only site 2, reinforced by No. 4 bars, has experienced no repairs within the central 500 ft of the section. Sites 4 and 6 containing 0.6 and 0.7

percent steel, respectively, have had no areas within the central 500 ft requiring repair. The data for sites 2, 4 and 6 thus indicate behavior of pavement in which continuity has been maintained. The loss in continuity in panels 1, 3 and 5 resulted in loss of the initial tension. Continuity was restored by the repairs, however, and daily and annual cyclic stress fluctuations resumed, but over a different range of stresses.

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