REPORT ON TEN-YEAR SERVICE OF EXPERIMENTAL CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN MICHIGAN

TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

REPORT ON TEN-YEAR SERVICE OF EXPERIMENTAL CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN MICHIGAN

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PREFACE

This interim report represents a study of the behavioral characteristics of an experimental continuously reinforced concrete pavement during its first ten years of life. The characteristics studied include absolute pavement displacement, crack formation, surface crack width variation, reinforcement stresses, surface roughness, and reinforcement corrosion. Also included are the load deflection behavior after one year and 4-1/2 years of service, air temperature variation, and traffic volume to which the pavement was subjected, as well as a discussion of the causes and repair procedures developed for failures which occurred during this period. Initial costs of this and other continuously reinforced pavements built during the past ten years are compared with standard jointed pavement costs during the same period and in the same construction zones.

Since this study is continuing, and further observations of the pavement as well as other continuously reinforced pavements are planned, especially in the areas of progressive pavement growth and reinforcement corrosion, no final conclusions and recommendations are presented at this time. Trends, results, and observations for the individual characteristics studied are presented in respective chapters in this report.

INTRODUCTION

In 1958 the Michigan Department of State Highways, with the approval of the Federal Highway Administration, constructed an experimental continuously reinforced concrete pavement for the purpose of studying its performance and cost as compared with Michigan's conventional concrete pavement. An earlier report covered the design, construction, instrumentation, and scope of study, in some detail (1). A summary of pertinent portions of this report is presented here for the reader's convenience.

The experimental pavement is part of Interstate 96, located between Portland Rd and M 66 approximately 23 miles northwest of Lansing. It consists of a 4.14-mile portion of the westbound roadway and a 5.09-mile length of the eastbound roadway.

The westbound roadway is continuously reinforced throughout the length of the project with plain welded wire mesh in the eastern 10,331 ft and deformed bar mat in the remaining 10,530 ft of the western end. On the eastbound roadway the eastern 10,550 ft is reinforced with deformed bar mat and the western 10,557 ft contains plain welded wire mesh. The center portion of the eastbound roadway (3,804 ft) is standard reinforced pavement. A 493-ft standard reinforced relief section is located at each end of the continuously reinforced sections.

The deformed bar mats were 6 ft 2 in. wide and 16 ft long with 11 No. 5 bars in the longitudinal direction and 7 No. 3 bars in the transverse direction. The longitudinal bars were spaced 6 in. center to center. The first transverse bar was placed 1 ft 1 in. from the mat ends, the second bar at each end was spaced on 2 ft 5-in. centers and the remaining three bars spaced at 2 ft 3 in. center to center. Mats of the plain welded wire fabric were 11 ft 6 in. wide and 12 ft long. The longitudinal reinforcement consisted of 46 No. 5/0 gage wires, spaced 3 in. on centers, except for three spaces in the center being 4 in. In the transverse direction, 12 No. 1 gage wires were spaced 12 in. on centers. The standard reinforcement mat size was 11 ft 6 in. wide and 10 ft long. In the longitudinal direction 24 No. 2/0 gage wires were spaced on 6-in. centers and, transversely, 10 No. 4 gage wires were spaced on 12-in. centers. The steel percentage provided by each type of reinforcement was 0.59, 0.60, and 0.16 for the deformed bar mats, plain welded wire mesh, and the standard mesh, respectively. The physical properties of the reinforcement, as determined from tests on representative samples of the longitudinal members, are given in Table 1.

Each roadway pavement is 24 ft wide, divided into two 12-ft lanes by a 1/8 in. wide by 2-in. deep sawed centerline joint. Lane ties consisted of No. 4 deformed bars, 30 in. long and spaced at 40 in. All continuously reinforced sections are of 8-in. uniform thickness whereas the standard pavement section and the six relief sections are of 9-in. uniform thickness.

Construction joints in the continuously reinforced sections, in addition to the pavement reinforcement, contained 1-1/4-in. diameter steel dowels, 18 in. long, spaced on 12-in. centers. Load transfer at contraction joints in the standard pavement and at expansion joints in the relief sections was provided by installing steel dowels of the same size and spacing as used in the construction joints. Contraction joint spacing in the conventional pavement section is 99 ft. Each relief section consists of 11 1-in. expansion joints spaced alternately at 56 ft 3 in. and 42 ft 4 in.

TABLE 1
STEEL REINFORCEMENT PROPERTIES

Properties	Bar Mat	Plain Wire Mesh	Standard Mesh
Yield Strength,* psi	77,000	78,000	72,000
Ultimate Strength, psi	140,000	84,000	81,000
Breaking Strength, psi	127,000	56,000	54,000
Mod. of Elasticity, psi	30×10^6	30×10^6	29×10^6
Elongation, % (2"G. L.)	16	12.8	10
Average Diameter, in.	0.509	0.434	0.334

* Based on 0.2 percent offset

Paving operations began September 22, 1958 and were completed October 20, 1958. The concrete was poured in two lifts, the first being struck off 3 in. below the pavement surface with the reinforcement placed on top. The bar mats were lapped 13 in., with the ends of the longitudinal bars placed against the last transverse bar of the preceding mat. The plain welded wire mesh was lapped 12 in., so that the first transverse wire of the mat being laid rested behind the last transverse wire of the preceding mat. After the reinforcement was in position, the second lift was poured and the concrete consolidated and finished in the normal manner required for standard pavement.

The concrete mix for the project was designed for a constant cement content of 5-1/2 sacks per cu yd. Based on tests at the time of construction, the concrete had an average air content of 5.4 percent and an average slump of 2 in. Tests on sample beams, taken at approximately 600-ft intervals throughout the projects, showed an average modulus-of-rupture value of 590 and 760 psi at 7 and 28 days, respectively.

The entire pavement was placed on a 12-in. granular subbase overlying, in general, a Type A-4 clay subgrade.

TEMPERATURE VARIATIONS

The average air temperature for the time period (approximately 2-1/2 hr) required to conduct each quarterly performance survey is plotted in Figure 1. Yearly extreme high-low temperatures obtained from a U. S. Weather Bureau Station located about three miles north of the project are also shown in Figure 1. Based on the Weather Bureau data, the pavement has been subjected to annual temperature range variations from 105 F in 1960 to 122 F in 1963.

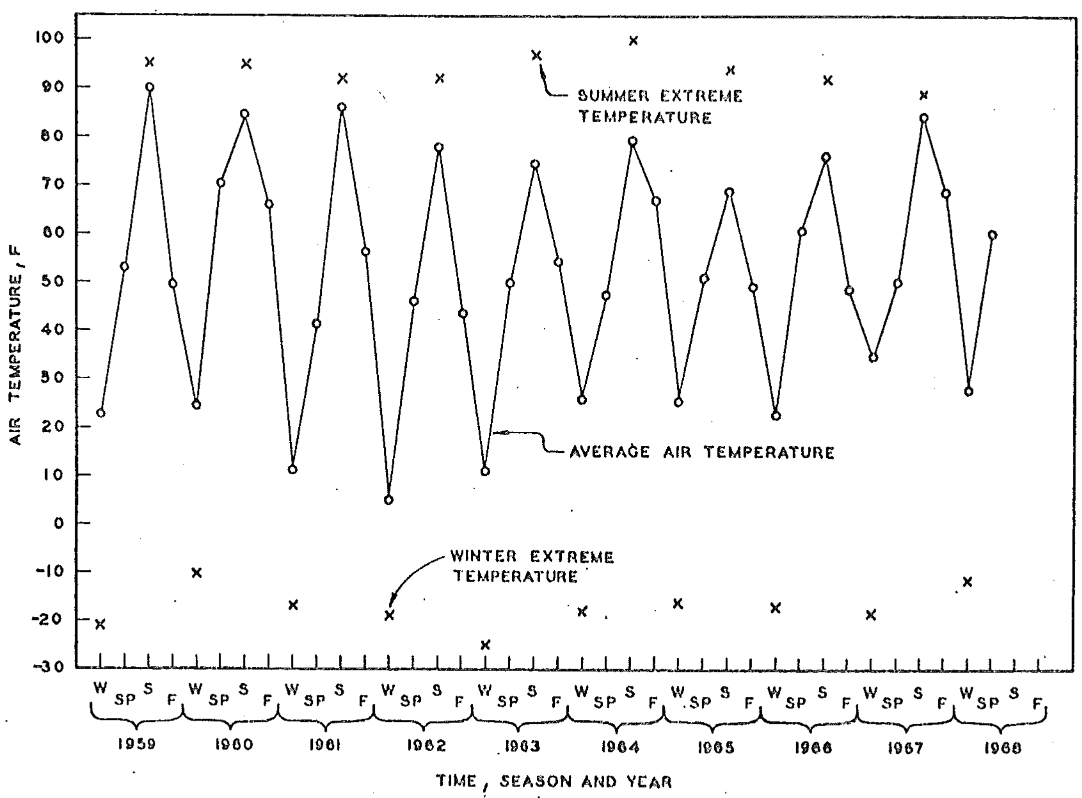


Figure 1. Average air temperatures for survey period and yearly extreme temperatures.

From the temperature plot it may be noted that the temperatures at which summer and winter measurements were taken varied from the extremes. For summer measurements the average variation was 14 F, with the range of variation being 5 F in 1959 to 23 F in 1963 below the extreme high temperature. In the winter the average temperature variation was 36 F, with a maximum variation of 44 F in 1964 and a minimum of 24 F in 1962 above the extreme low temperature. It should, therefore, be kept in mind that measurement values associated with the various performance factors under study and presented in this report represent the temperature condition at the time of survey and not the extremes to which the pavement has been exposed.

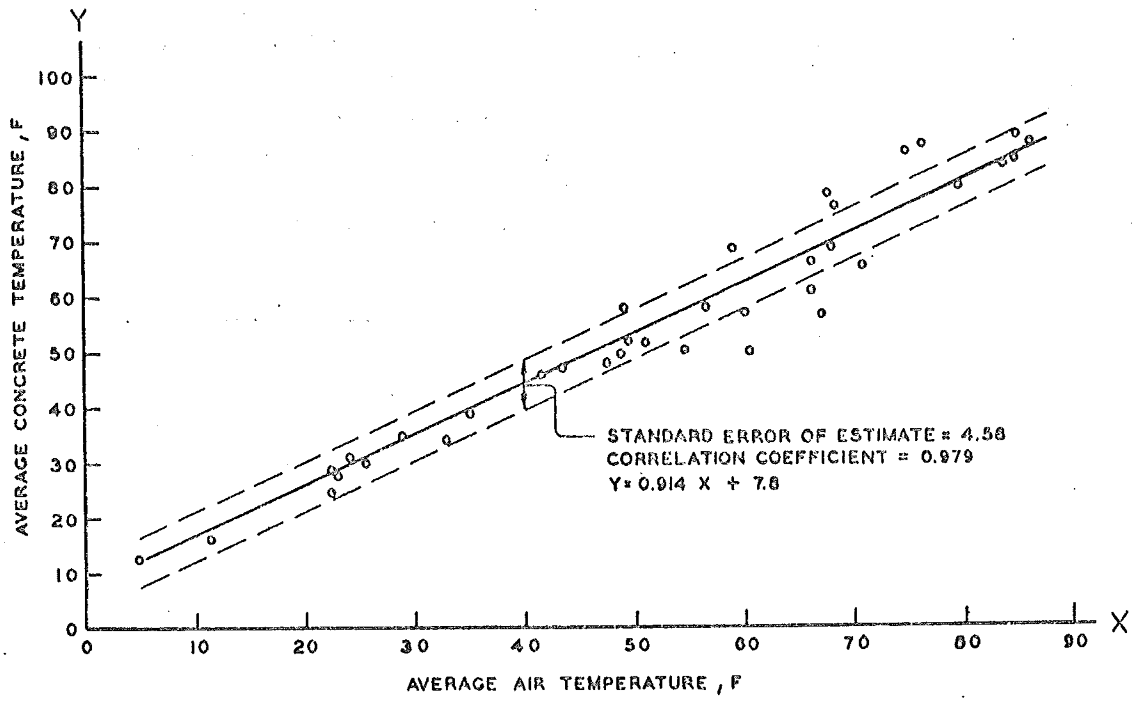


Figure 2. Correlation of average air and average slab temperatures.

In all cases the quarterly surveys were conducted from about 11:00 a.m. to 1:30 p.m. in order to minimize the effect of slab and air temperature differentials. Thirty-seven sets of slab and air temperature data were obtained during the survey periods and a regression line of the average slab and average air temperature is shown in Figure 2. Each point on the graph represents the average of 8 to 10 air temperature readings and 48 to 60 concrete temperature readings. As noted on the Figure, the standard error of estimate of the average concrete temperature with respect to the average air temperature was 4.58 degrees.

TRAFFIC VOLUME

Annual traffic volumes of commercial vehicles for each roadway for each of the ten years are shown in Table 2. The total annual traffic volume, including all vehicles and both roadways, is also given. These figures were obtained from the Department's Traffic Planning and Analysis group based on 8 and 24-hr sample periods and estimating factors for commercial traffic proportions.

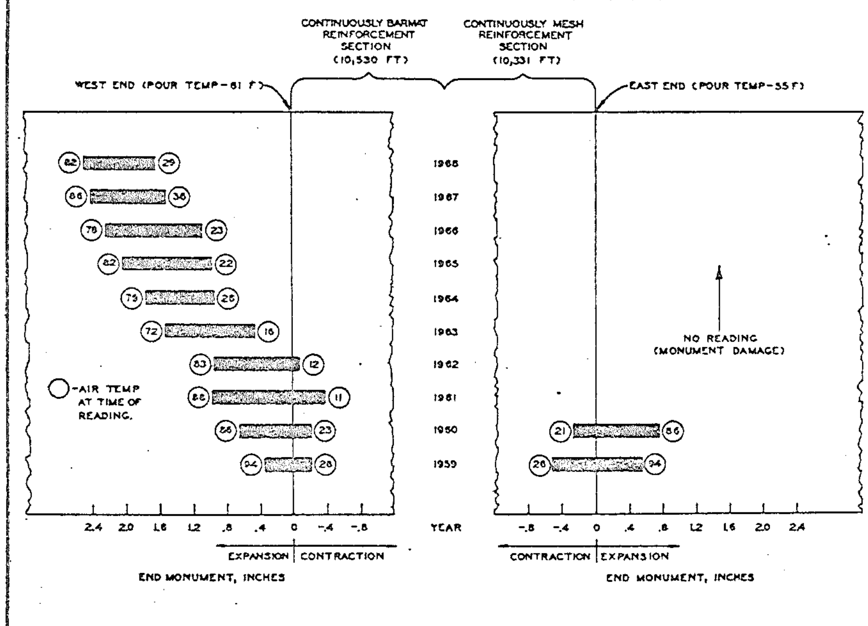
TABLE 2
ANNUAL NUMBER OF VEHICLES

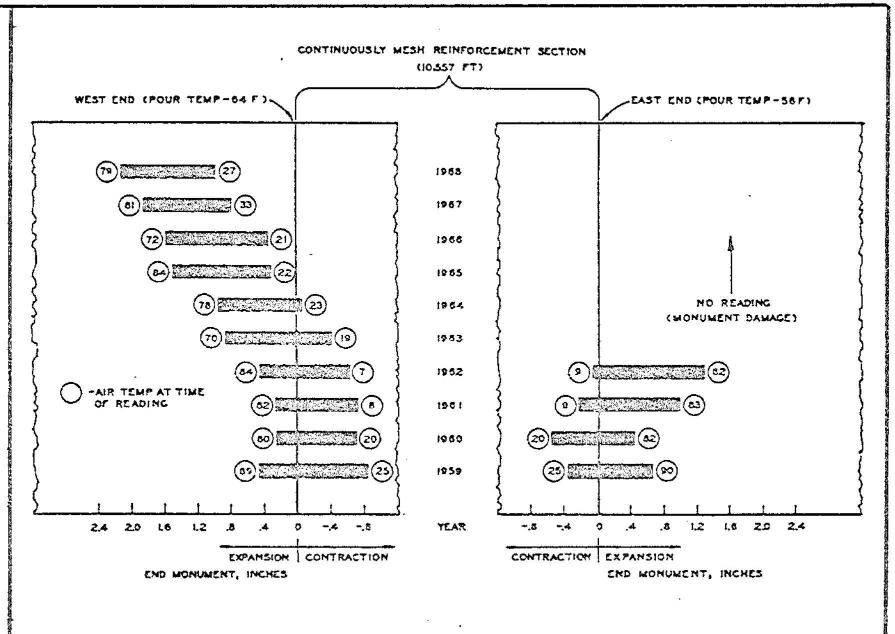
Year	Commercial Vehicles [1]		Total Vehicles [2]
	Westbound	Eastbound	
1959	185,000	160,000	2,080,000
1960	193,000	168,000	2,190,000
1961	201,000	176,000	2,300,000
1962	210,000	185,000	2,409,000
1963	214,000	199,000	2,640,000
1964	235,000	209,000	2,884,000
1965	311,000	255,000	2,774,000
1966	373,000	306,000	3,577,000
1967	373,000	306,000	3,468,000
1968	477,000	392,000	3,780,000
Total	2,772,000	2,356,000	28,102,000

(1) Includes pickup trucks
(2) Both roadways

ABSOLUTE PAVEMENT DISPLACEMENT

The relief sections, consisting of 11 1-in. wide expansion joints, were installed to provide for seasonal changes in length of the continuously reinforced sections as well as expansion space for permanent increase in length. To measure these longitudinal changes, a permanent reference monument was installed at the end of each continuously reinforced section at the time of construction. An initial zero reading at each monument was taken within 48 hours after pouring the end pavement portion. Subsequent measurements were made four times a year thereafter.





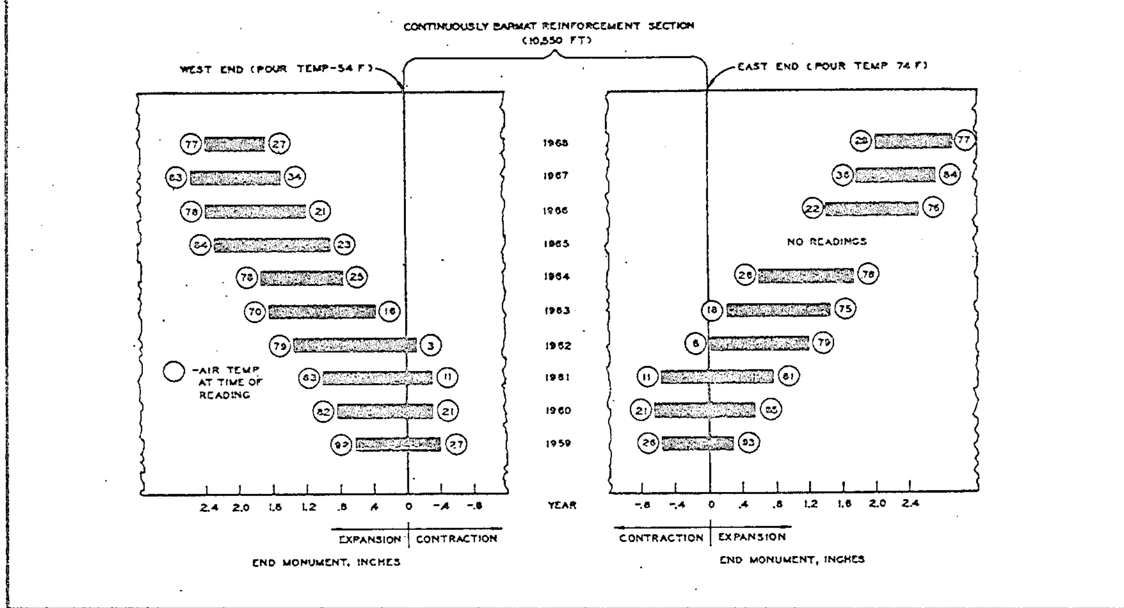


Figure 3 (top left). Yearly range of length changes of section containing both bar mat and mesh reinforcement.

Figure 4 (top right). Yearly range of length changes of section containing mesh reinforcement.

Figure 5 (bottom left). Yearly range of length changes of section containing bar mat reinforcement.

The measured range in length change from summer to winter at the six ends is shown in Figures 3, 4, and 5. Figure 3 illustrates the movement for the 20,861-ft section of the westbound roadway which contains bar mat reinforcement in the east half and plain mesh reinforcement in the west half. Figure 4 shows the movement for the 10,557-ft plain mesh reinforced section on the western portion of the eastbound roadway, and Figure 5 the movement of the 10,550-ft bar mat reinforced section on the eastern part of the eastbound roadway. Measurements at the east end of the mesh reinforced section on the eastbound roadway were discontinued in 1963 because damage to the monument by maintenance equipment prevented correct fastening of the measuring device. At the east end of the westbound roadway, measurements were discontinued in 1961 due to failure in the pavement continuity near the end. Readings at the east end of the bar mat reinforced section on the eastbound roadway were inadvertently omitted in 1965.

As anticipated, relatively large seasonal movements were measured at the free ends of the continuously reinforced sections. The average measured yearly range of change in length was 1.1 in. at each end or, in other words, the sections were on the average 2.2 in. longer in the summer than in the winter. The average recorded temperature change associated with this winter to summer increase in length was 60 F. The seasonal change in length of the four mile section on the westbound roadway was in the same general range as the two two-mile sections on the eastbound roadway, indicating that seasonal movement at the free ends is independent of length for long continuously reinforced sections.

As seen in Figures 3, 4, and 5 each section has increased permanently in length. On the basis of the winter measurements, an average growth of 2.1 in. has occurred at each section end from 1959 to 1968. The maximum growth, 2.6 in., occurred at the east end of the bar mat reinforced section on the eastbound roadway (Fig. 5) and the minimum 1.8 in., at the west end of the mesh reinforced section on the eastbound roadway (Fig. 4). Measurements at the free ends indicate that increase in length commenced the first year and progressively increased each year thereafter.

To obtain information on the movement of the slab in the end region of the continuously reinforced sections, a set of seven gage plugs were installed in the pavement 12 in. from the outside edge of the passing lane at each end during construction. The plugs were spaced 99 ft apart, the first plug being set at the end of the continuously reinforced pavement. Since the end plug was also used to measure the movement of the free end of the slab with respect to a fixed reference point, it was possible to obtain the absolute displacement of each plug as well as the increase in length of each successive 99-ft slab segment by measuring the distance between plugs.

The plug-to-plug distance was measured with an invar tape in combination with a small plastic plate and a set of vernier calipers with a 0.001 in. resolution. The plugs and the plastic plate have a conical drilled hole in the center of crosshairs scribed on their top surface, and the caliper legs are equipped with conical points. Spring scales were used to measure the tension in the tape when a measurement was taken. A 16-lb tape tension was applied during all measurements.

The procedure used in taking a measurement was as follows: The zero mark on the tape was aligned along the crosshairs of one plug and the 100 ft mark along the crosshairs of the plastic plate which was placed on the pavement in line with, and near the next plug location. The proper tension was applied, the correct alignment was established, and the plate held securely in position while a caliper reading of the distance between the plug and the 100-ft mark was taken.

It is obvious that this method of measuring changes in the lengths of 99-ft slab segments is not as precise as desired. To establish the precision of the readings, test measurements were conducted on 14 99-ft slab segments. Five measurements were taken on each slab segment using different personnel to tension the tape, hold the plastic plate, and take the caliper reading. The average standard deviation of these data was 0.043 in., with the expected maximum error for a 95 percent confidence level for one reading being 0.086 in. Since the test measurements were conducted in the winter under the adverse effect of cold weather conditions, it is expected that the error in warm weather readings would be less. Error in the measurement due to change in length of the tape as a result of temperature variations would also be present. The tape had a coefficient of linear expansion of 2.2×10^{-7} per unit length per degree F and was calibrated at 68 F. Based on the average winter temperature reading the corresponding error in the tape length would be 0.013 in. Although the measurements contain these errors, it is believed they are of sufficient accuracy to indicate the magnitude of the free end displacements.

The recorded yearly range in length change of the 99-ft slab segments at each of the six free ends is shown in Figures 6, 7, and 8. Figure 6 illustrates the displacement range for the westbound roadway, Figure 7 shows the range of the mesh reinforced section on the eastbound roadway, and Figure 8 shows the movements of the bar mat reinforced section on the eastbound roadway.

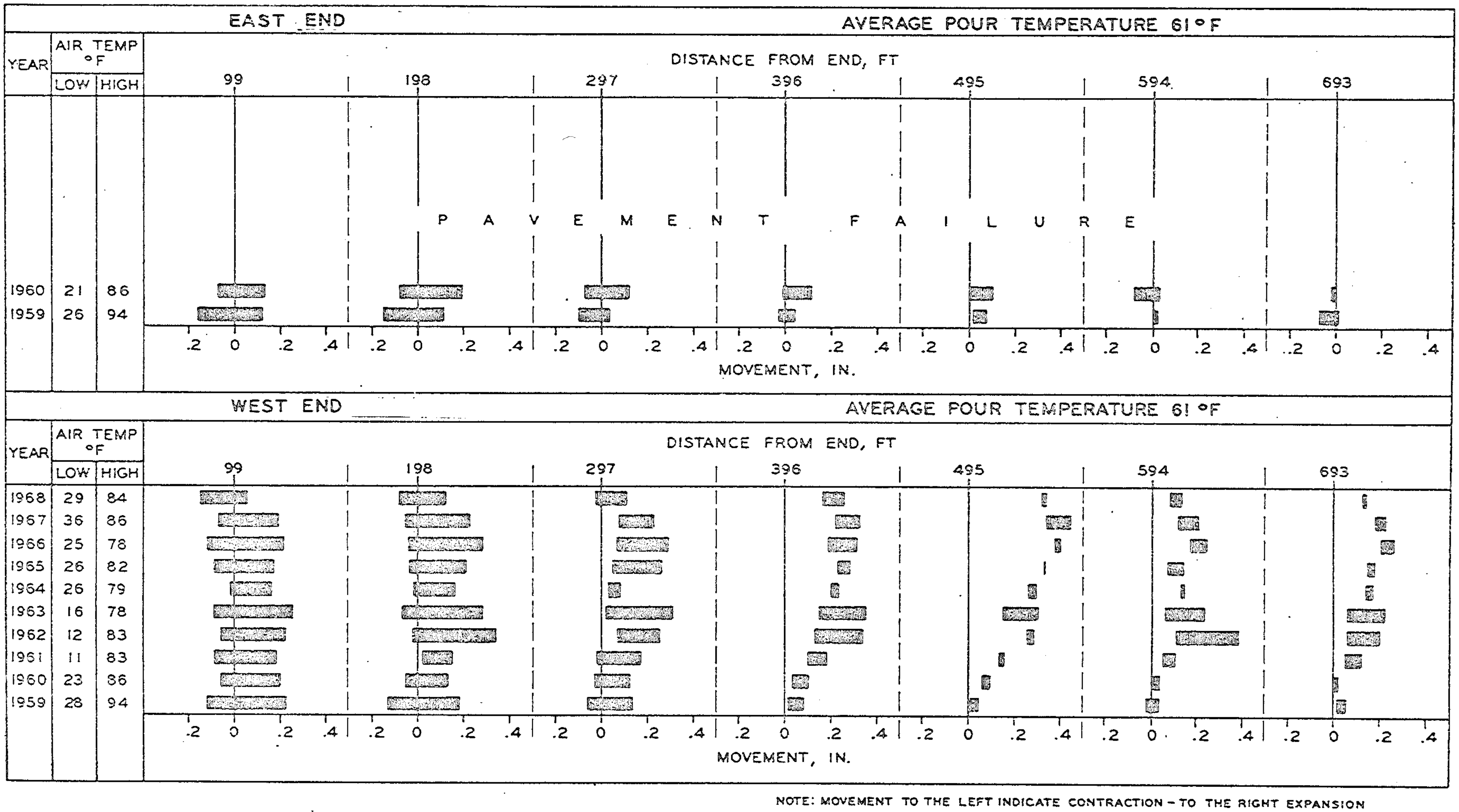
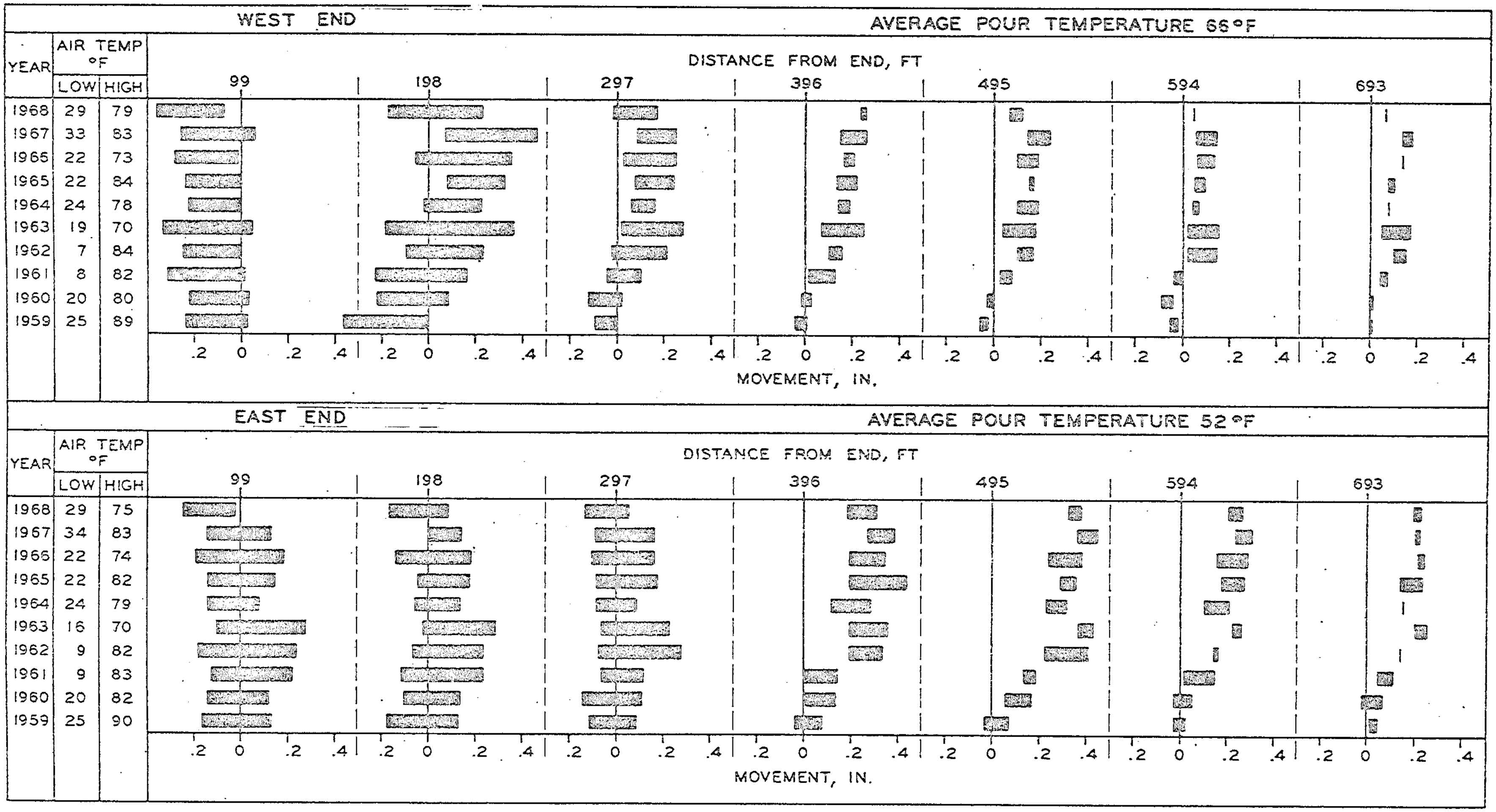


Figure 6. Yearly range in movement at points 99 to 693 ft away from free ends of westbound roadway. (Total length of section: 20,861 ft; west end 10,530 ft bar mat; east end 10,331 ft mesh).



NOTE: MOVEMENT TO THE LEFT INDICATE CONTRACTION – TO THE RIGHT EXPANSION

Figure 7. Yearly range in movement at points 99 to 693 ft from free ends of continuously reinforced mesh section on the eastbound roadway. (Total length of section: 10,557 ft).

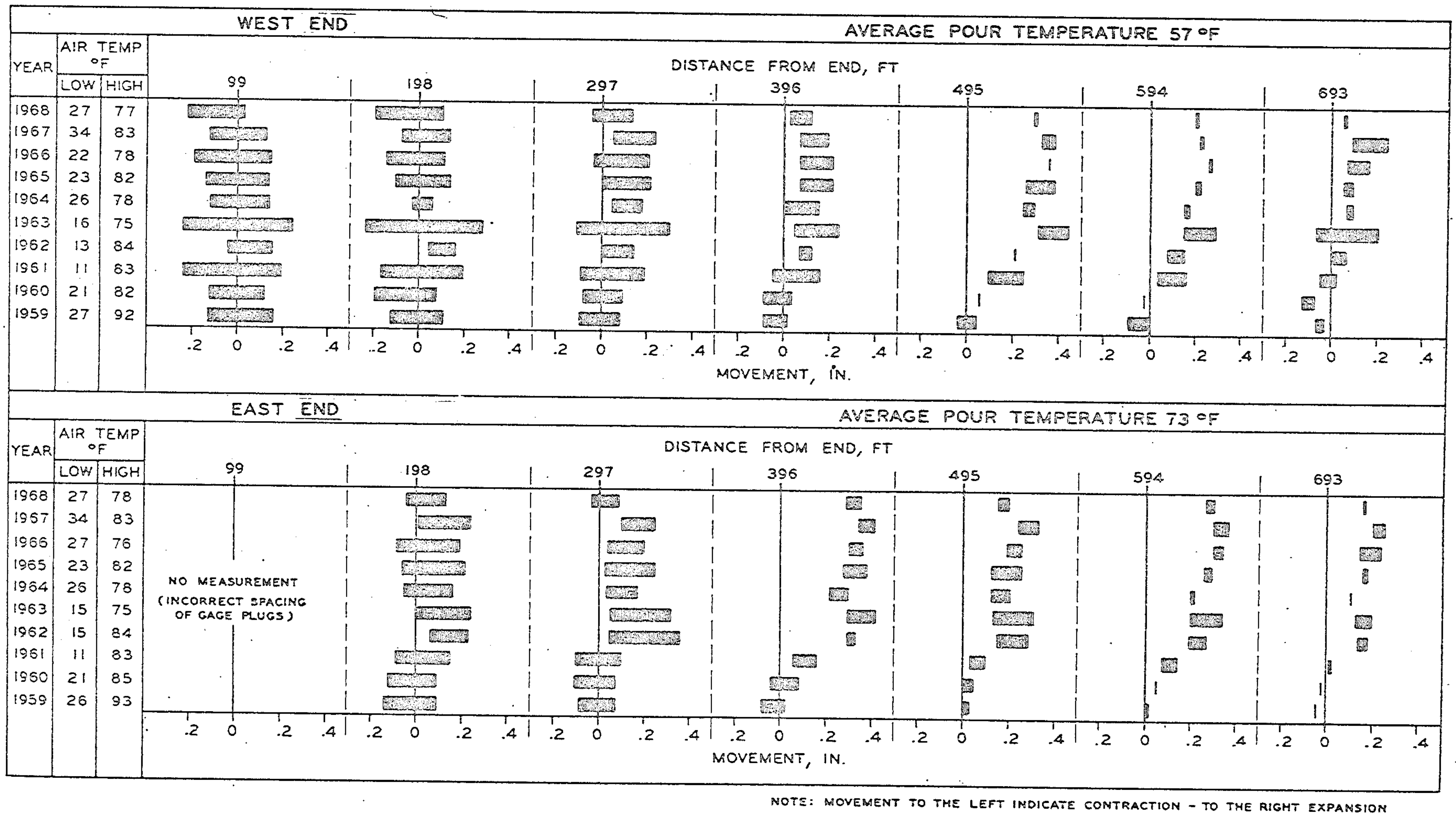


Figure 8. Yearly range in movement at points 99 to 693 ft from free ends of continuously reinforced bar mat section on the eastbound roadway. (Total length of section: 10,550 ft).

At some distance away from the free end of a continuously reinforced pavement the accumulated friction force will be equal to the temperature-induced force in the slab and there will be no displacement at this point. The seasonal length change would, of course, be greatest at the free end and then diminish as the distance from the end increases until the point of zero displacement is reached. The recorded seasonal movements indicate this to be true. There is, however, only a little difference in the movements at the 99, 198, and 297-ft points from the end. At the 396-ft point, the movements show a pronounced decrease in magnitude and at the three remaining points only slight movements occurred. As can be seen from the graphs the seasonal movements at 396 ft from the end and beyond are, in most cases, within plus or minus one standard deviation of the measurement accuracy. The length of pavement required at each end to anchor the central portion of a continuously reinforced pavement is dependent upon the subbase friction developed, the temperature and moisture variation to which the pavement is subjected, and the rate and degree at which cracks fail to close completely after each environmentally induced temperature cycle. The pavement acts as an articulated slab in tension with the induced forces resisted by the concrete and steel. In compression, however, the slab tends to act more like a continuous concrete slab. In effect, this behavior is analogous to a continuous uncracked slab which has a lower modulus-of-elasticity in tension than in compression. Thus, the apparent anchorage length would be less when the slab is under tension. Since this tendency for the pavement to respond as a continuous slab in compression is dependent on the degree of crack closure, the point of zero movement from the free end would progressively tend to increase with each seasonal temperature cycle and the free end movement would correspondingly tend to expand or grow.

Measurements of absolute displacement at the free ends and of expansion joint widths in the relief sections (see Joint Width Variations) show that the continuously reinforced sections have increased permanently in length. This increase in length or growth of the pavement is also reflected in the measurements of yearly range of length change shown in Figures 6, 7, and 8, because as can be noted, the yearly change in length is displaced to the right or expansion side of the graph. Based on these data, it appears that about four years after construction, permanent growth had occurred in a section of pavement about 700 ft from the free end.

The average absolute displacement of the seven points along the slab is shown for the summers of 1960, 62, 64, 66, and 68 in Figure 9. Because some of the gage plugs have spalled out and the pavement has failed in one end region of one section, the plotted values are the average of only three free end movements where the instrumentation is still intact.

Since the plotted values are summer measurements, it must be kept in mind that the seasonal expansion of the pavement is included in these displacements. It should be noted that as the age of the pavement increases the point of zero movement is displaced further away from the free end. For example, in 1960 the plot indicates that at about 550 ft there would be no displacement, but in 1968 the point of zero displacement would be about 1,300 ft from the free end. Further observations will be continued to determine the rate and extent of this movement. A short length of pavement near a free end will be removed to allow the pavement growth to continue normally.

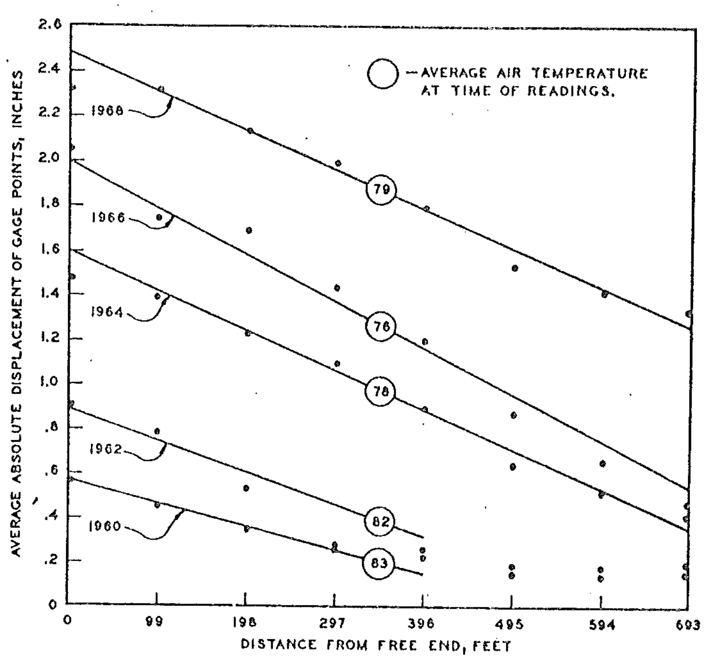


Figure 9. Average absolute summer displacements of points along slab at free end of pavement.

On the westbound roadway, two 500-ft sections, one in each type of reinforcement and in the center of a days pour, were instrumented with gage plugs for the purpose of measuring length changes in the slab away from the free end. The plugs were spaced at 99-ft intervals along the edge of the passing lane and the measurements were performed with an invar tape in the manner described previously.

The average change in length of a 99-ft slab segment as measured in the summer and winter from 1959 through 1968 is shown in Figure 10. Unfortunately, the measuring method was not accurate enough to provide data on which to base any conclusion as to exactly how much change in length occurred. It can be noted that the changes were quite erratic and, in some cases, the winter measurements indicate greater increase in length than the following summer readings.

All the measured changes were small with only five readings greater than ± 0.05 in. On the basis of a maximum expected error of ± 0.086 in., in one measurement at a 95 percent confidence level all readings except four could have been zero. It appears, therefore, that any changes in length occurring in the central region of the slab are extremely minute. Also, based on the 10 year data, there is no permanent increase in the length of a slab segment located in the central region of a long continuously reinforced pavement.

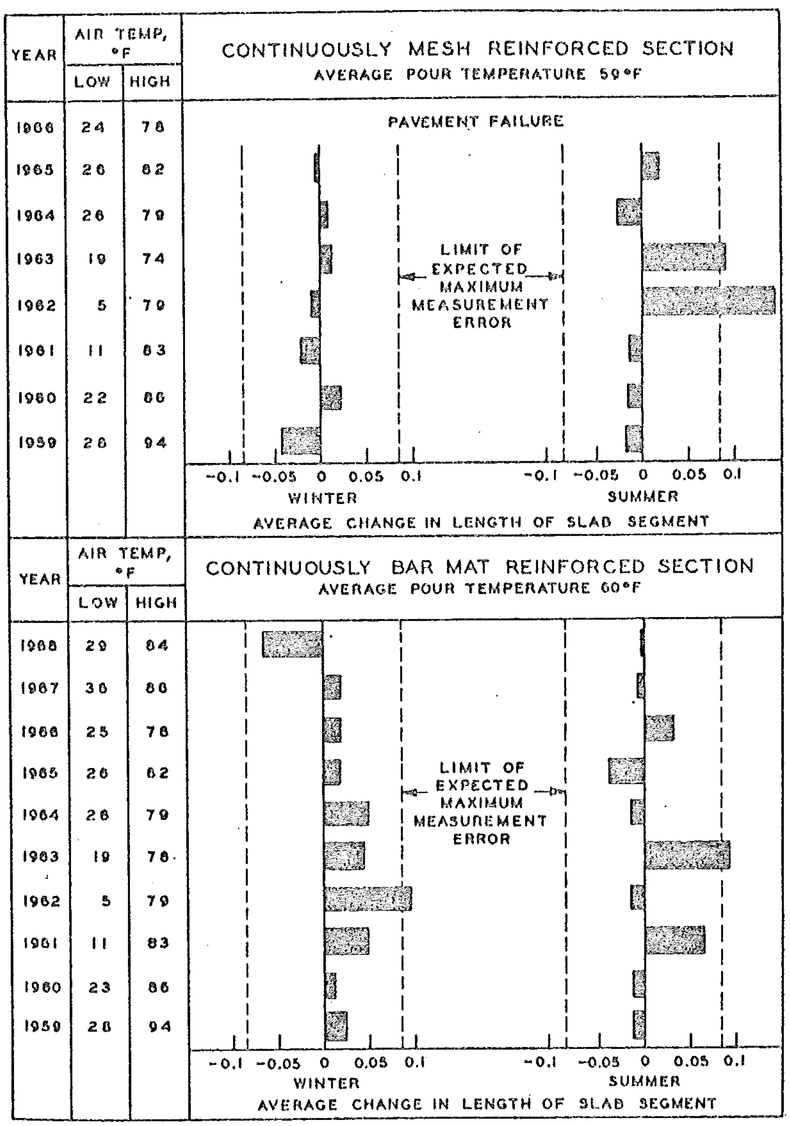


Figure 10. Average change in length of five 99-ft slab segments in center of mesh reinforced section (top) and bar mat section (below).

JOINT WIDTH VARIATIONS

Expansion Joints

As previously mentioned, the expansion joint relief sections were installed to provide for seasonal changes in length and anticipated growth of the continuously reinforced sections. Seasonal joint width variations were obtained by vernier caliper measurements of the distance between gage plugs installed 4 in. each side of the joint centerline. The average annual summer to winter variation in joint width for each of the 11 1-in. expansion joints in the six relief sections is illustrated in Figure 11. The progressive consumption of expansion space can also be seen on this graph.

The openings of the joint adjacent to the free end of the continuously reinforced sections (Joint No. 1) have increased on the average from the original 1 in. width to 1.77 in. during each winter cycle. The maximum average width, 1.94 in., was recorded in 1963 and the average minimum, 1.55 in., in 1968. In the summer, the average joint opening was 0.73 in. with a 1-in. opening in 1964, and an average minimum opening of 0.52 in. was recorded in 1967. Thus, the average seasonal joint width variation measured at the free end joints was 1.04 in.

With joint openings of this magnitude the hot-poured, rubber-asphalt sealant failed in adhesion shortly after construction (Fig. 12). As a result of sealant failure, incompressible materials could readily enter the joints and prevent proper closure of the joints during pavement expansion. Sealant failure and contamination of the joints were so extensive by 1964 that restoration of the joints was necessary. The old sealant and expansion filler were removed and the joints cleaned of extraneous material with compressed air. New filler material was inserted and the joint grooves sealed with the same type of sealant used originally. Because the sealant in the remaining 10 expansion joints in each relief section has primarily been in compression these seals have, in general, performed quite well.

Examination of Figure 11 reveals the progressive closing of the relief section expansion joints. Note that Joints No. 6 and 7 are the only two joints showing only slight permanent closure after 10 years. Therefore, it appears that closure of Joints No. 2, 3, 4, and 5 is caused by growth of the continuously reinforced section and Joints No. 8, 9, 10, and 11 are closing because the adjacent standard jointed concrete pavement also exhibited a permanent increase in length.

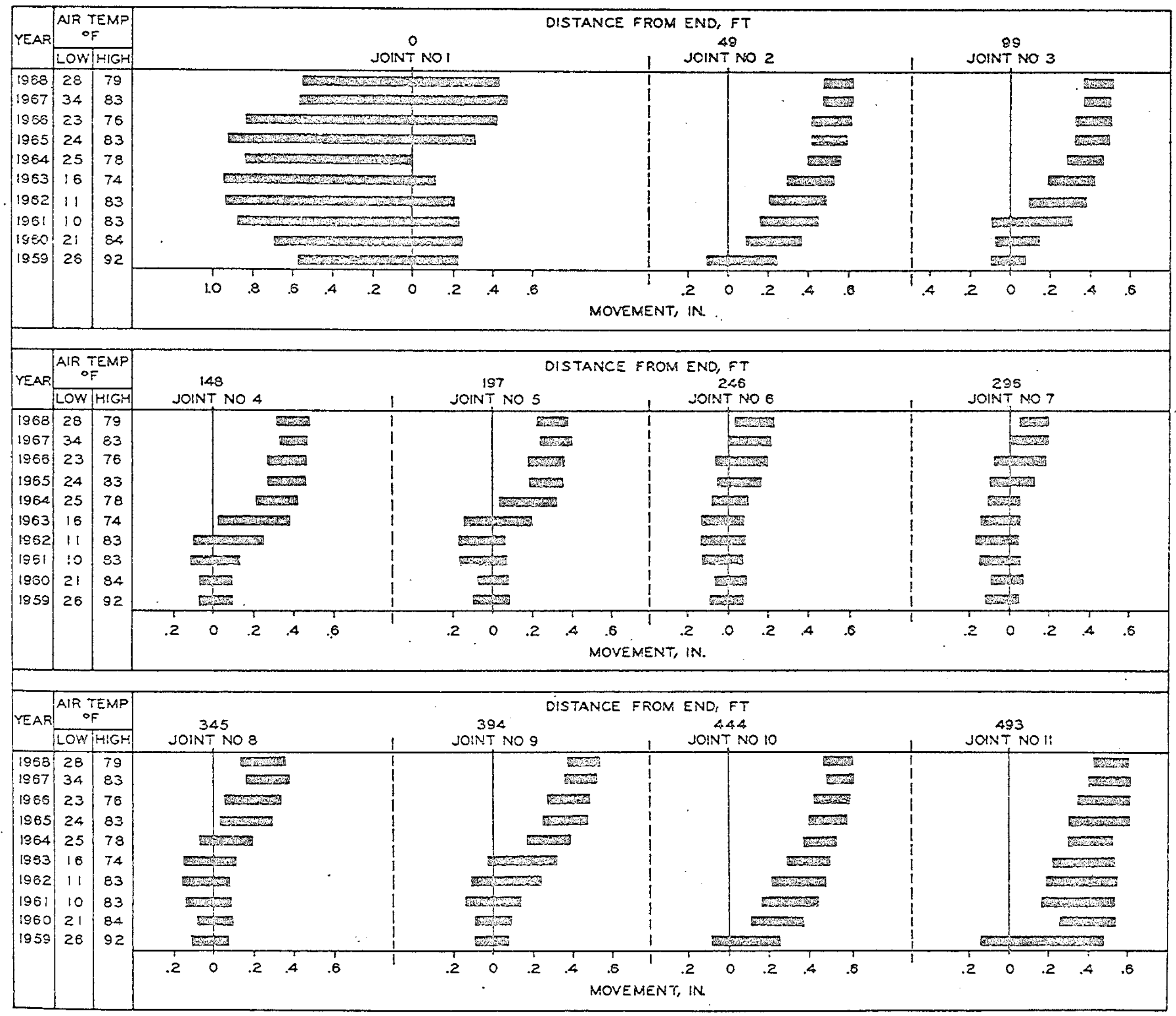


Figure 11. Average annual range in expansion joint widths at each of 11 joints abutting the free ends of the continuously reinforced sections (movement to the left equals increased joint width, to the right decreased joint width).

Based on the joint width data the maximum compression in the filler material in the 1-in. expansion joints is about 0.6 in. On this basis the available expansion at the time of construction afforded by the 11-joint relief section was 6.6 in. In the summer of 1968, 76 percent of this space was consumed. The growth of the continuously reinforced section accounted for 40 percent of the space and the growth of the standard pavement used up to 36 percent. The total loss of expansion space in the 11-joint relief section occurred at an average rate of about 0.5 in. per year. That is, both the continuously reinforced pavement and the adjacent standard jointed pavement have grown at an average rate of about 1/4 in. annually.

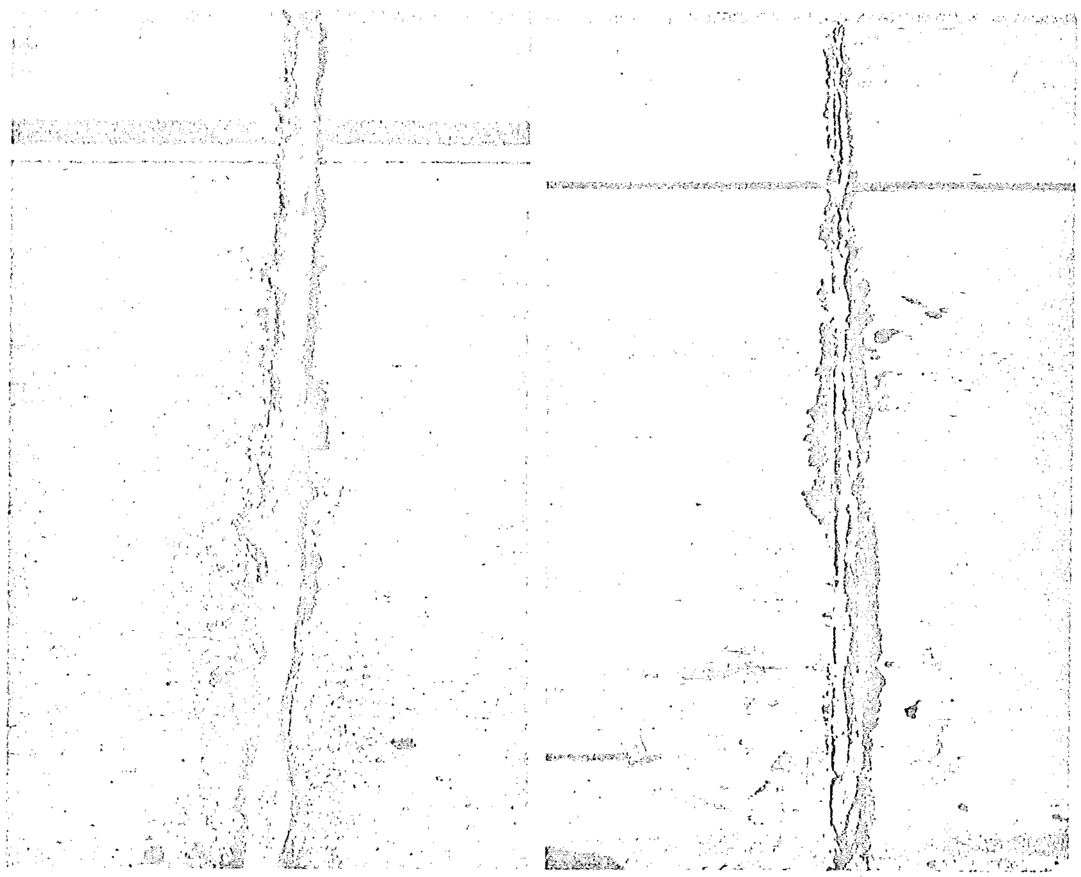


Figure 12. Typical seal failure at the free end of the continuously reinforced sections in 1960 (left) and 1964 (right).

Construction Joints

The continuously reinforced pavement sections contain a total of 14 construction joints. Of these, six are in the bar mat reinforced sections and eight in the mesh reinforced sections. Each joint was constructed at

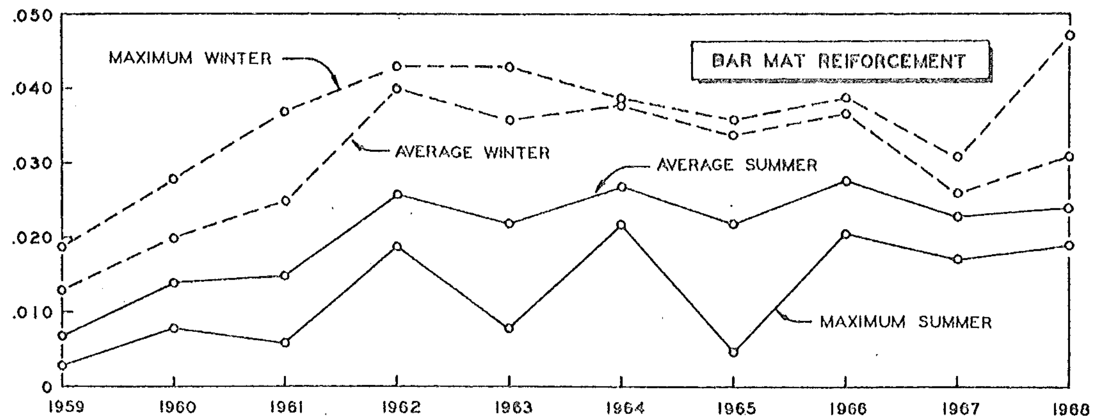


Figure 13. Construction joint width variation data (bar mat reinforcement).

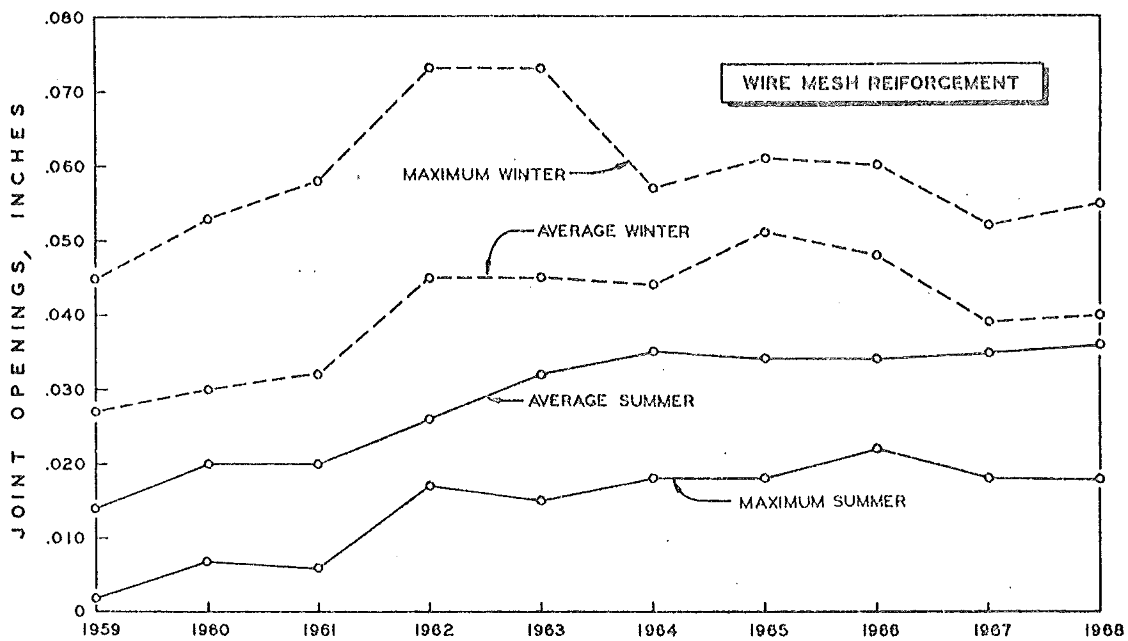


Figure 14. Construction joint width variation data (mesh reinforcement).

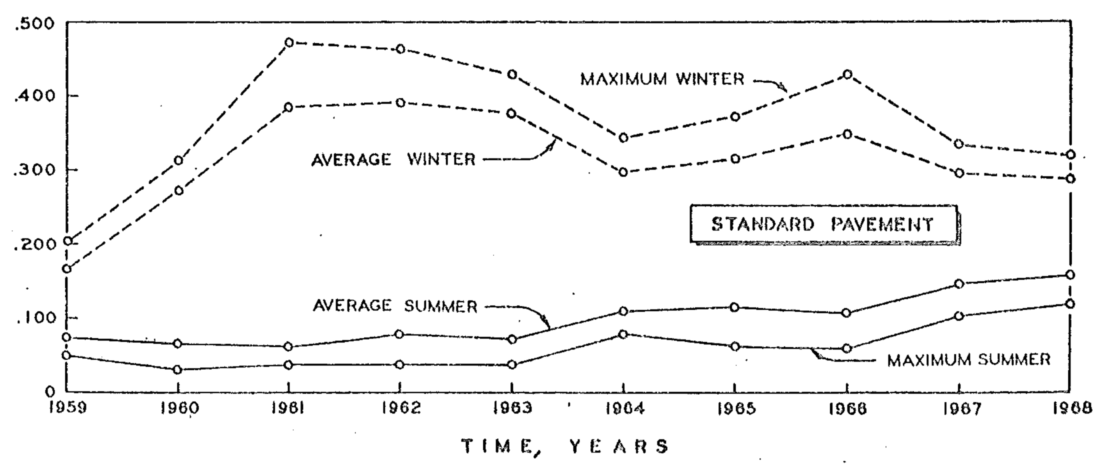


Figure 15. Contraction joint width variation data (standard pavement).

a minimum distance of 5 ft away from a reinforcement lap. In addition to the steel being carried through the joint, 1-1/4 in. diameter steel bars, 18 in. long were installed at 12-in. centers to assure proper load transfer across the joints. All joints were instrumented with gage plugs in order to obtain information on the width changes occurring at this type of joint.

At two of the joints in the bar mat reinforced sections, unusually wide cracks were noted in the morning side pour during the first winter after construction. By the spring of 1962 the pavement at these locations had deteriorated to the point where repair was necessary. One joint in a mesh reinforced section failed by blowup in the summer of 1965. The cause of failure and the repair methods utilized at these locations are discussed in detail in this report in the section titled Failure and Repair. Measurements of joint width changes at the failed joints as well as at one joint in the mesh reinforced section where one of the gage plugs spalled out were deleted from the data presented.

The average summer and winter openings for each year for joints in the bar mat sections and the extreme closure and opening recorded each year are shown in Figure 13. On the average, the yearly joint width variation was 0.009 in. and the average yearly extreme variation was 0.023 in. It is of interest to note that the joints have never completely closed since they were constructed. Rather the average summer opening steadily increased until 1962 when it leveled off at a value of approximately 0.025 in. The average winter opening also continued to increase for the first four years and since then has remained relatively constant at 0.036 in.

Joint width variation data for joints in the mesh reinforced sections are presented in Figure 14. Generally speaking these joints performed in the same manner as those in the bar mat sections, but the width variations were greater. The yearly average variation from summer to winter was 0.011 in. and the average yearly extreme variation was 0.044 in. Since 1962 the average summer opening has been near 0.035 in. and the winter opening has remained around 0.045 in.

Contraction Joints

Ten contraction joints in the standard pavement control section in the middle of the eastbound roadway were instrumented for joint width variation measurements. The extreme and average joint openings measured in the summer and winter of each year are shown in Figure 15.

The joint openings given represent those of standard contraction joints spaced at 99 ft. None of the joints have returned to zero opening since construction. The average summer joint opening has been close to 0.07 in. during the first five years and since then has increased to about 0.15 in. Infiltration of foreign material into the joints through failed seals could account for this increase in joint opening during periods of maximum pavement expansion.

The average joint opening in the winter increased from 0.17 in. in 1959 to 0.38 in. in 1961 and maintained this approximate value during subsequent winter measurements until 1963. In 1964 the average winter joint opening decreased to 0.30 in. and has remained at about this level during subsequent measurements through 1968. The reason for the reduced joint opening during the last few years may be attributed to the formation of several transverse cracks in the slabs between joints.

CRACK FORMATION

Transverse cracks in a continuously reinforced pavement occur in an erratic manner and at irregular spacings. Some cracks extend across the full pavement width; some originate at either pavement edge, and either terminate, join other cracks or divide into two cracks before reaching the opposite edge; and some originate and end without reaching either pavement edge.

Because of the erratic nature of crack formation, the results of any crack spacing survey will depict the crack conditions in relation to the definition used to describe what constitutes a crack and the location chosen to measure the crack spacings. In this project, any transverse crack greater than 12 ft in length (one-lane width) regardless of where it originated or ended was counted as an individual crack. Where surveys were conducted to obtain the crack condition in one lane, cracks greater than 6 ft in length (half-lane width) were counted as a single crack regardless of their origin or ending point. During both type of surveys the crack spacings were measured at the pavement edge.

Daily inspections of several pavement sections were planned beginning the first day of pouring in order to determine the manner or pattern in which cracks develop. However, because of inclement weather conditions and personnel scheduling difficulties this inspection schedule could not always be followed. The early crack development data are presented in Table 3 in terms of average crack spacing from 2 to 12 days after pour and 70 days after pour. As the Table illustrates, the average crack spacing during the first few days after pour varies from section to section. This is to be expected since the concrete consistency, curing conditions, and temperature influence the development of cracks, and these characteristics naturally would vary on a project of this length.

From Table 3 it appears that the crack spacing in the mesh reinforced sections decreased faster than in the bar mat reinforced section during the first few days after construction. This was most probably due to the fact that the average pour temperature for the bar mat reinforced sections was eight degrees lower than the temperature at which the mesh reinforced sections were placed. After the pavement had attained an age of two to two and one-half months, there was little difference in the crack spacing among the various sections with an average spacing of 10.5 ft in the bar mat reinforced sections, and 10.3 ft in the wire mesh reinforced sections.

TABLE 3
SUMMARY OF EARLY TRANSVERSE CRACK DEVELOPMENT

Time After Pouring (Days)	Average Transverse Crack Spacing, ft.												
	Bar Mat Reinforcement						Wire Mesh Reinforcement						
	Eastbound Pour Length, ft			Westbound Pour Length, ft			Eastbound Pour Length, ft				Westbound Pour Length, ft		
	2763	3168	2256	2510	3409	3696	2110	2410	2344	2332	3225	2780	2582
2	1,382							1,205			1,075		
3							88		117		124	927	
4				627			35	28		130			
5						205			22	52	43		
6	921						15	18	19	25	18	21	
7			84						19			17	24
8			24	120				17					
9		15		30	487		14			20			
10					39	116			16				
11					27	30							
12						25							
70 (1)	10	11	10	11	10	11	11	10	10	10	10	10	11

(1) Average of all individual pours; range: 56-85 days.

From the time of construction in 1958 through 1961, quarterly surveys of crack spacing for the full-width pavement were conducted in six daily pour sections reinforced with bar mat and in seven daily pour sections of wire mesh reinforcement. Since the crack spacing at an unrestrained end of a continuously reinforced pavement does not represent the spacing in restrained pavements a 300-ft length of pavement at these ends was deleted from the survey in all pour ends adjacent to a relief section. The results of these surveys, shown in Figure 16, illustrate the relationship between average crack spacing and pavement age for both types of reinforcement. During this time interval the average crack spacings for both sections decreased slowly with time with a consistently shorter interval in the bar mat reinforced sections. In the fall of 1958 the average crack spacing in the bar mat reinforced sections was 10.5 ft and in late 1961, 4.8 ft. In the mesh reinforced sections, the average crack spacing interval for the same dates was 10.3 and 5.9 ft, respectively. It is evident from Table 3 and Figure 16 that the average crack spacing decreases rapidly during the first few months and then proceeds to decrease at a much slower rate from then on.

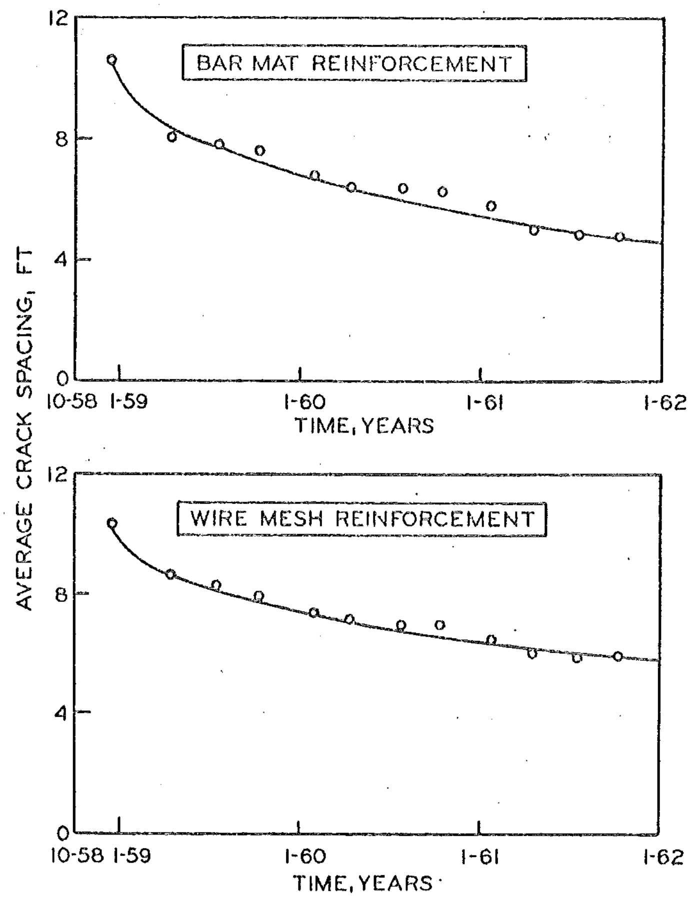


Figure 16. Average crack spacing in bar mat and wire mesh reinforced sections.

To obtain information on the frequency distribution of the crack spacing interval in the traffic and passing lane, four 1,000-ft sections in the center of four daily pours in each type of reinforcement were surveyed. These surveys were conducted in the fall of 1958, spring of 1959, fall of 1961, and spring of 1969. Figures 17 and 18 show the frequency distribution of the crack spacing interval in the traffic and passing lanes for these survey dates in the bar mat reinforced and mesh reinforced pavement sections, respectively.

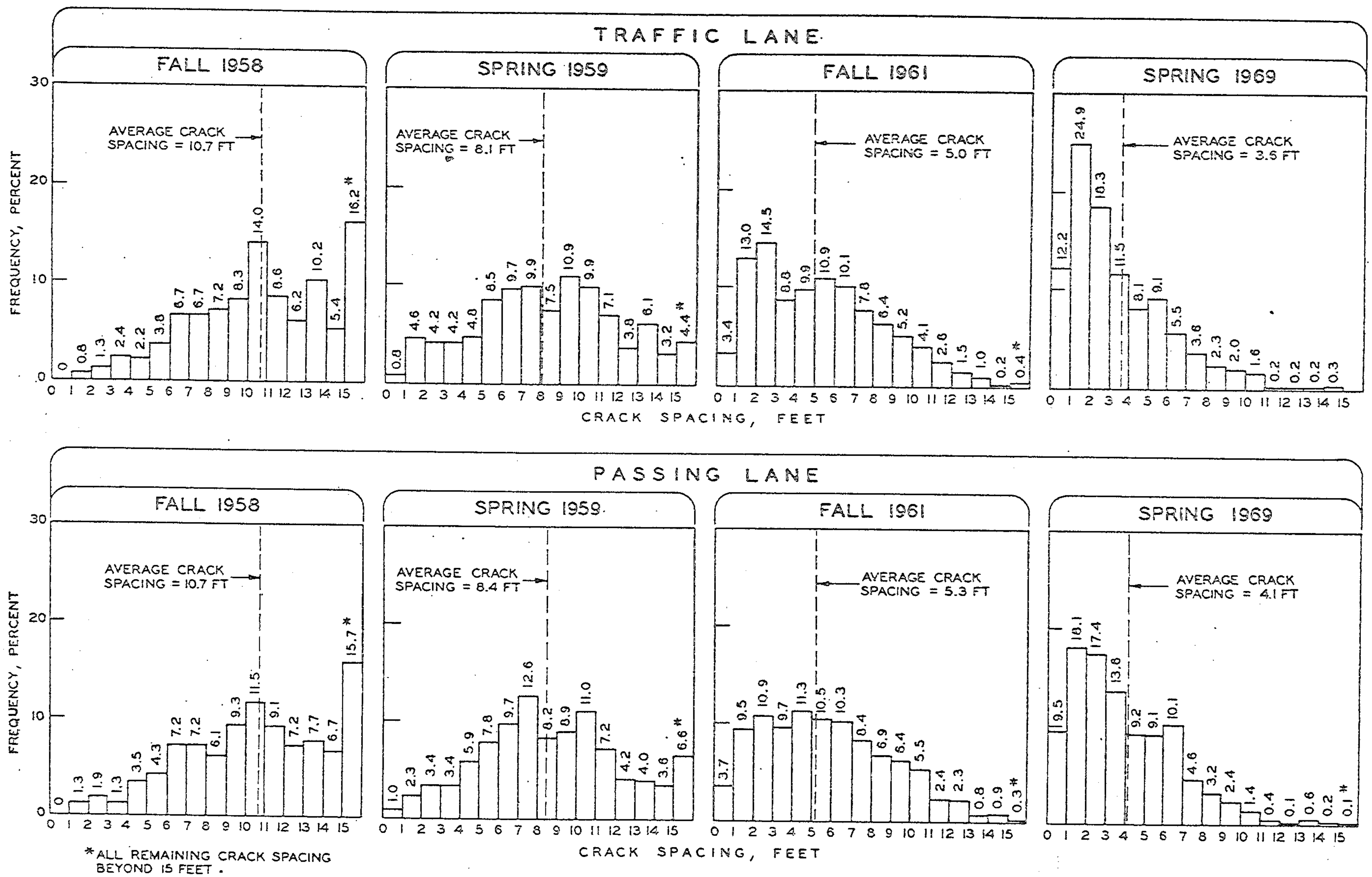
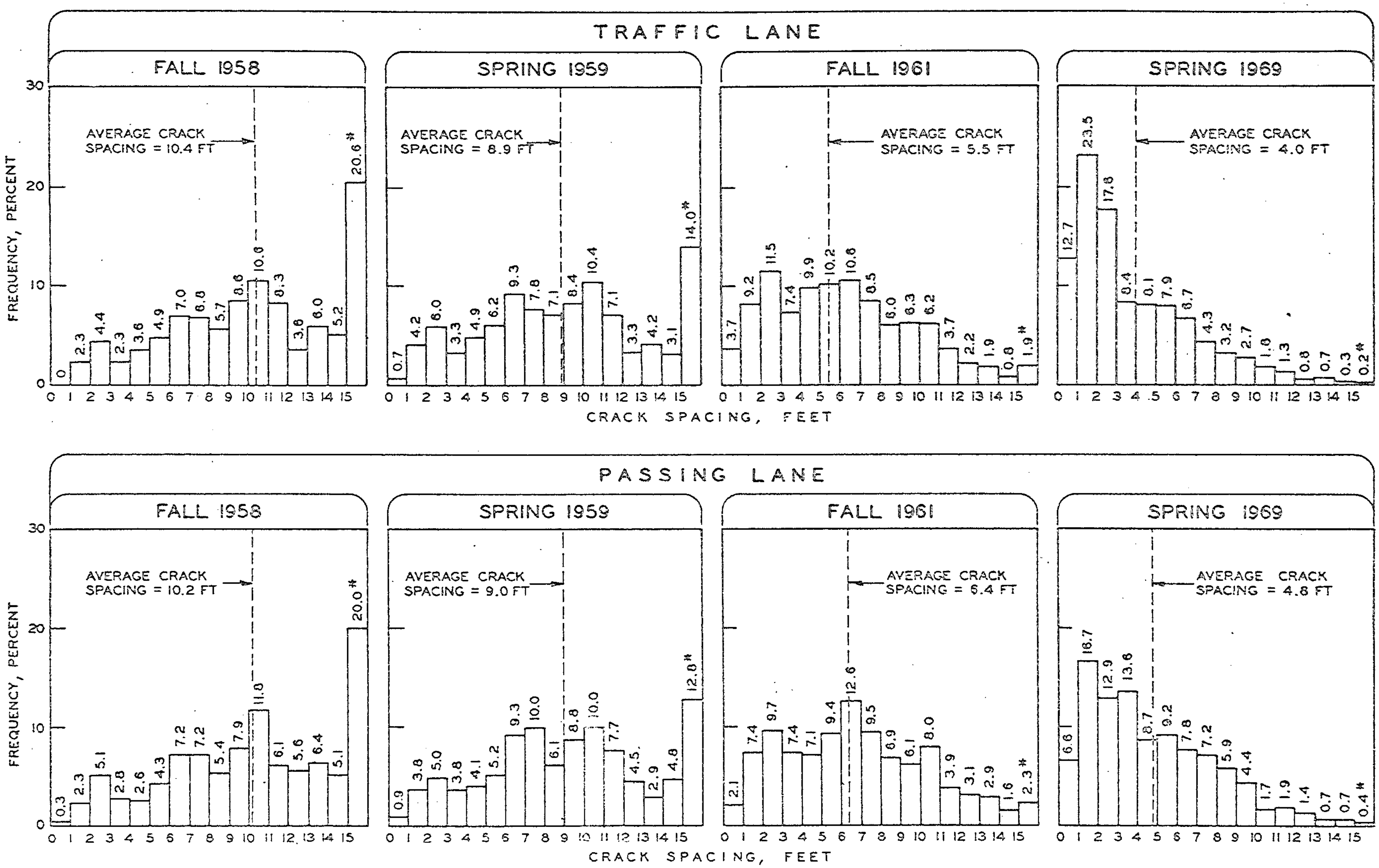


Figure 17. Frequency distributions of crack spacings in bar mat reinforced sections.



*ALL REMAINING CRACK SPACING BEYOND 15 FEET

Figure 18. Frequency distributions of crack spacings in wire mesh reinforced sections.

These Figures show that the average crack spacing decreased with time. It is perhaps of most interest to note that in the fall of 1958 about 16 percent of the cracks in the bar reinforcement, and 20 percent in the wire reinforcement were spaced at over 15 ft; but by the spring of 1959 the greater than 15-ft interval had decreased to about 6 percent and 13 percent, respectively. Also, during the first survey there were hardly any cracks spaced at less than 1 ft whereas the percentage of crack intervals at this distance had increased to an average of 10 percent in 1969. As can be noted from these frequency distributions, the initial crack spacing interval tends to be more uniformly distributed and with time becomes more skewed toward the shorter crack spacings.

There was no appreciable difference in crack spacing between the traffic lane and passing lane in the fall of 1958. However, in the spring of 1969 the average crack spacing in the passing lane was 0.5 ft and 0.8 ft greater than that in the traffic lane in the bar mat and mesh reinforced sections, respectively. This relationship between average crack spacing in the two lanes is illustrated by the line graph in Figure 19. This greater crack spacing in the passing lane indicates that the heavier truck traffic which normally uses the traffic lane influences the formation of new cracks in that lane.

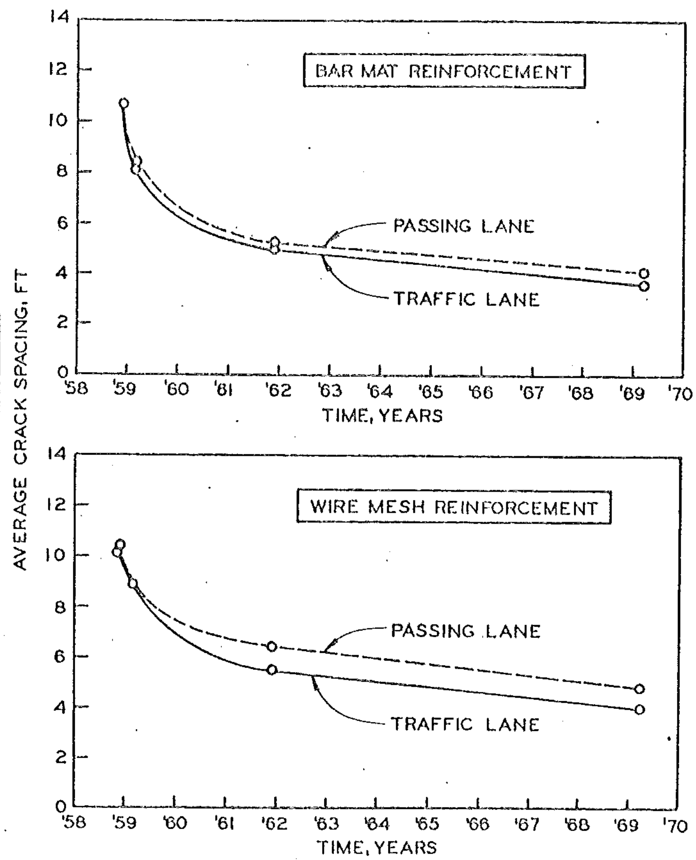


Figure 19. Average crack spacings in traffic and passing lanes of bar mat and wire mesh reinforced sections.

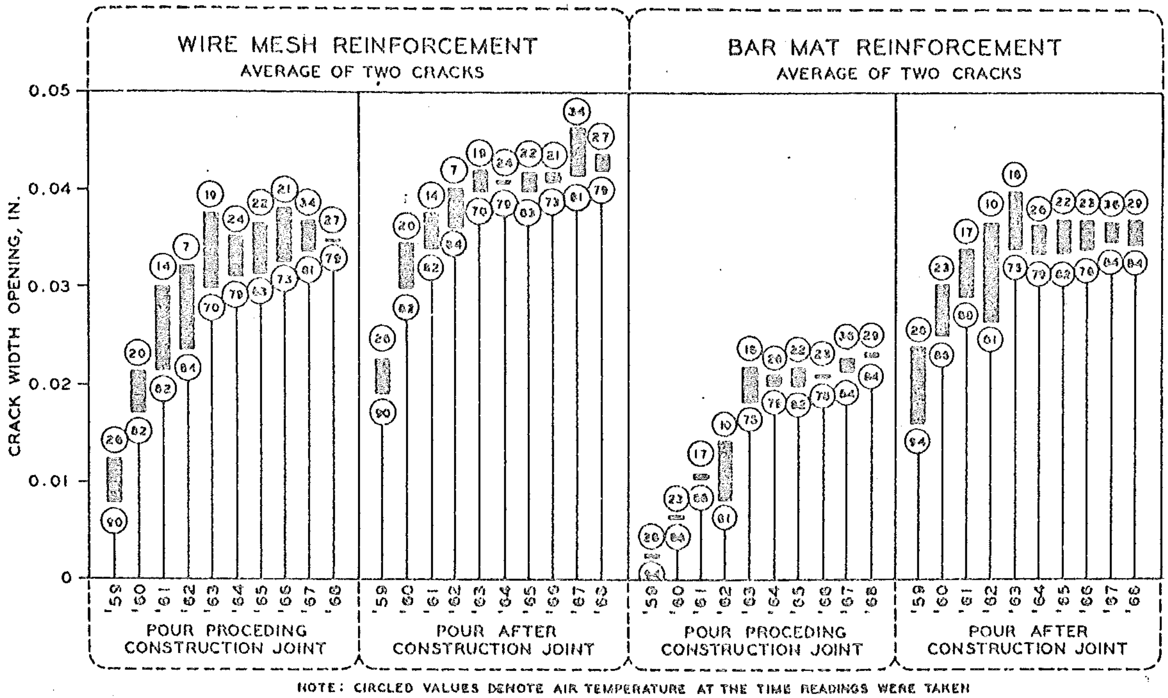


Figure 20. Summer-winter crack width variations near construction joints in bar mat and wire mesh reinforced sections.

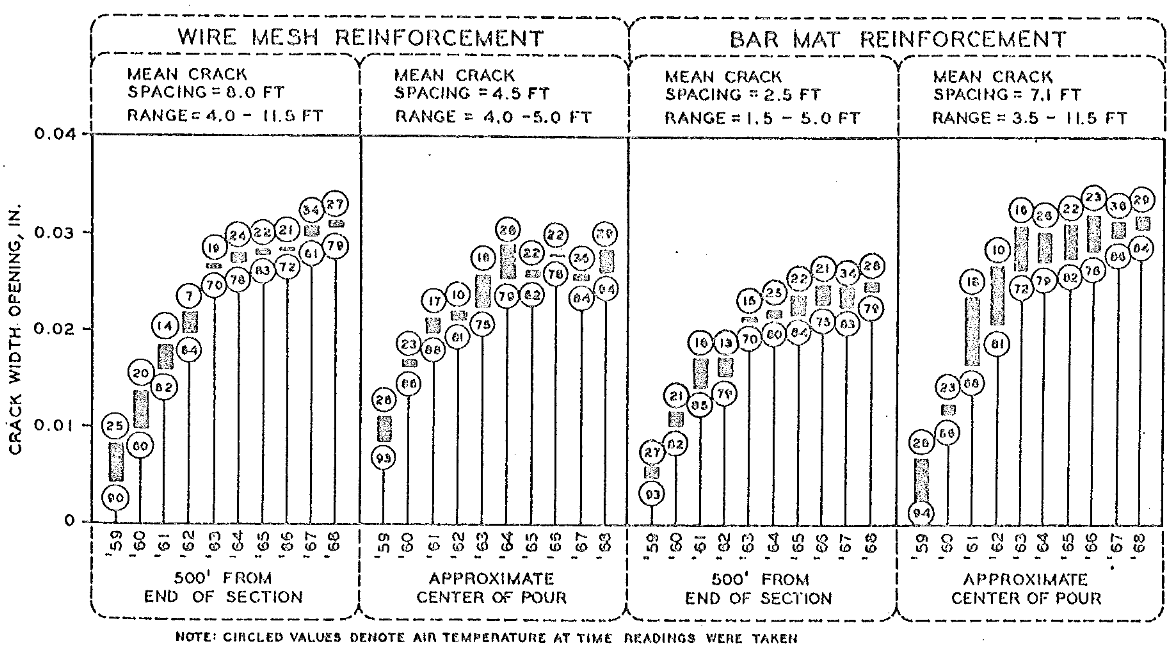


Figure 21. Summer-winter crack width variations 500 ft from the end of section and approximately at the center of a day's pour.

SURFACE CRACK WIDTHS

Data on crack width variation were obtained from three locations in both the bar mat and mesh reinforced sections. These locations were as follows:

1) near a construction joint

2) approximately 500 ft from a free end

3) in the approximate center of a day's pour.

The first four cracks which developed at these locations were selected for measurement. At each crack two gage plugs were installed, one each side, 5 in. from the crack and 12 in. from the pavement edge. The distance between plugs was measured with a Whittmore 10-in. mechanical strain gage, and the average surface crack width at the time of the initial strain gage measurement was obtained with a scale microscope. This reading was applied as a correction to the initial mechanical gage reading.

At the construction joints, two of the instrumented cracks were located in the pour preceding the joint and two in the pour following the joint. The summer-winter crack width variations for cracks on both sides of the joint and in both types of reinforced sections are shown in Figure 20. It can be seen that in all cases the summer to winter width variation is less than 0.01 in. Moreover, the cracks never close but progressively continue to open during the first four years after which there appears to be no further increase in opening. The chart indicates that the cracks preceding the construction joint do not open quite as much as those just following the joint.

Figure 21 shows the summer-winter average crack width variations 500 ft from the end of a section and in the center of a day's pour for both reinforcement types. The summer-winter variations in crack width is small; less than 0.005 in., with the exception of the 1961-62 changes for cracks in the center of a day's pour in the bar mat reinforced section which averaged between 0.006 and 0.007 in. As in the case of cracks near a construction joint, the width increases sharply during the first four years, tends to level off and maintain about the same width thereafter. The average crack width magnitude at which there appears to be no further increase is about 0.028 in. at 500 ft from the end, and 0.026 in. at the center of a day's pour in the mesh reinforced section. For the same two locations in the bar mat reinforced section the value at which the crack widths stabilized was about 0.023 and 0.028 in., respectively.

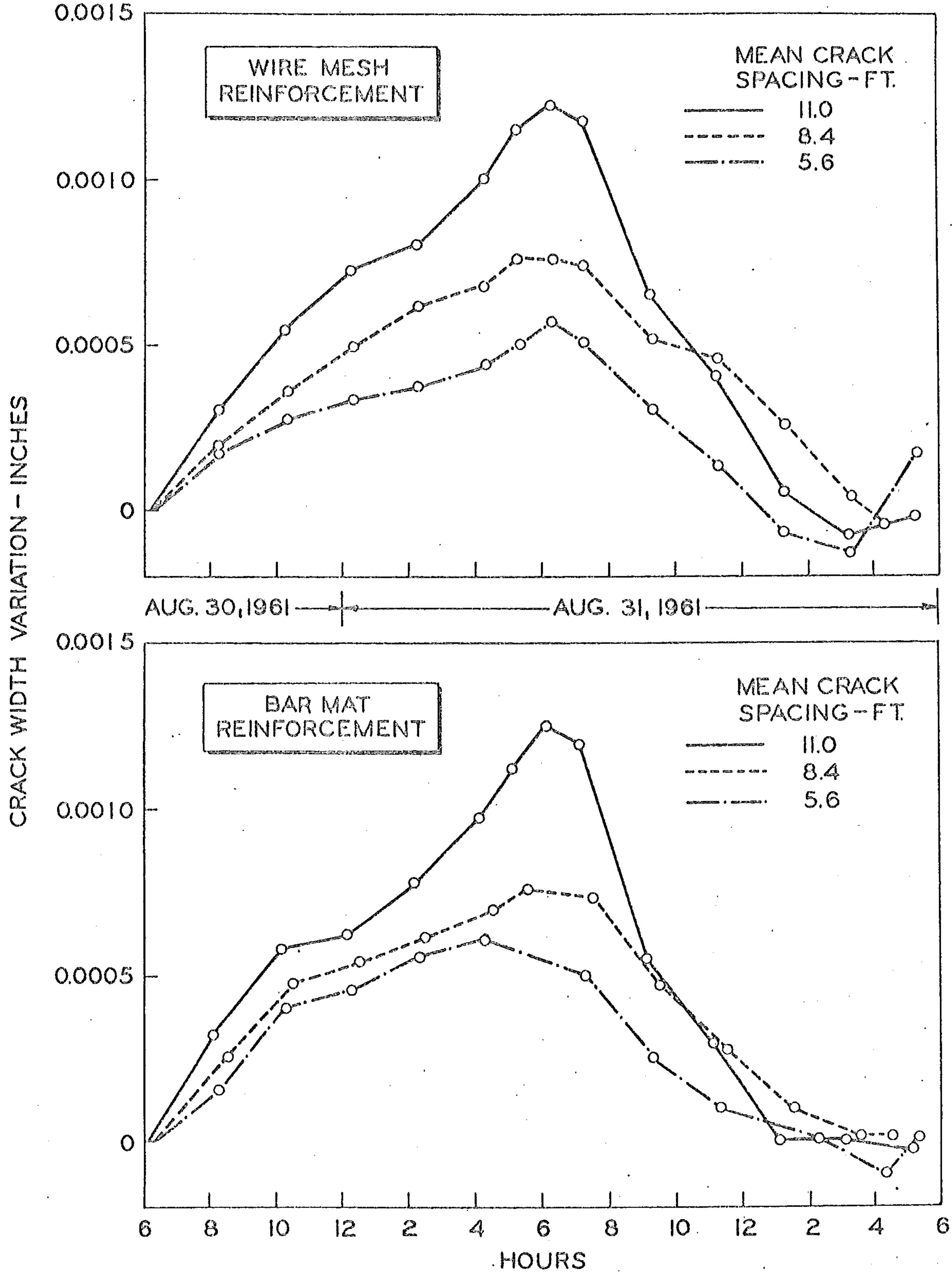


Figure 22. Crack width variations in wire mesh and bar mat reinforced sections over a 24-hr period.

In addition to the yearly cycle, the cracks are also subjected to daily cycles of width variation. To obtain information on the magnitude of the daily width changes, the width variations of the instrumented cracks at the previously mentioned three locations were measured every hour from 6:00 p.m. August 30 to 5:00 p.m. August 31, 1961. The measured air temperature change for this period was 26 F with a low of 62 F occurring at 6:00 a.m. and a high of 88 F at 4:00 p.m. on August 31.

The average crack width variations for cracks having a mean spacing of 5.6, 8.4, and 11.0 ft are shown in Figure 22 for both the bar mat and mesh reinforced sections. It can be seen that the greater the mean spacing the larger the crack width variation. It can also be seen that the variation was very nearly the same in both types of reinforcement. The average maximum variation was 0.0013, 0.0008, and 0.0007 in. for the long, intermediate, and short crack spacing, respectively.

In order to investigate the steel condition at several typical cracks, twelve cores were taken in September 1963, six in March 1966, and eight in March 1968. Each core was cut so the crack divided the core in approximate halves. Of the first set of twelve cores, four cores were taken where the crack intervals ranged from 8 to 12 ft, four where the crack spacing was 3 to 8 ft, and four where a 1 to 3 ft crack spacing was found. The cores taken in 1966 and 1968 were selected from areas where some of the larger crack spacings were found (7 to 15 ft). Each time an equal number of cores were cut from the two types of reinforced sections.

Prior to investigating the steel condition the crack width variation from top to bottom of the cores was measured with a scale microscope. Table 4 gives the average crack width at successive levels below the surface of each core taken in 1963 and 1966. Of the eight cores taken in 1968, four were cut in half as shown in Figure 23 and the crack width variation in the vertical plane of each of these four cores is also included in Table 4.

On the basis of these measurements, it is evident that the width of the cracks decreases rather rapidly in the top 2-in. layer, and at the steel level (3 to 4 in. below the surface) the crack widths have decreased to a magnitude which is generally considered non-detrimental with respect to steel corrosion. Although the cracks were visible for the full pavement depth, their widths were extremely small from the steel level to the bottom of the cores. Generally speaking, it appears that for the crack spacings and reinforcement types involved, those variables have little influence on the magnitude of the crack widths at the steel level and below.

TABLE 4
CRACK WIDTH VARIATIONS IN VERTICAL PLANE

Date Cored	Crack Spacing Interval, ft	Reinforcement Type	Core No.	Average Crack Widths at Successive Levels				
				At Pavement Surface	1 to 2 in. Below Surface	At Steel	4 to 6 in. Below Surface	Below 6 in.
September 1963	8-12	Bar Mat	3	0.070	0.020	0.020	0.012	0.002
			9	0.050	0.028	0.010	0.010	Minute
		Mesh	6	0.045	0.028	0.015		0.010
			12	0.045	0.015	0.010		0.004
	3-8	Bar Mat	1	0.045	0.014	0.015		0.002
			8	0.025	0.013	0.010	0.006	0.003
		Mesh	4	0.035	0.018	0.020		0.004
			10	0.025	0.012	0.015	0.004	0.003
	1-3	Bar Mat	7	0.015	0.010	0.010	0.002	0.002
			2	0.005	0.002		0.002	Minute
		Mesh	5	0.025	0.018	0.010		0.003
			11	0.010	0.011	0.010	0.008	0.005
March 1966	8-15	Bar Mat	3	0.040	0.018	0.013	0.006	0.005
			4	0.033	0.020	0.009	0.007	0.006
			6	0.037	0.020	0.008	0.003	0.002
		Mesh	1	0.028	0.026	0.008	0.009	0.003
			2	0.048	0.018	0.010		0.004
			5	0.052	0.032	0.009		0.006
March 1968	7-15	Bar Mat	5	0.047	0.032	0.015	0.007	0.010
			8	0.045	0.025	0.005	0.007	Minute
		Mesh	4	0.055	0.015	0.008	0.007	0.003
			2	0.055	0.028	0.010	0.012	0.010

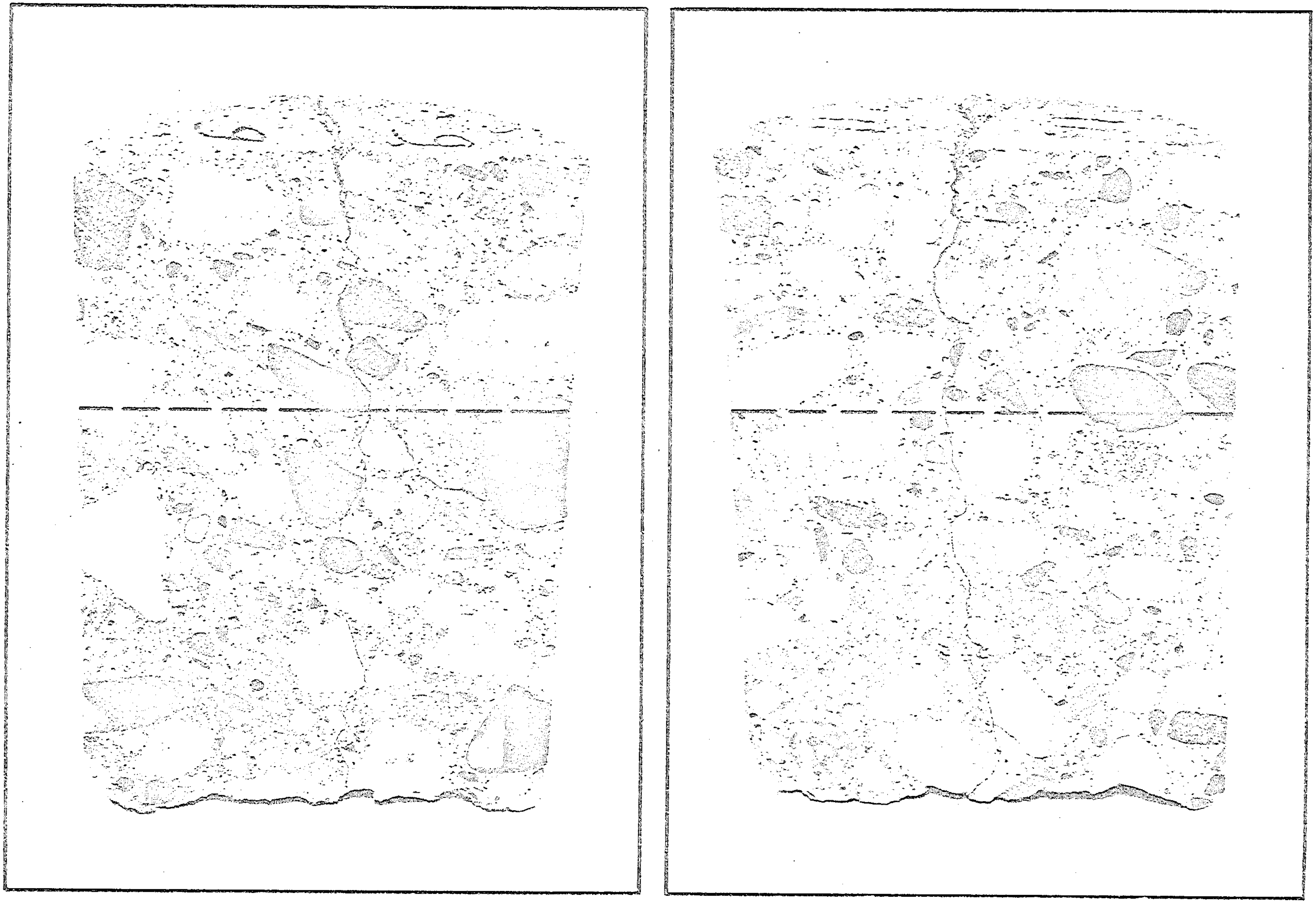


Figure 23. Typical appearance of crack widths in bar mat (left) and wire mesh (right) reinforced pavement after 10 years of service.

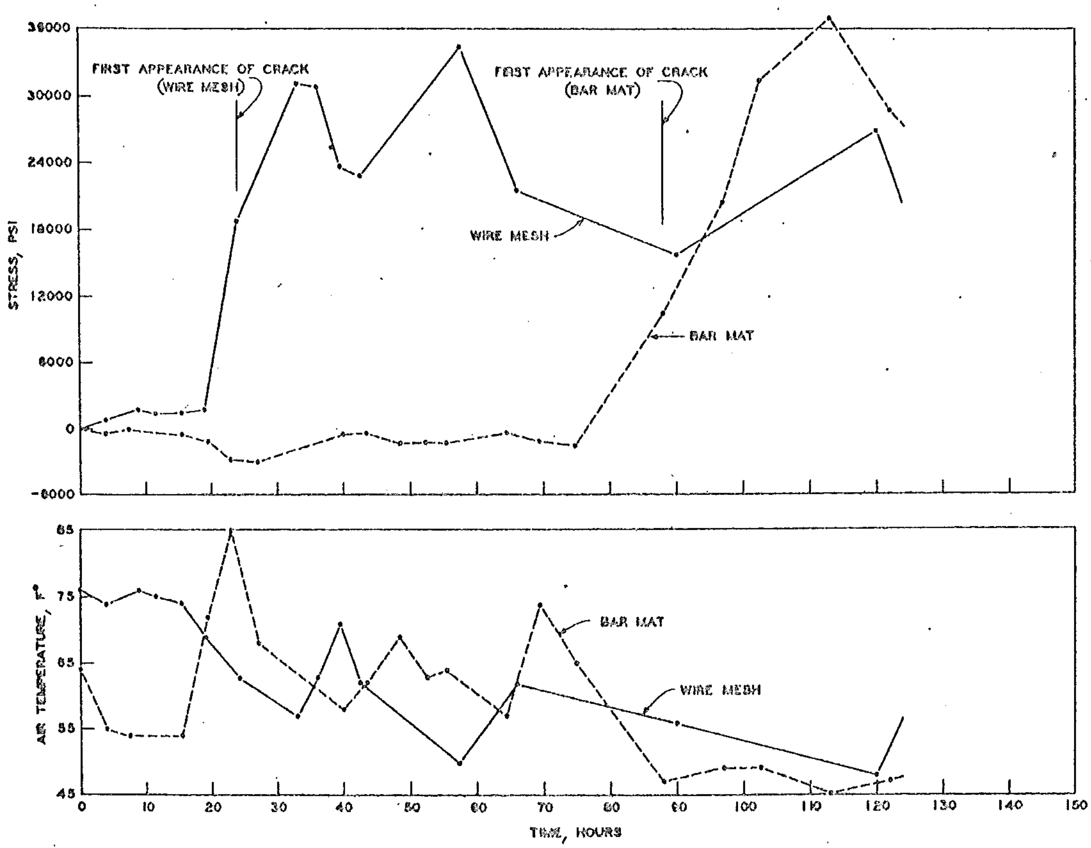


Figure 24. Stress and air temperature variation for 120-hr period.

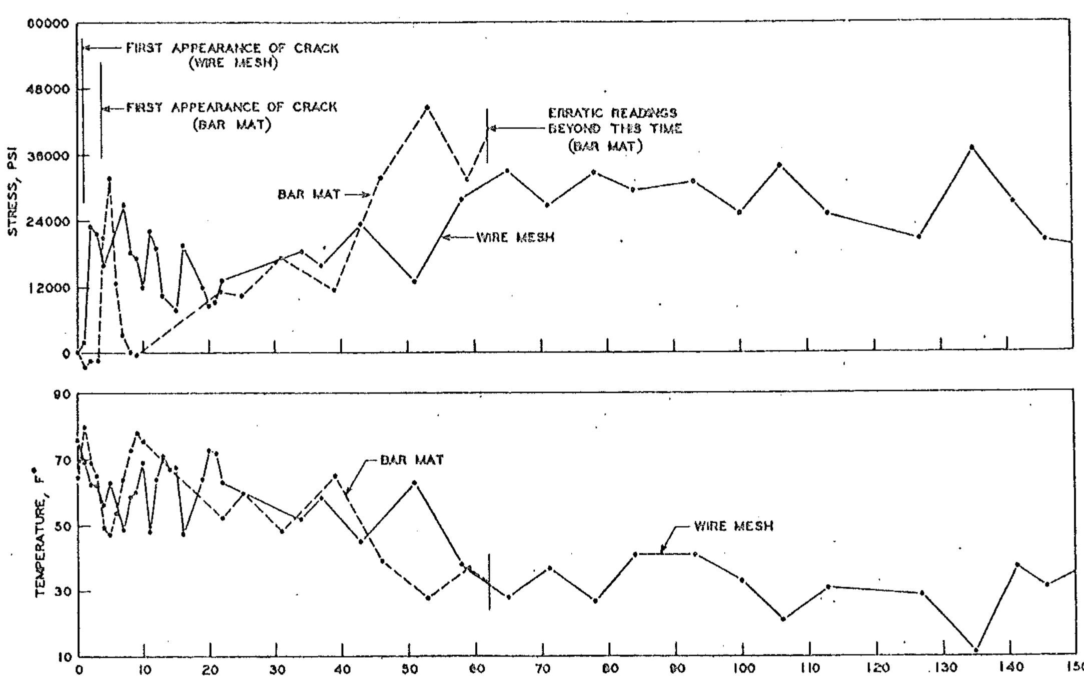


Figure 25. Stress and air temperature variation for 150-day period.

REINFORCEMENT STRESSES

During construction one-lane reinforcing mat each of deformed bars and wire mesh were instrumented with SR-4 strain gages to measure the resulting reinforcement strain intensities with time. Five of the longitudinal bars or wires in each of the two reinforcement mat types were gaged and two 12-ft lengths of No. 28 corrugated steel, 3 in. high were placed in line with the gage centers and across both lanes to insure the formation of a crack at these locations.

Strain gages were also mounted on hot rolled steel plates which were enclosed in an air-foam rubber and paraffin block, so as to allow unrestrained movement of the plate due to temperature change. These blocks were then placed in the concrete and, with the bar gages, completed the strain measuring bridge circuit. A complete description of this instrumentation is given in Reference (1).

Plots of the resultant strain measurements for each reinforcement type together with the air temperature variation for time periods of 120 hours and 150 days after initial construction are shown in Figures 24 and 25, respectively.

With reference to Figure 24, the crack in the wire mesh section formed sometime between 19 and 24 hours after placement and the steel stress changed from about 1,800 psi tension before cracking to about 31,000 psi tension after cracking. In the case of the bar mat section, the crack occurred sometime between 75 and 88 hours after placement and the corresponding steel stress changed from about 1,500 psi compression before cracking to 37,500 psi after cracking.

The strain and air temperature variation over a 62-day period in the case of the deformed bar mat and about five months for the wire mesh reinforcement are depicted in Figure 25. Small resistance to ground measurements of the bar mat gages resulted in erratic strain readings and these measurements were discontinued after the two month period indicated. The gages on the wire mesh reinforcement yielded erratic readings after five months, and these measurements were also discontinued after this period.

The maximum recorded stresses for the time periods indicated were about 44,000 psi in the bar mat section and about 37,000 psi in the wire mesh section.

Although the recorded stresses do not represent the maximum that occurred in the reinforcement in this experimental pavement, they are

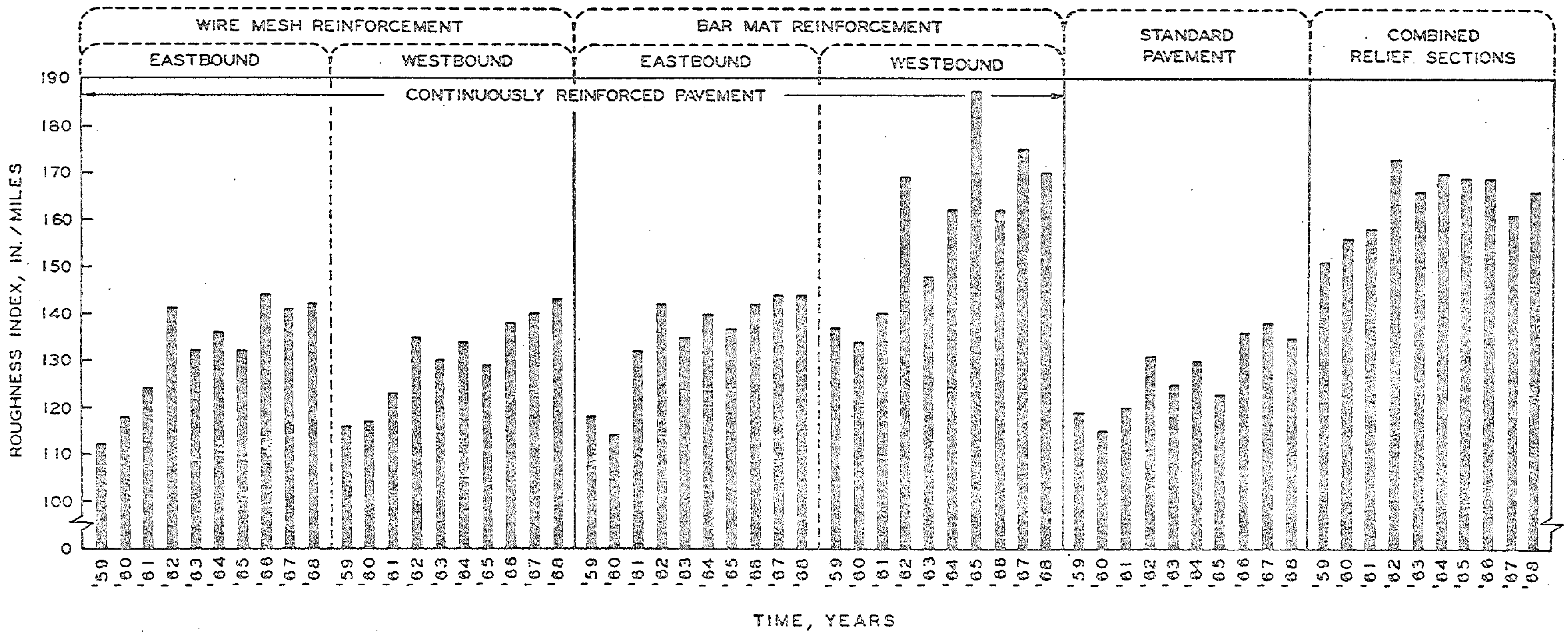


Figure 26. Roughness indices for continuously reinforced, standard, and relief sections.

indicative of the general stress variation that takes place during the initial time interval after placement. There is also, however, no evidence to indicate that the reinforcement stresses anywhere in the experimental pavement have exceeded the yield strengths of the respective steels.

SURFACE ROUGHNESS

The surface roughness of the experimental pavement was measured with the MDSH roughometer which is of the same general type as the Federal Highway Administration's (BPR-Type) roughometer. The roughness value for a particular test section of the roadway was obtained by measuring the roughness in the center of the traffic and passing lane and then averaging the two values. Figure 26 gives the roughness indices thus obtained for each year for the continuously reinforced sections, the standard pavement section and the relief sections.

In Michigan, three arbitrarily determined roughness classifications are used: good, average, and poor, with the respective roughness ranges in inches per mile being 0 to 130, 131 to 174, and 175 or more. On the basis of this classification the initial roughness indices of the continuously reinforced sections and the standard pavement section were in the good category, whereas the relief sections were in the average group range. As can be noted in Figure 26, the roughness of all sections tended to increase for the first four years, and then level off for the remaining six years.

The 1968 average roughness indices of the four continuously reinforced sections, standard pavement, and the combined relief sections were 150, 135, and 166 inches per mile, respectively, placing them in the average roughness category.

The relatively short slab lengths and the 11 1-in. wide expansion joints in the relief sections would account for the higher roughness indices measured in these areas. The relatively greater initial and subsequent roughness indices of the bar mat reinforced section on the westbound roadway is considered only indicative of the variability of roughness that results on any given project as a function of the environmental and construction conditions that exist at the time of concrete placement.

Comparing the roughness of the two pavement types reveals that the standard pavement section has a smoother surface. However, the difference is small (15 in. per mile based on the average value for the 1968 readings) and is of no real significance.

A random selection of 30 Michigan construction projects for standard 99-ft jointed pavement reveals that initial roughness indices range from 109 to 151 with a mean of 129. Based on these data, the assumption that continuously reinforced pavements provide a substantially smoother surface than jointed pavements is not necessarily true.

PICTORIAL RECORD OF SURFACE CRACK CONDITION

At each of the three locations in both types of reinforced sections where crack width measurements were made, the surface condition of each crack was recorded pictorially on a yearly basis. The photographs were taken in the fall of each year, except in 1968 when they were taken in the latter part of July. Figures 27, 28, and 29 show the appearance of a typical crack in the traffic lane in 1958, 61, 63, and 68 in the wire mesh reinforced pavement at a construction joint, 500 ft from a free end, and in the approximate center of a days pour. Figures 30, 31, and 32 illustrate a typical crack condition for the same years and same locations in the bar mat reinforced pavement. The adjacent crack spacing as of 1968 is also shown on each Figure.

Generally speaking, the appearance of the cracks is reasonably good. It can be seen that the surface width has increased with time. However, close inspection of the cracks revealed that a good part of the apparent surface width is due to the crack edges being rounded. It is also evident that small spalls have occurred along the cracks through the years. The rounding and spalling of the cracks are more prevalent in the wheel path areas and no surface deterioration of this type appears near the outside lane edges.

Cracks in areas of relatively large spacing exhibit greater surface width and deterioration than cracks in areas of relatively small spacing. To illustrate this, photographs taken in July 1968 of cracks representing both large and small spacings are shown in Figures 33, 34, and 35. Figure 33 shows cracks at small spacing in both types of reinforced pavements, and Figures 34 and 35 show the condition of cracks at large spacing in plain welded wire mesh and bar mat reinforced pavement, respectively.

There is little difference in the surface condition of cracks in the two types of reinforced pavement. With the exception of a few cracks having rather large spalls or passing through a surface pop-out, the cracks cannot be felt nor do they create any thumping, such as joints do, when driving over the pavement at normal speeds.

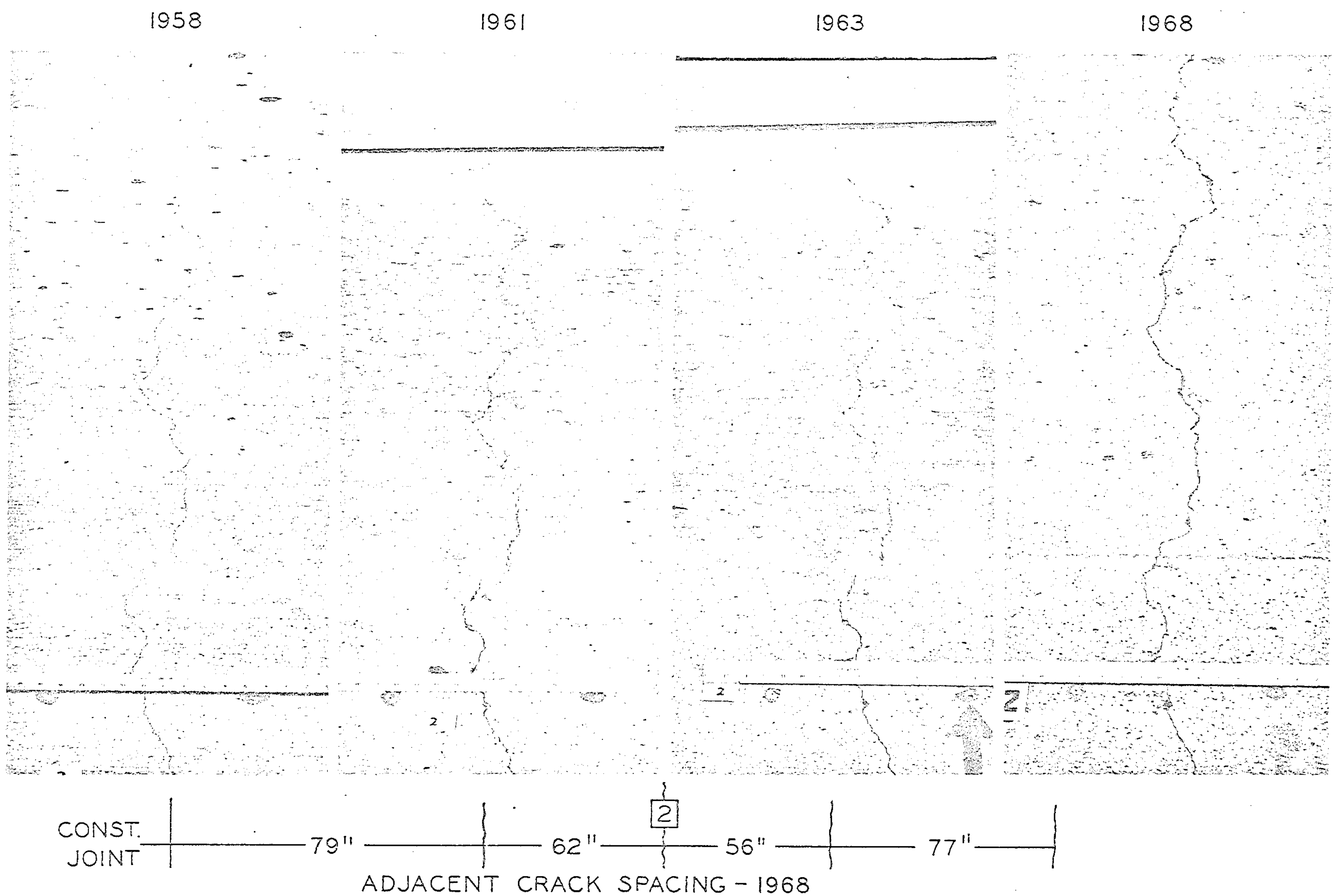


Figure 27. Transverse crack in wire mesh section at construction joint.

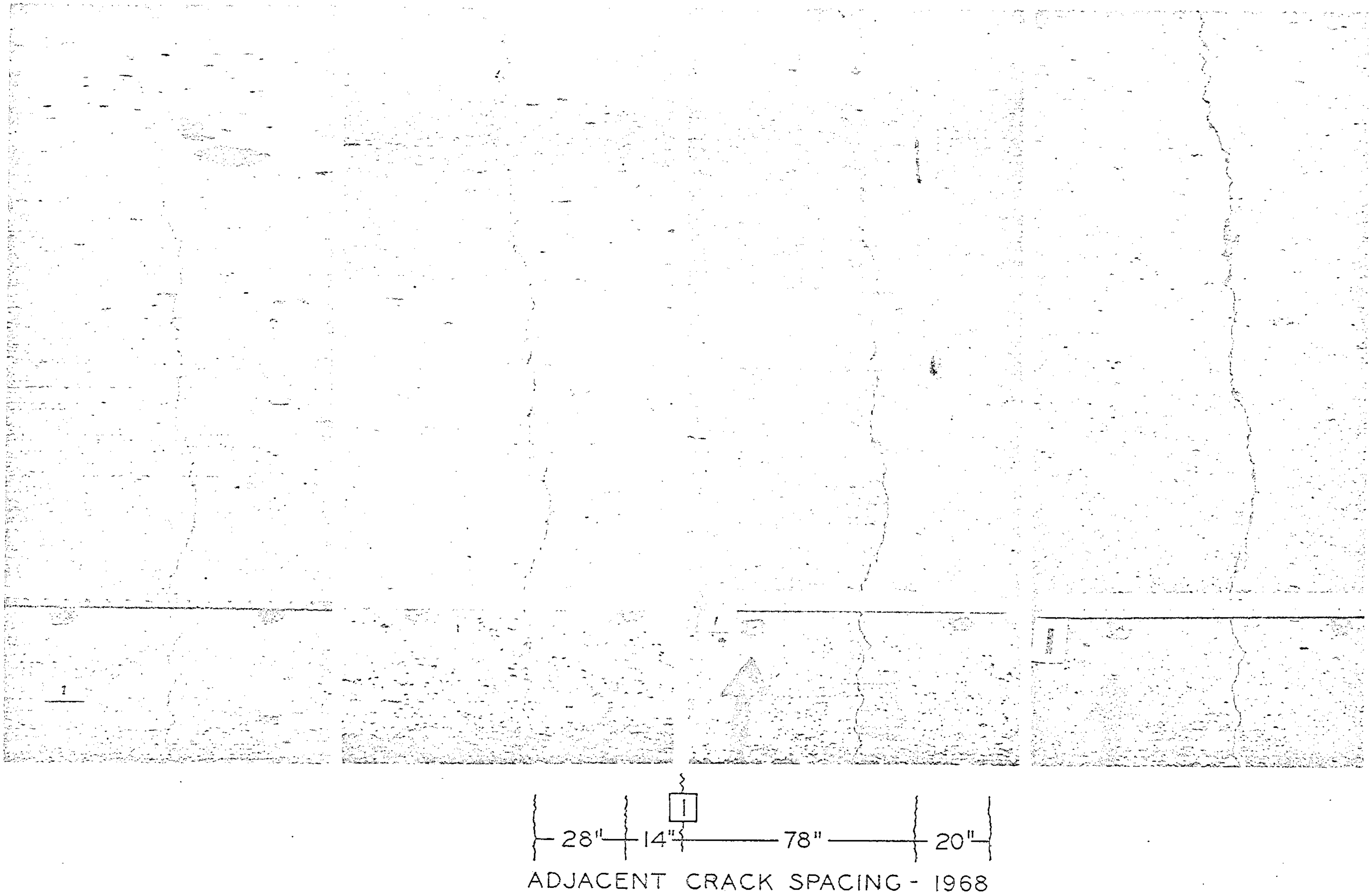


Figure 28. Transverse crack in wire mesh section 500 ft from free end.

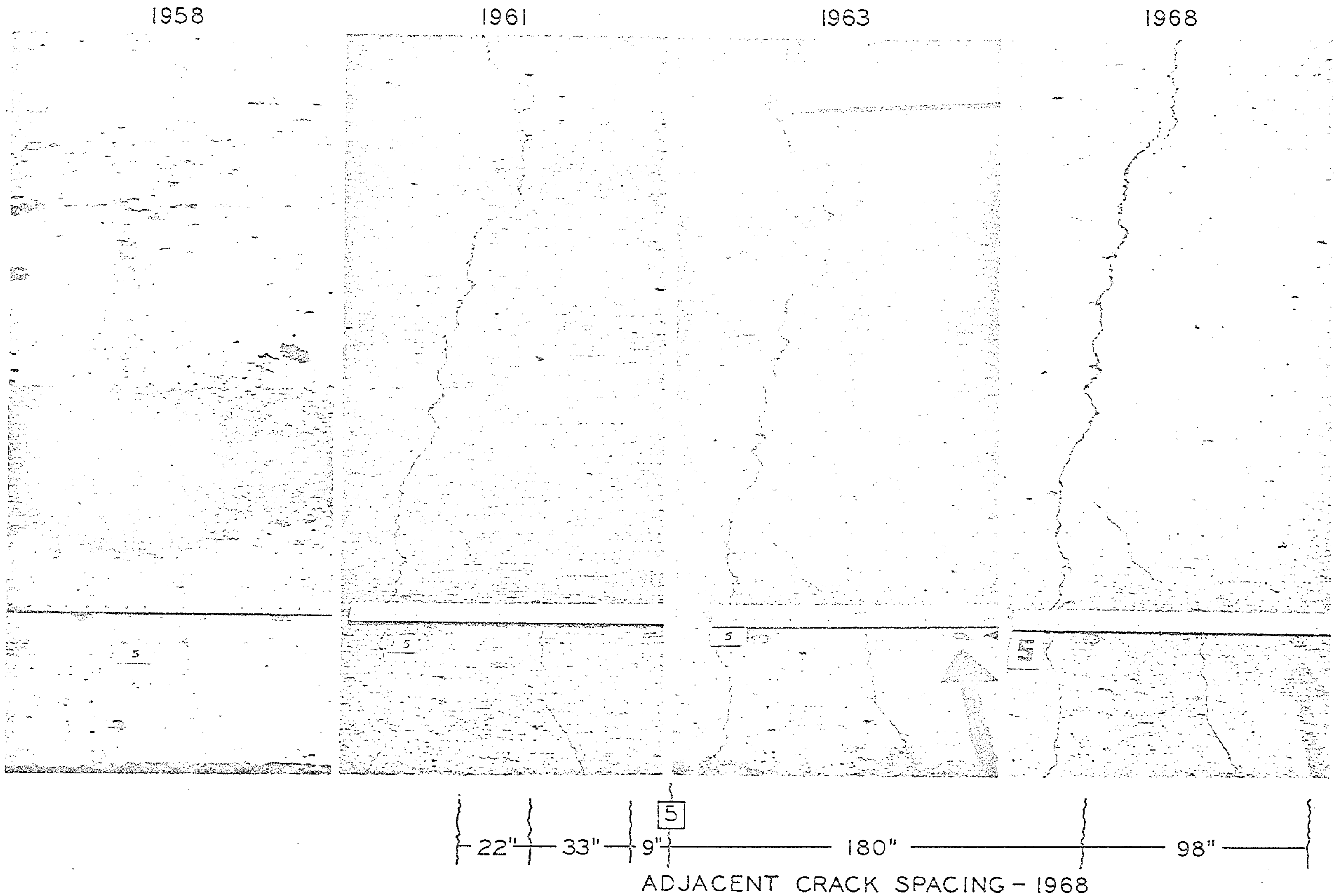


Figure 29. Transverse crack in wire mesh section in the center of a day's pour.

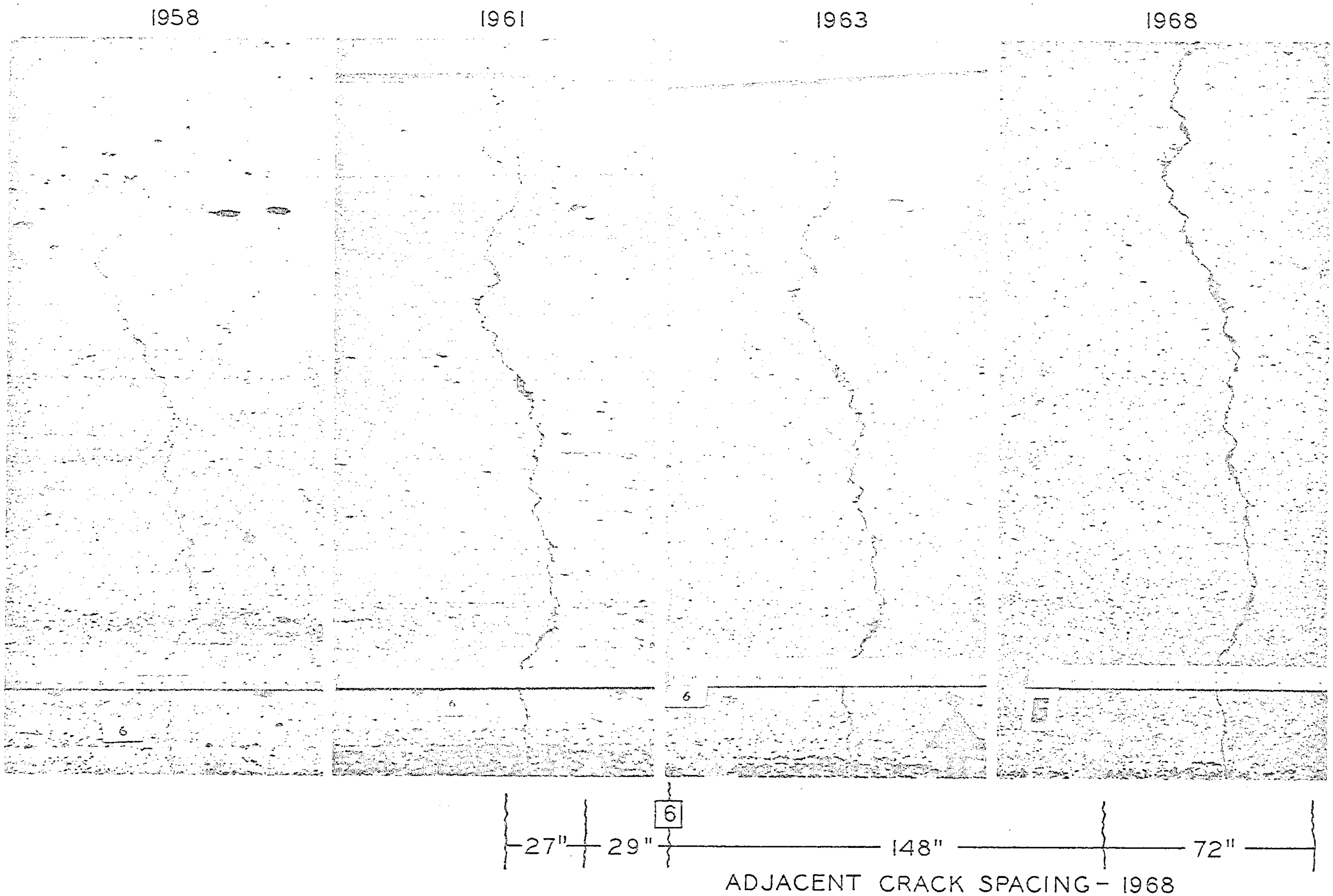


Figure 30. Transverse crack in bar mat section at construction joint.

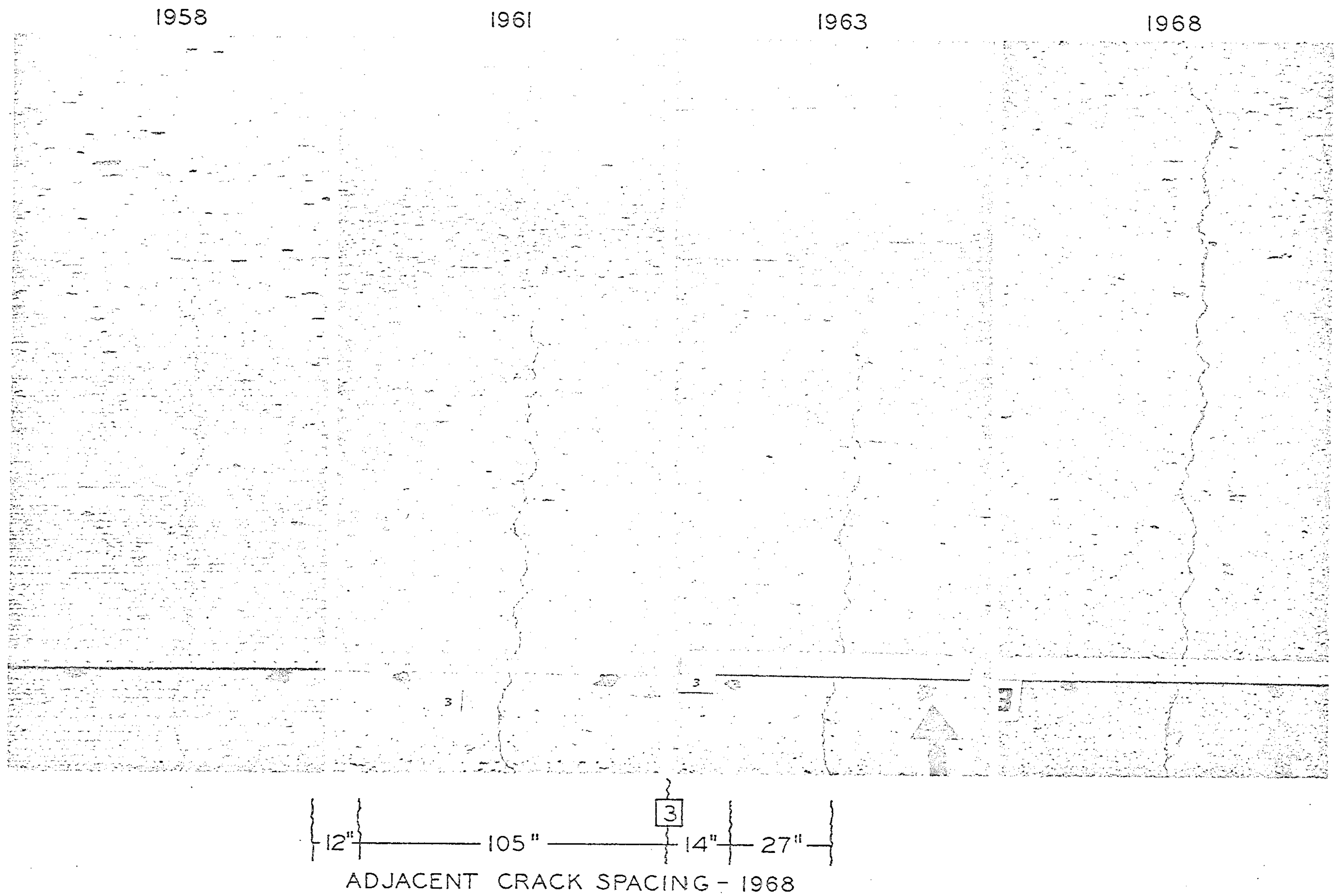


Figure 31. Transverse crack in bar mat section 500 ft from free end.

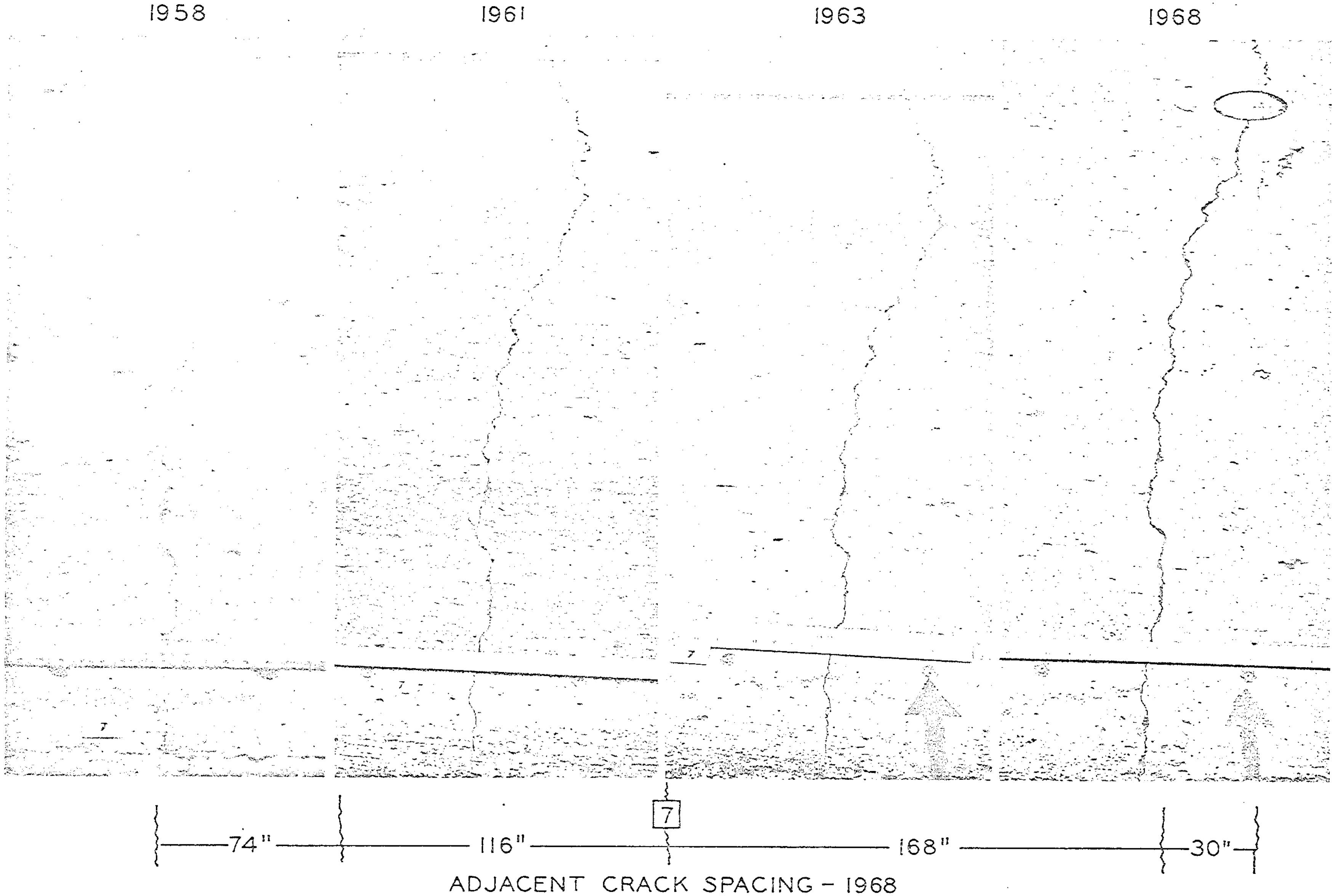
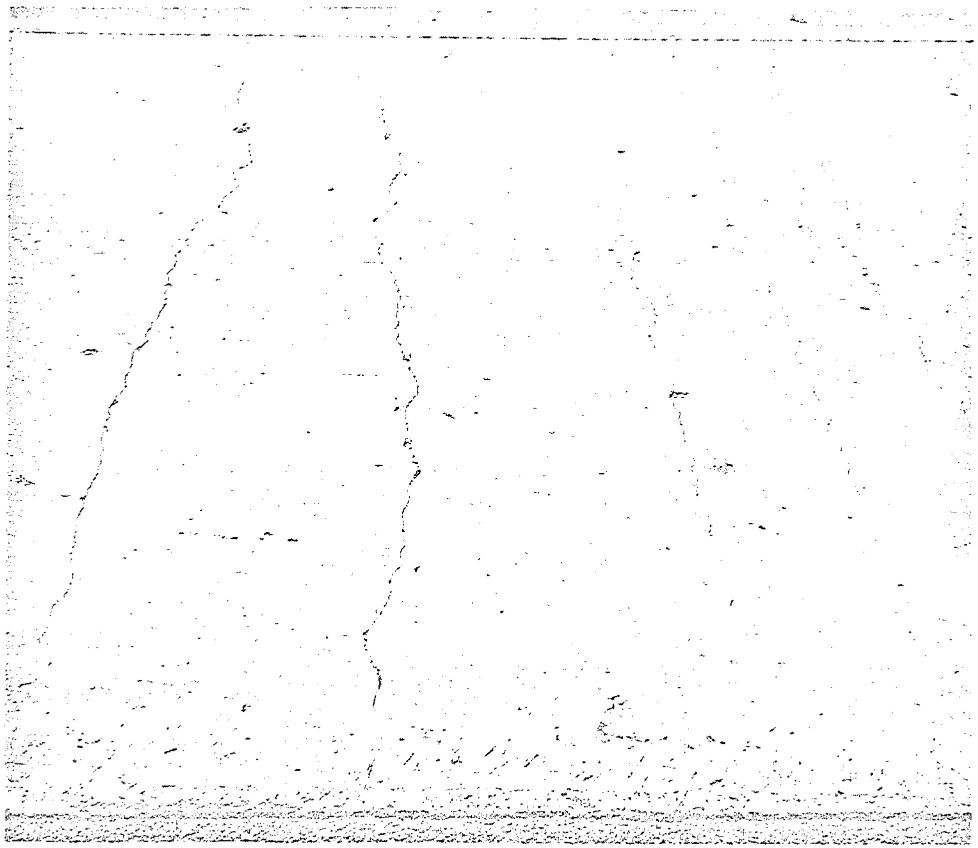


Figure 32. Transverse crack in bar mat section in the center of a day's pour.



Plain welded wire mesh reinforced pavement
Group of seven cracks

Bar mat reinforced pavement
Group of six cracks

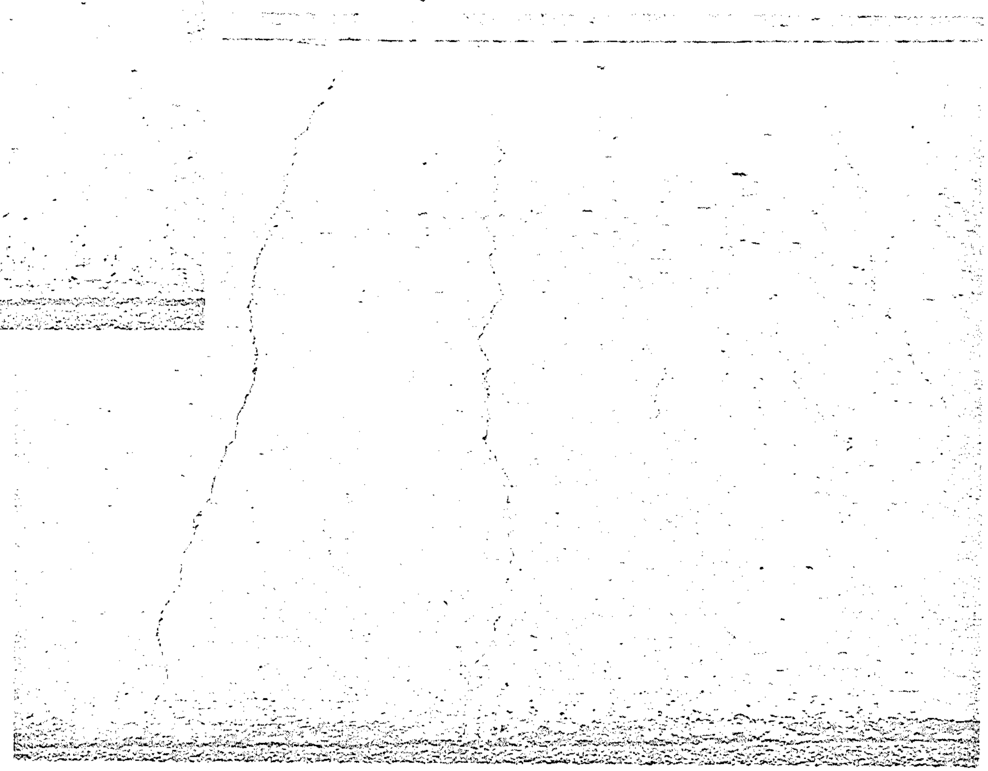


Figure 33. Appearance of cracks in area of small crack spacing in both types of reinforced pavement.

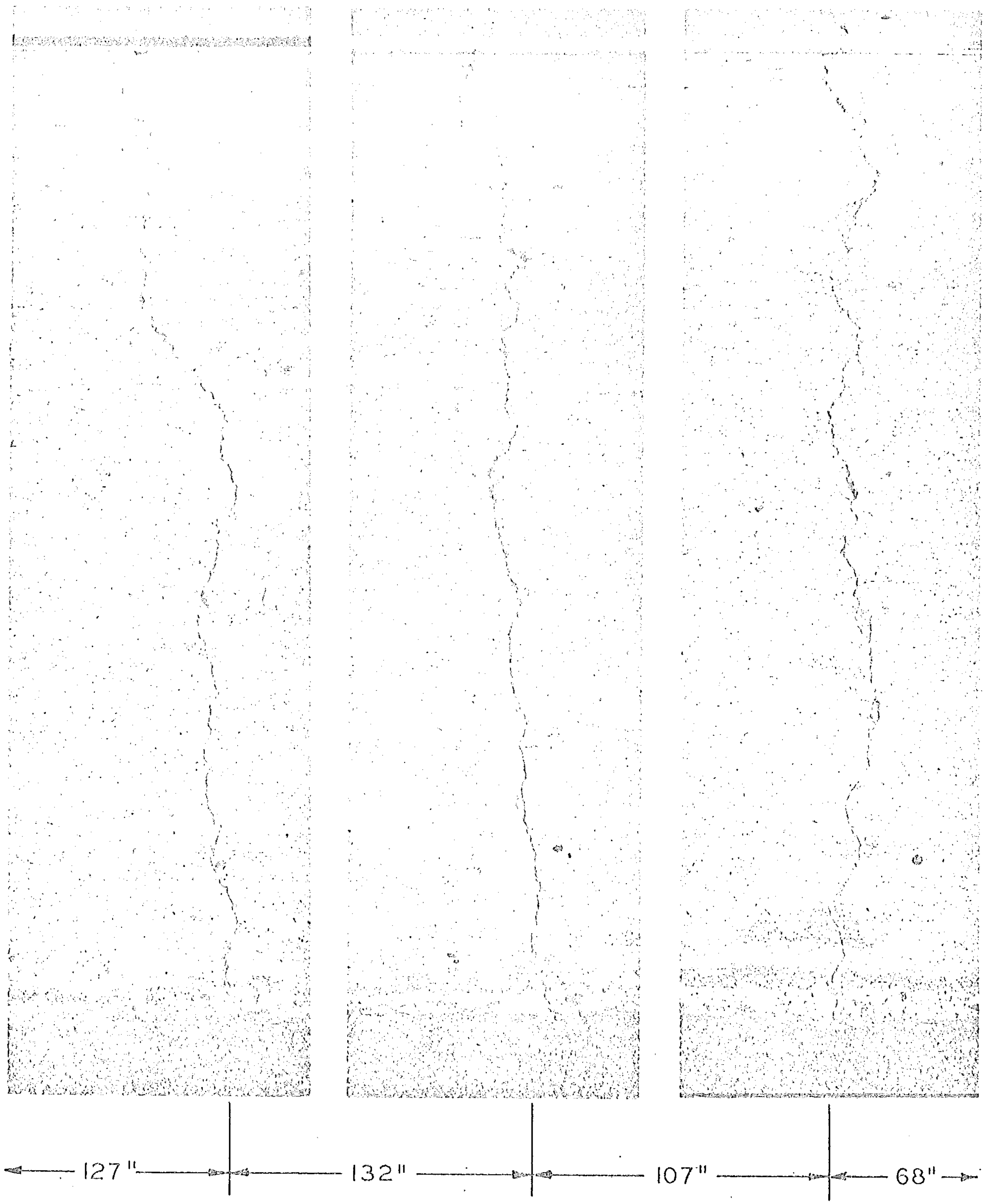


Figure 34. Appearance of cracks in area of large crack spacing in wire mesh section.

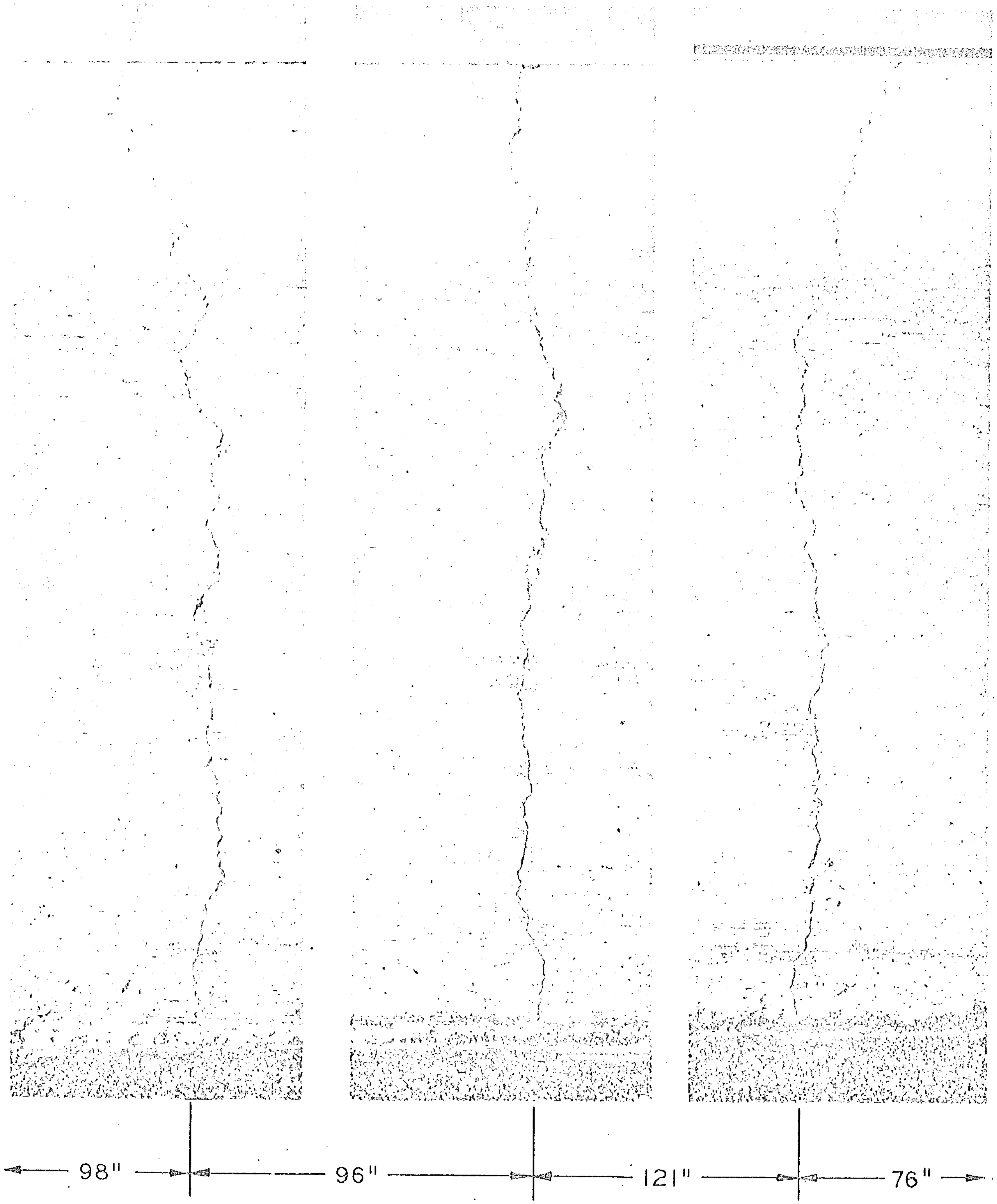


Figure 35. Appearance of cracks in area of large crack spacing in bar mat section.

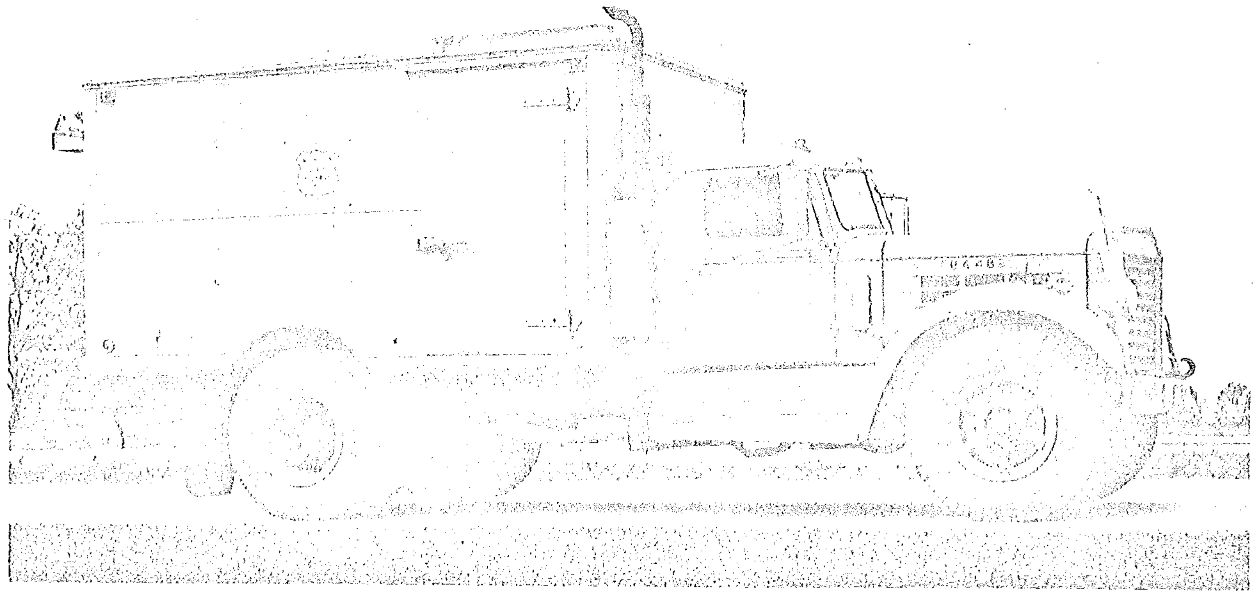


Figure 36. Two-axle load truck.

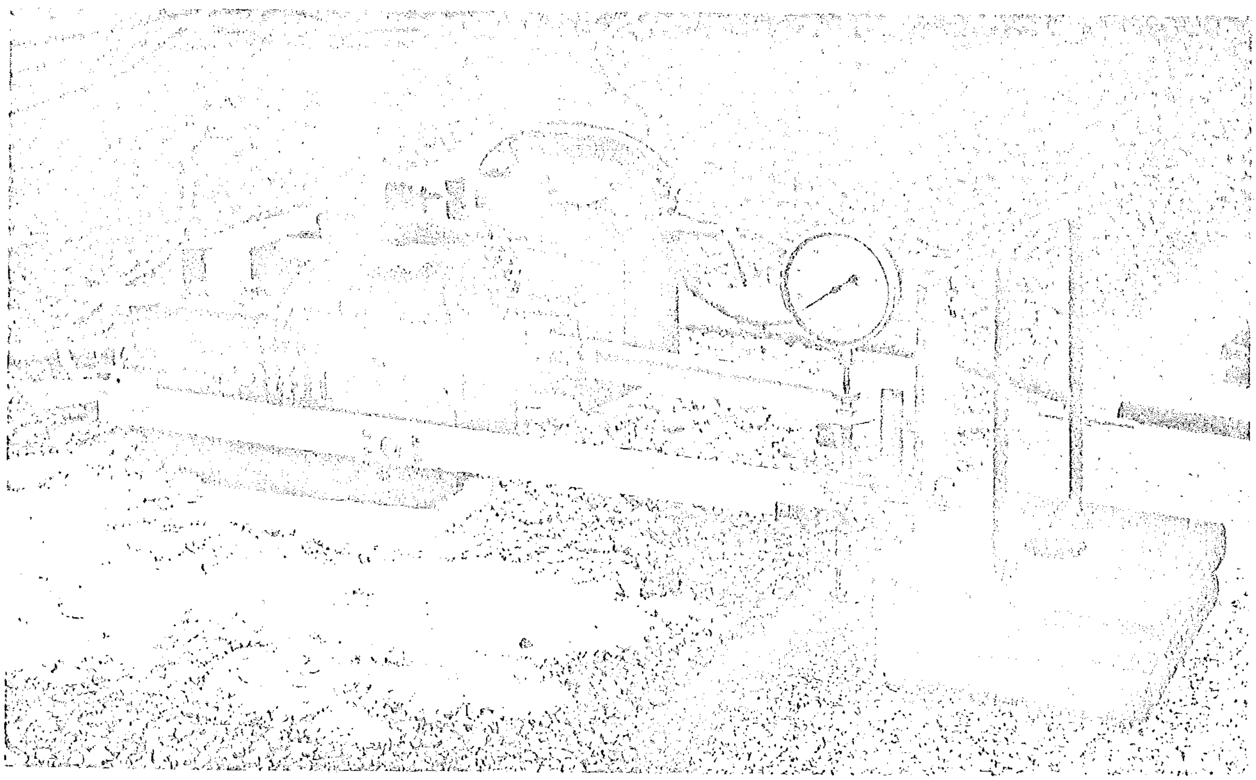


Figure 37. Deflectometer used in 1959 study (dial gage used for calibration only).

LOAD DEFLECTION TESTS

As part of the overall evaluation program, load deflection studies were performed on the experimental project in September 1959 and again in May 1962. Deflection measurements were made at cracks, midway between cracks, and at construction joints in the bar mat and welded wire mesh continuously reinforced sections. In the standard pavement section deflections were measured at contraction joints, at the midpoint of the slab, or midway between a joint and a crack. The tests were conducted for the purpose of comparing the deflections at the various points in the two types of continuously reinforced pavement sections as well as the deflections of the standard pavement section.

Equipment

A two-axle truck (Fig. 36) was employed for applying the load to the pavement during both studies. Steel blocks, weighing 1,000 lb each, and 1,200-lb concrete blocks were used as weight. Resulting static axle weights were 8,400 and 19,600 lb on the front and rear axles, respectively, during the 1959 tests and 5,000 and 17,600 lb, respectively, for the 1962 tests. The truck axle spacing was 14 ft, and a tire pressure of 70 psi was maintained in the rear tires.

For the purpose of measuring the deflections, special deflectometers were constructed. Those used in the 1959 tests (Fig. 37) consisted of an aluminum bar secured to a 50-lb steel anchor block. One end of the bar was connected to the point on the pavement where the deflection was desired. This connection was made by means of a bolt attached to the bar with a wire and a threaded nut cemented to the pavement with epoxy resin. The other end of the bar was attached to a small strain-gaged cantilever beam in such a manner that vertical movement of the slab would activate the gages.

The type of deflectometer used in the 1962 tests is shown in Figure 38. It consisted of an aluminum I-beam with a Linear Variable Differential Transformer (LVDT) fastened to one end of the beam. The deflectometer was positioned so that the LVDT core insert was resting on the pavement point where the vertical movement of the slab was to be measured. The I-beam was supported on the shoulder 6 ft from the point of deflection; whereas, the distance between the support and deflection point for the bar used in the 1959 deflectometers was 1 ft. This modification was made in order to reduce the possibility of the deflectometer being supported inside the deflection basin.

Recording was done on a four-channel Sanborn Oscillograph system. Calibration for all tests was accomplished by depressing the bar end, or in the case of the LVDT sensors moving the core a given amount and then adjusting the recording equipment for desired output.

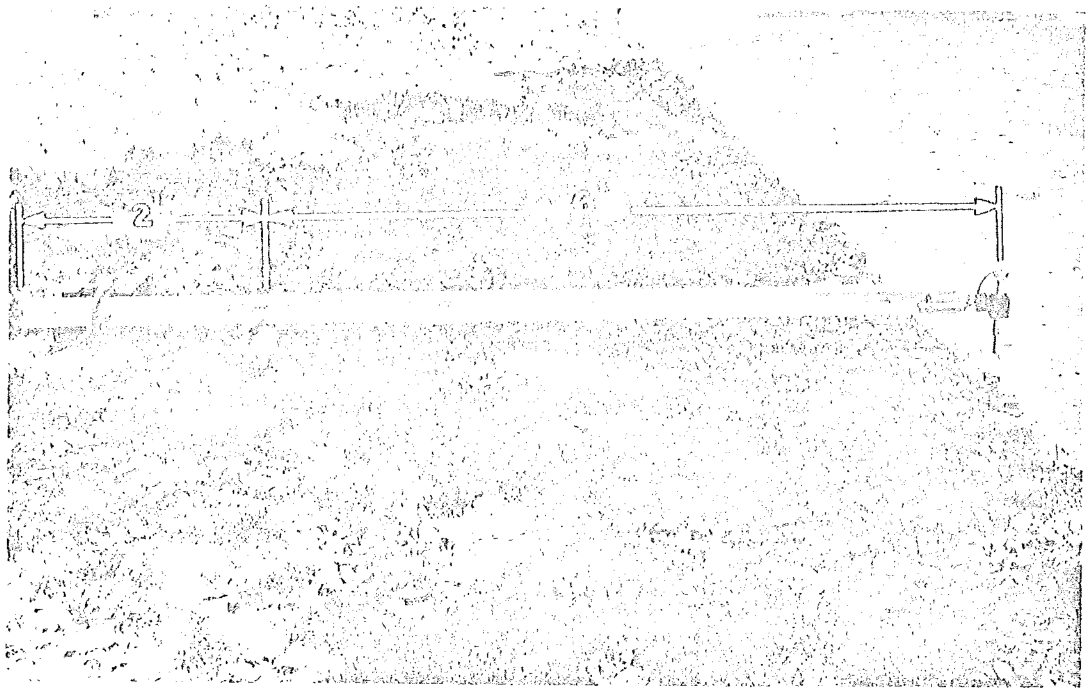


Figure 38. Deflectometer used in 1962 study.

Procedure

At each test area deflections were measured at four different locations. Two longitudinal deflectometer spacing arrangements were used in 1959: 1) a deflectometer placed 4 in. each side of a joint and 4 in. each side of the adjacent crack, 2) a deflectometer placed 4 in. each side of a crack and at the midpoint of each adjacent slab or slab segment. In the 1962 tests, the longitudinal spacing was as follows: 1) a deflectometer placed 2 in. each side of a crack or joint and at the midpoint of each adjacent slab or slab segment, 2) a deflectometer placed at the midpoints of two adjacent slab segments and directly over two adjacent cracks. In all tests the deflectometers were positioned to measure the deflection 1 to 1-1/2 in. from the pavement edge.

Before recording any deflections at any test area the pavement was conditioned or "ironed out" by four passes of the load truck. During the test runs the outside rear tire was positioned from 8 to 14 in. away from the pavement edge. Test runs in 1959 were made at both creep and 30 mph speeds and one set of data was taken during the day and one set at night. In 1962, all tests were conducted at creep speed and at night only.

For each test period the air temperature was obtained by an automatic temperature recorder and the concrete slab temperature measured at the thermocouple installation incorporated in the pavement during construction. Joint and surface crack openings were measured at the time of testing. A vernier caliper was used to obtain the joint openings and an optical micrometer used to measure crack widths.

1959 Test Results

The daytime tests were conducted from 1:15 to 4:30 p.m. and the night tests from 12:15 to 4:30 a.m. The average air temperatures for the day and night period were 86 and 62 F, respectively. During the daytime period the average slab temperature at the surface was 11 F higher than at the bottom whereas during the night period the average slab temperature at the top was 6 F lower than the bottom surface.

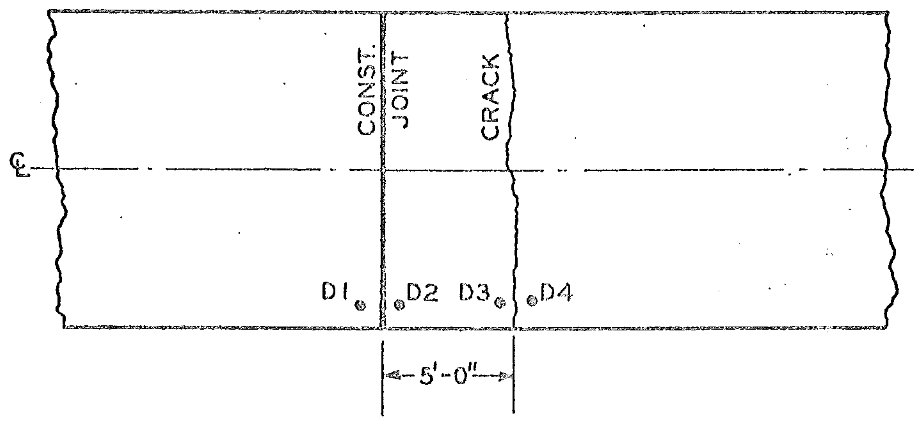
Sketches of the set-up and test area at each of five locations are shown in Figures 39 through 43, with individual and average maximum deflections at the various points given below each sketch. Figure 44 shows typical deflection traces for Trial Number 2 at Test Areas No. 3, 4, and 5 at creep speed at night. As can be seen from these traces, the maximum deflections represent the influence of the loading and axle spacing of the test vehicle utilized, and are not the result of individual axle loads.

Summaries of the average maximum deflections for the various test conditions at points along the pavement edge are presented graphically in Figures 45 and 46. Because the data are very limited in quantity and because no attempt was made to determine the subgrade properties at the individual test sites, no specific conclusions can be made with regard to the relative deflection performance of the continuously reinforced sections and the standard reinforced pavement. However, based on the data obtained, the following results of the deflection characteristics of the continuously reinforced and standard reinforced pavement sections are summarized.

TEST AREA NO. 1

Bar Mat Reinforcement

DEFLECTOMETER LOCATION



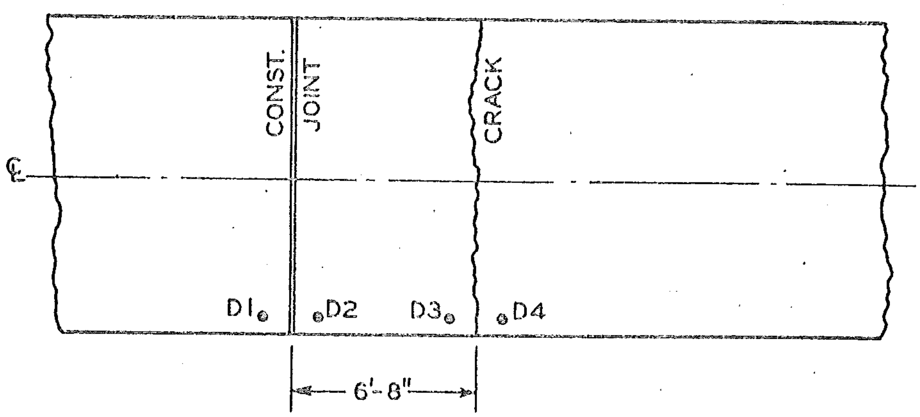
LOAD DEFLECTION DATA

	Location	Creep Trial 1	Creep Trial 2	Creep Trial 3	Creep Avg	30 mph Trial 1	30 mph Trial 2	30 mph Trial 3	30 mph Avg
DAY	D1	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
DAY	D2	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
DAY	D3	0.003	0.003	0.003	0.003	0.002	0.002	0.002	0.002
DAY	D4	0.003	0.003	0.003	0.003	0.003	0.002	0.003	0.003
DAY	Joint Opening - None					Crack Opening - None			
NIGHT	D1	0.012	0.012	0.012	0.012	0.011	0.009	0.011	0.010
NIGHT	D2	0.010	0.010	0.010	0.010	0.009	0.008	0.009	0.009
NIGHT	D3	0.011	0.012	0.011	0.011	0.010	0.008	0.010	0.009
NIGHT	D4	0.014	0.014	0.013	0.014	0.012	0.010	0.011	0.011
NIGHT	Joint Opening - 0.002					Crack Opening - 0.011			

Figure 39. Deflectometer locations and deflection data for Test Area No. 1, Sta. 894+99 westbound roadway.

TEST AREA NO. 2
Special Mesh Reinforcement

DEFLECTOMETER LOCATION



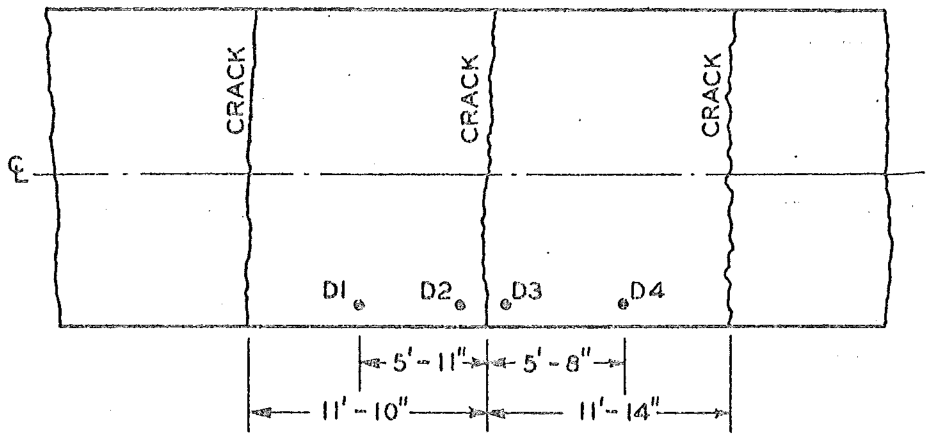
LOAD DEFLECTION DATA

	Location	Creep				30 mph			
		Trial 1	Trial 2	Trial 3	Avg	Trial 1	Trial 2	Trial 3	Avg
DAY	D1	0.005	0.005	0.004	0.005	0.005	0.005	0.004	0.005
	D2	0.006	0.006	0.005	0.006	0.005	0.005	0.005	0.005
	D3	0.003	0.003	0.003	0.003	0.002	0.002	0.002	0.002
	D4	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003
	Joint Opening - 0.013					Crack Opening - 0.007			
NIGHT	D1	0.023	0.024	0.023	0.023	0.020	0.019	0.020	0.020
	D2	0.020	0.020	0.020	0.020	0.016	0.016	0.017	0.016
	D3	0.021	0.021	0.020	0.021	0.017	0.017	0.017	0.017
	D4	0.022	0.022	0.022	0.022	0.019	0.017	0.018	0.018
	Joint Opening - 0.021					Crack Opening - 0.008			

Figure 40. Deflectometer locations and deflection data for Test Area No. 2, Sta. 890+16 eastbound roadway.

TEST AREA NO. 3
Special Mesh Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

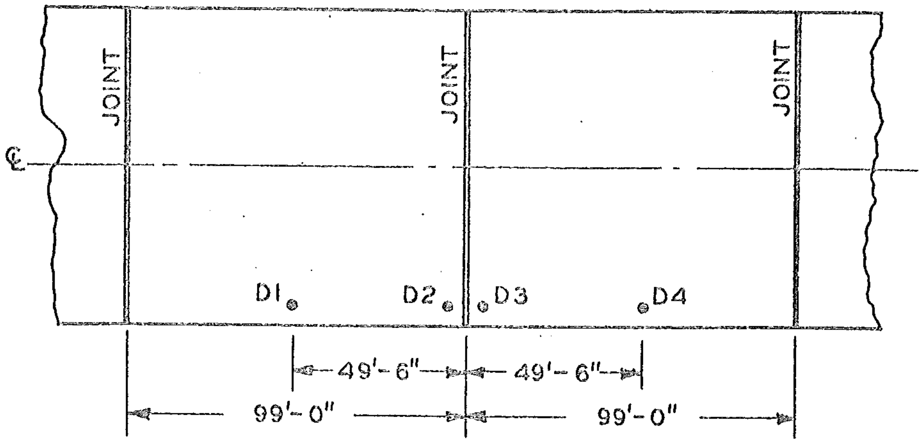
	Location	Creep				30 mph			
		Trial 1	Trial 2	Trial 3	Avg	Trial 1	Trial 2	Trial 3	Avg
DAY	D1	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003
	D2	0.006	0.006	0.006	0.006	0.007	0.006	0.007	0.007
	D3	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	D4	0.004	0.004	0.004	0.004	0.004	0.003	0.004	0.004
	Joint Opening - None					Crack Opening - 0.008			
NIGHT	D1	0.018	0.018	0.019	0.018	0.015	0.015	0.015	0.015
	D2	0.021	0.021	0.022	0.021	0.017	0.017	0.017	0.017
	D3	0.019	0.020	0.020	0.020	0.015	0.015	0.016	0.015
	D4	0.018	0.018	0.019	0.018	0.014	0.014	0.014	0.014
	Joint Opening - None					Crack Opening - 0.008			

Figure 41. Deflectometer locations and deflection data for Test Area No. 3, Sta. 930+00 eastbound roadway.

TEST AREA NO. 4

Standard Reinforcement

DEFLECTOMETER LOCATION



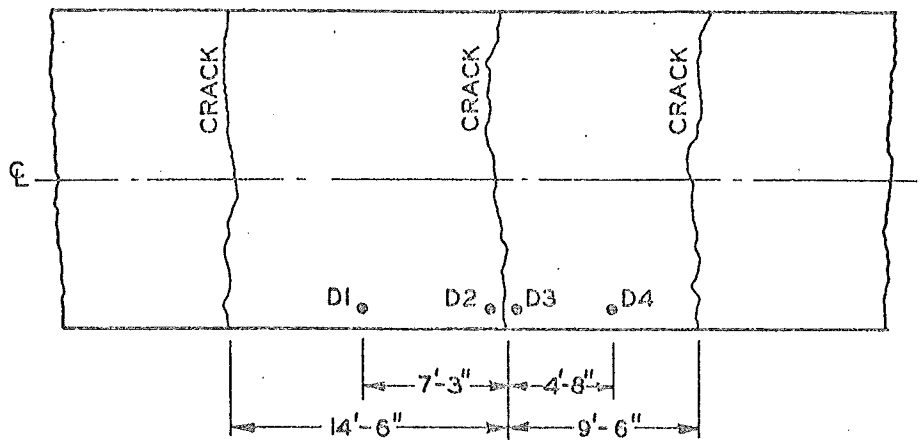
LOAD DEFLECTION DATA

	Location	Creep Trial 1	Creep Trial 2	Creep Trial 3	Creep Avg	30 mph Trial 1	30 mph Trial 2	30 mph Trial 3	30 mph Avg
DAY	D1	0.004	0.004	0.004	0.004	0.005	0.004	0.005	0.005
DAY	D2	0.009	0.009	0.009	0.009	0.008	0.008	0.008	0.008
DAY	D3	0.009	0.009	0.009	0.009	0.009	0.009	0.009	0.009
DAY	D4	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004
DAY	Joint Opening - 0.50					Crack Opening - None			
NIGHT	D1	0.019	0.019	0.018	0.019	0.015	0.015	0.016	0.015
NIGHT	D2	0.052	0.053	0.053	0.053	0.040	0.039	0.041	0.040
NIGHT	D3	0.057	0.056	0.056	0.056	0.044	0.043	0.046	0.044
NIGHT	D4	0.020	0.019	0.019	0.019	0.015	0.014	0.016	0.015
NIGHT	Joint Opening - 0.50					Crack Opening - None			

Figure 42. Deflectometer locations and deflection data for Test Area No. 4, Sta. 973+20 eastbound roadway.

TEST AREA NO. 5
Bar Mat Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

	Location	Creep				30 mph			
		Trial 1	Trial 2	Trial 3	Avg	Trial 1	Trial 2	Trial 3	Avg
DAY	D1	0.005	0.004	0.004	0.004	0.004	0.004	0.004	0.004
	D2	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007
	D3	0.009	0.009	0.009	0.009	0.008	0.008	0.008	0.008
	D4	0.004	0.003	0.004	0.004	0.004	0.003	0.004	0.004
	Joint Opening - None					Crack Opening - 0.009			
NIGHT	D1	0.012	0.012	0.012	0.012	0.011	0.010	0.010	0.010
	D2	0.018	0.018	0.018	0.018	0.017	0.016	0.016	0.016
	D3	0.019	0.019	0.019	0.019	0.018	0.017	0.017	0.017
	D4	0.014	0.014	0.013	0.014	0.013	0.012	0.012	0.012
	Joint Opening - None					Crack Opening - 0.011			

Figure 43. Deflectometer locations and deflection data for Test Area No. 5, Sta. 1029+63 eastbound roadway.

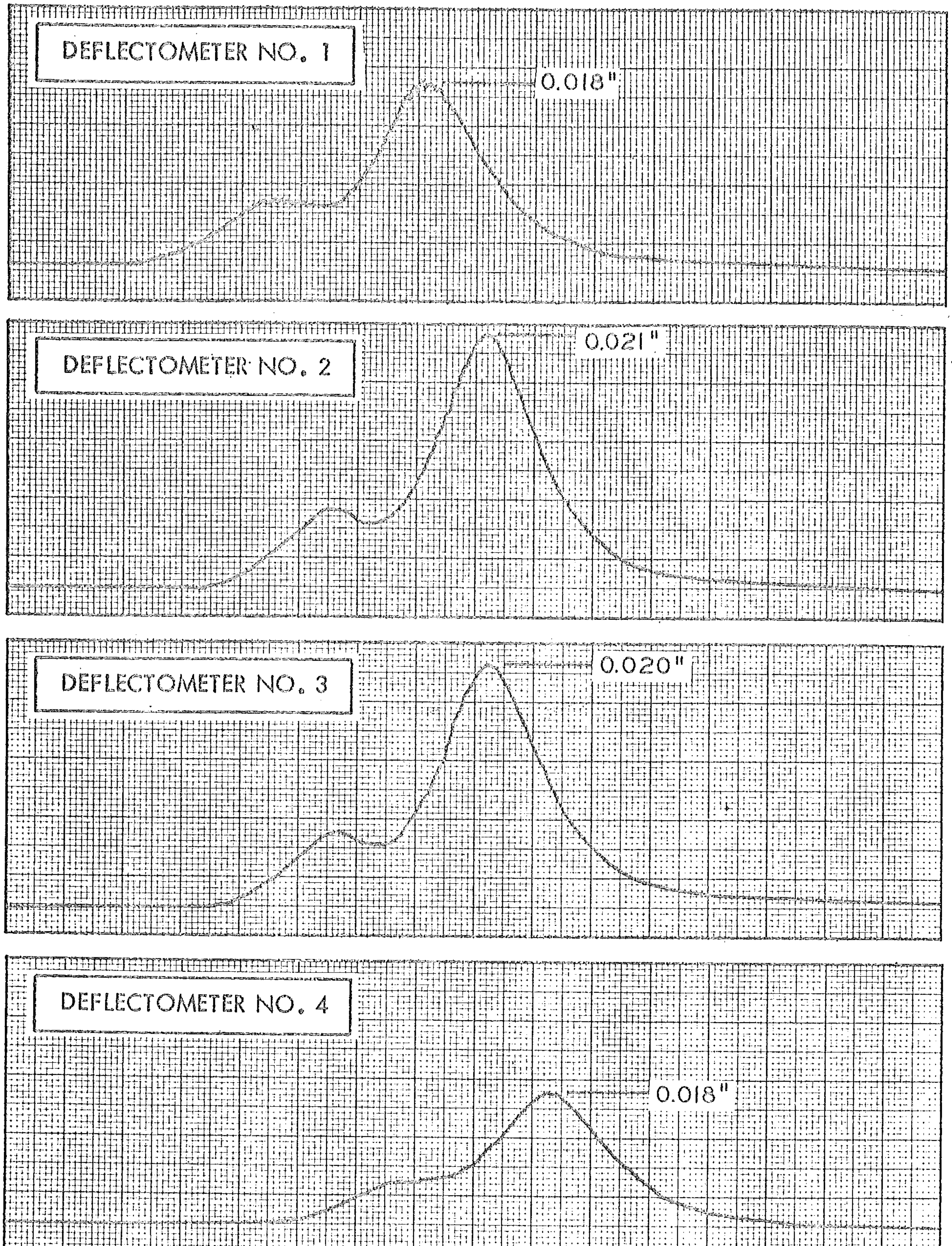


Figure 44. Deflection Test No. 2, Test Area No. 3.

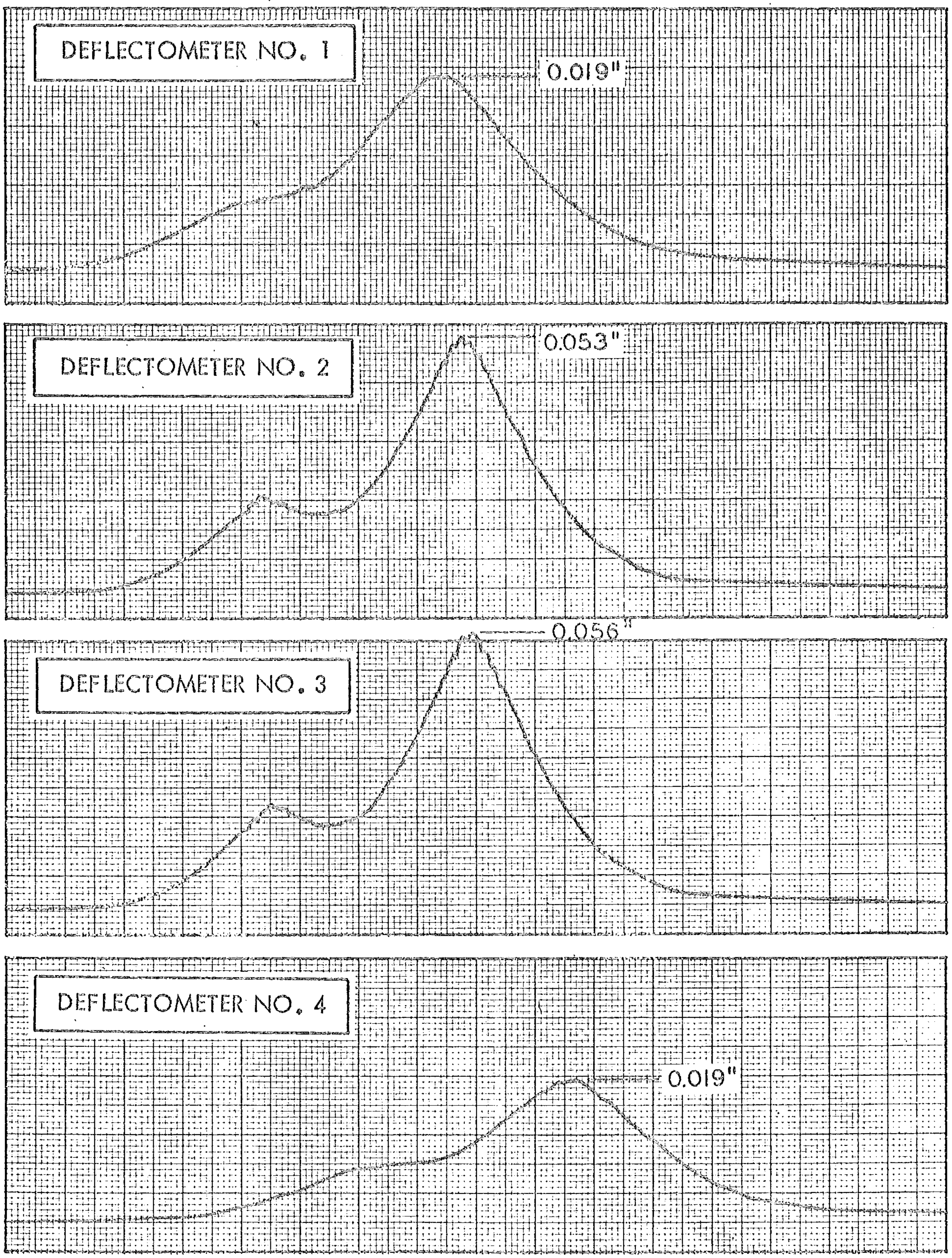


Figure 44 (cont.). Deflection Test No. 2, Test Area No. 4.

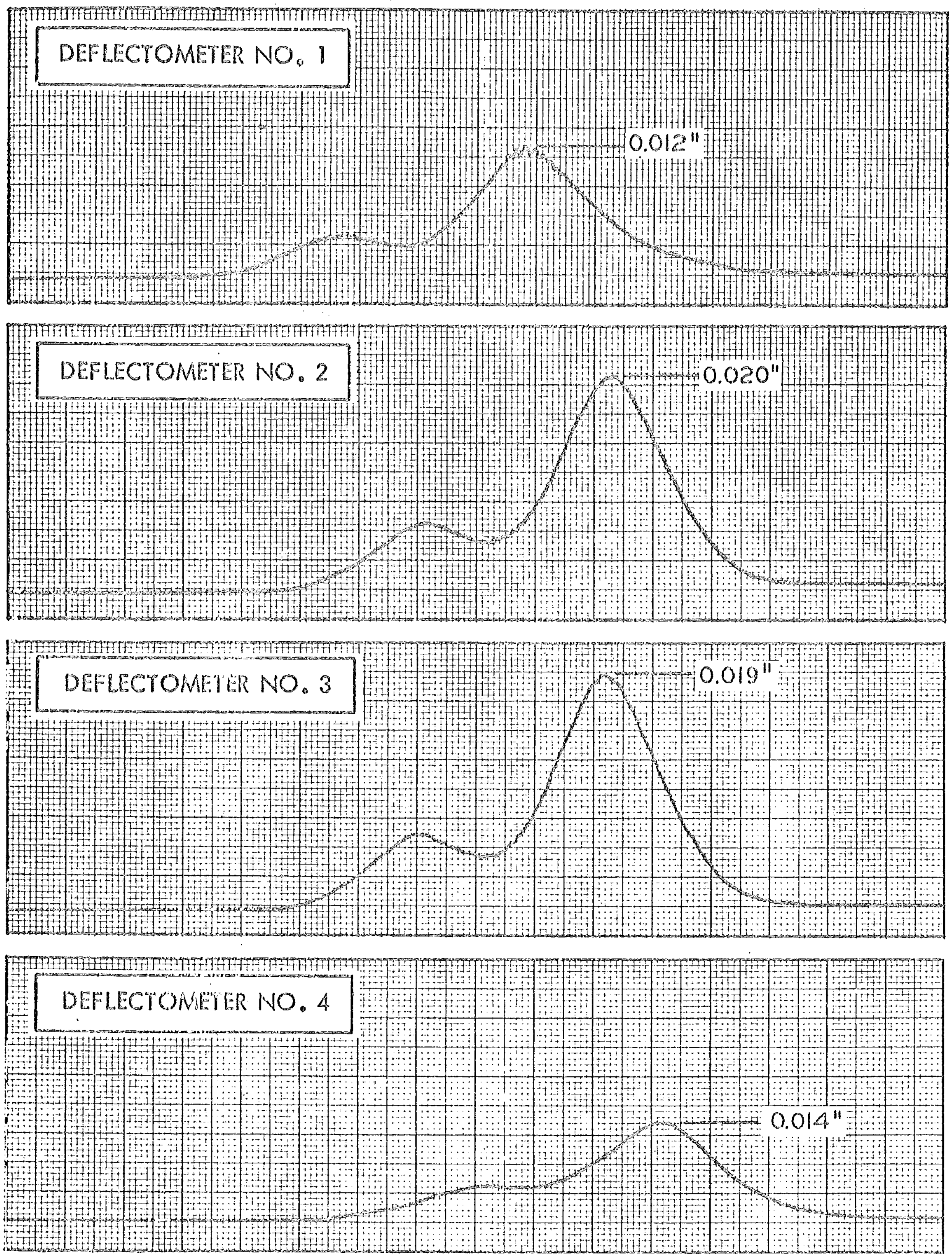


Figure 44 (cont.). Deflection Test No. 2, Test Area No. 5.

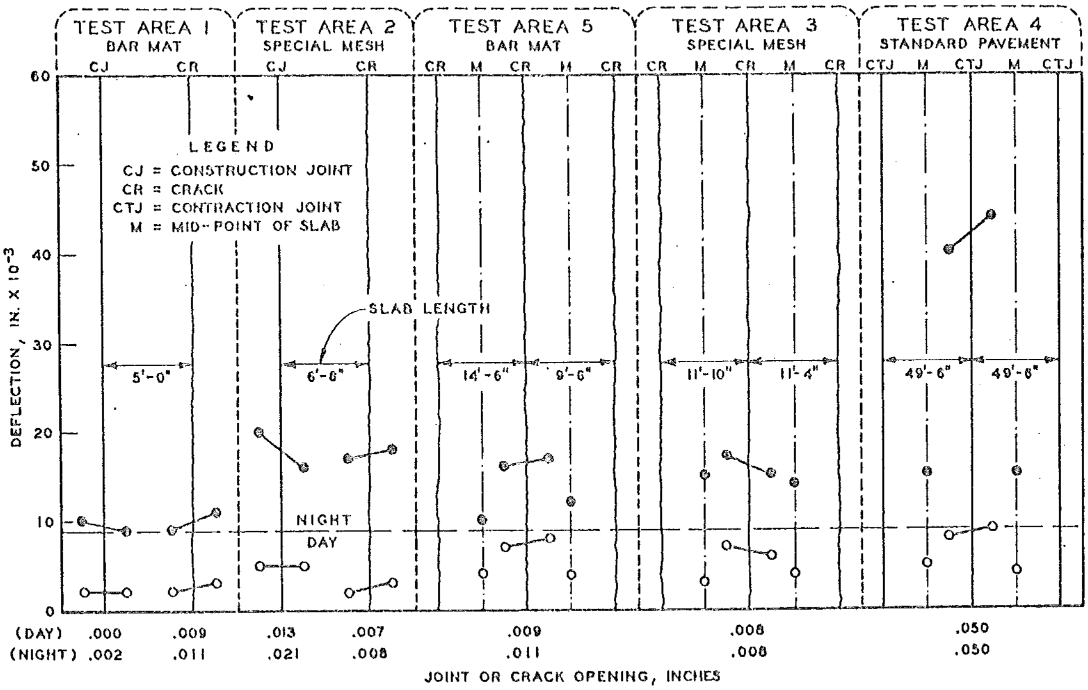


Figure 45. Average maximum deflection at various points along pavement edge during day and night tests at 30 mph.

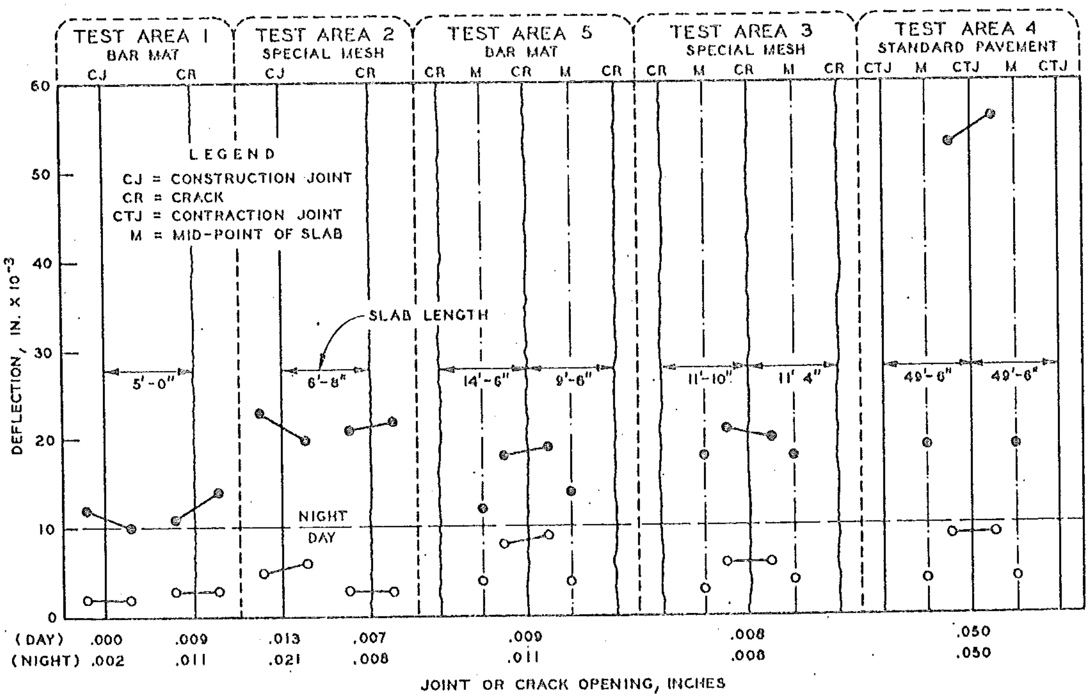


Figure 46. Average maximum deflection at various points along pavement edge during day and night tests at creep speed.

1. As expected, because of the pavement being warped upward during the night, the deflections resulting from the daytime tests are substantially lower than those recorded at night. The day measurements for the 20 deflectometer positions in the five test areas ranged from 0.002 to 0.009 in. whereas at night the range extended from 0.009 in. to a high of 0.056 in. at a joint in the standard pavement section.

2. Increase in speed from creep to 30 mph had practically no effect on the magnitude of the daytime deflection measurements at any of the five test areas. However, a reduction in deflection magnitude occurred at the 30 mph speed during the nighttime tests. Based on the deflections of all twenty points measured, the average reduction was 19 percent with a minimum of 10 percent occurring in Test Area No. 1 and a maximum of 24 percent at the contraction joint in Test Area No. 4.

3. The average maximum relative deflection at cracks and construction joints in the bar mat and wire reinforced continuous sections as well as at cracks in the standard jointed section was 0.001 in. for the daytime loadings and ranged from 0.001 in. to 0.004 in. for the night loadings.

4. The average deflection at cracks in the bar mat reinforced sections ranged from 0.005 in. for the day loadings to 0.014 in. for the night loadings. The average deflection at cracks in the wire mesh reinforced sections ranged from 0.004 in. to 0.019 in. for the day and night loadings, respectively.

5. The average deflection at construction joints in the bar mat reinforced sections ranged from 0.002 in. for the day loadings to 0.010 in. for the night loadings. The average deflection at construction joints in the wire mesh reinforced sections ranged from 0.005 in. to 0.020 in. for the day and night loadings, respectively.

6. The average deflection at contraction joints in the standard sections ranged from 0.009 in. for the day loadings to 0.048 in. for the night loadings.

7. The average deflection at points halfway between cracks in the bar mat, wire mesh, and standard reinforced sections for day and night loadings ranged from 0.004 to 0.012 in., 0.004 to 0.016 in., and 0.004 to 0.017 in., respectively.

1962 Test Results

As previously mentioned the 1962 load-deflection tests were conducted at creep speed and at night only. Two test periods were required to complete the scheduled tests; one from 11:45 p.m. to 3:50 a.m. and one from 12:30 a.m. to 2:00 a.m. on the following night. The average air temperature during the test periods was 42 F with a high of 45 and a low of 39 F. Recorded slab temperatures show that the slab surface was from 3 to 5 F cooler than the bottom surface during the test intervals.

Seven areas were selected for test; three in the bar mat reinforced section, three in the special mesh reinforced section, and one in the standard pavement section. In each test area, the deflectometer locations and deflection data for each point and trial loading are shown in Figures 47 through 53. Test Areas No. 1 and 5 are at the free end of a section of special mesh and bar mat reinforced pavement and, as shown in the sketches, two of the four deflectometers used at each location were placed directly over a crack. The remaining four areas in the continuously reinforced section are in the middle portion of a day's pour and were selected to obtain information on deflection with respect to different slab lengths as well as across cracks. Deflections at a contraction joint and at the slab midpoint in the standard pavement were measured for comparing the deflections of the two pavement types.

A typical load-deflection trace from Trial Number 4 at Test Area No. 7 is shown in Figure 54. Average deflections at various points along the pavement edge for each test area are plotted in Figure 55. As in the case of the 1959 tests the data are very limited and do not warrant specific conclusions; however, a summary of the data obtained is as follows:

1. The smallest deflections (0.008 to 0.014 in.) were measured in Test Areas No. 1 and 5 which are at the free ends of a continuously reinforced section which has relatively long slab segments and narrow crack openings. The average relative deflection across the cracks was 0.001 in., and the average midpoint slab deflection was 0.015 in. less (about 15 percent) than the average deflection at cracks. There was very little difference between the deflections measured in the bar mat and wire mesh reinforced areas.

2. In Test Areas No. 2 and 7, having intermediate slab lengths (3 to 7 ft) the average relative deflection across the cracks was less than 0.001 in. The average midpoint slab deflection was 0.028 in. less (about 15 percent) than the average deflection at the cracks. There was no significant difference between the deflections occurring in either the bar mat or wire reinforced sections.

3. The bar mat reinforced test area with slab segments in the short category (less than 2.5 ft) deflected equally at the crack and at the slab midpoints. These deflections measured 0.016 in. which were 0.003 in. less (about 16 percent) than the deflections at cracks in the areas with medium slab length segments and about the same for points midway between cracks in the long slab segment area. The special mesh reinforcement test area with short slab segments (less than 2.5 ft) deflected an average of 0.039 in. at the crack and 0.034 in. at the slab midpoints or about twice as much as other areas in the continuously reinforced pavement sections. There was no indication that the structural quality of the pavement at this location was inferior to that at other test locations and it is, therefore, anticipated that the subgrade support was less at this particular location.

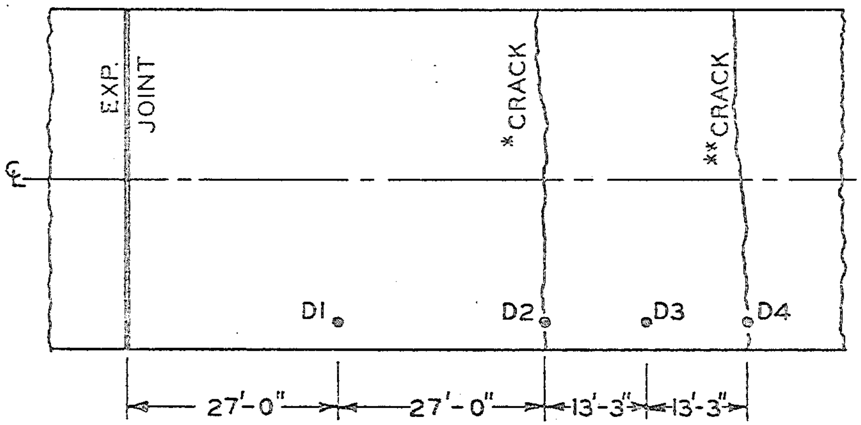
4. Slab midpoint deflections in the standard pavement were in the same general deflection range (0.018 in.) as deflections in the continuously reinforced sections, whereas deflections at the contraction joint (0.041 in.) were more than twice as much as the greatest deflection of any point measured in the continuously reinforced pavement test areas.

Summary

A qualitative assessment of the results of both of the above series of deflection tests indicates that the deflections at cracks, at points between cracks, and at construction joints in the continuously reinforced 8-in. pavement sections were essentially compatible for the various speeds and day and night loadings involved. There is no significant difference in the deflection behavior in the continuous sections reinforced with either type of reinforcement and, with the exception of deflections at the contraction joints in the standard jointed pavement, the deflection at cracks and at points between cracks are in the same range for both the continuous sections and the standard jointed section. The relative deflections across cracks in the continuous sections were small, with load transfer ratios ranging from 45 percent to an optimum 50 percent for all loadings involved. The relative deflections across cracks, and the percentage deflection increase at cracks over points between cracks for the short, intermediate, and long slab segments were essentially the same.

TEST AREA NO. 1
Special Mesh Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

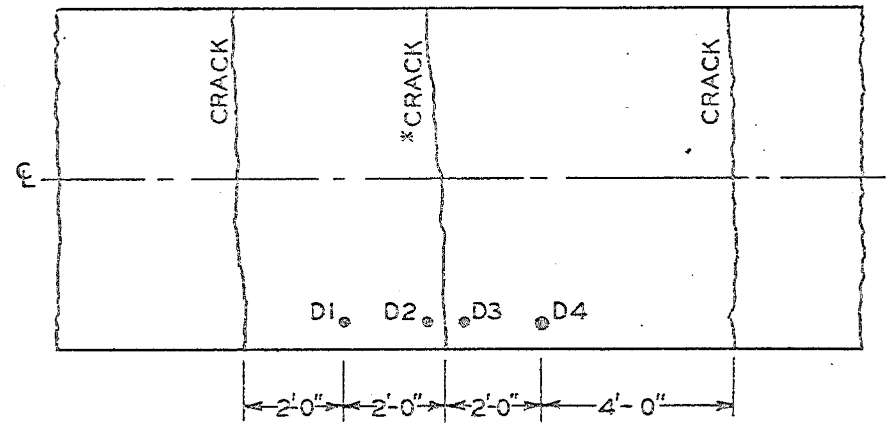
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.008	0.008	0.008	0.008	0.008
D2	0.014	0.013	0.014	0.014	0.014
D3	0.013	0.013	0.013	0.013	0.013
D4	0.012	0.012	0.012	0.011	0.012

*Crack Opening - 0.002 **Crack Opening - 0.003

Figure 47. Deflectometer locations and deflection data for Test Area No. 1, Sta. 943+49 eastbound roadway.

TEST AREA NO. 2
Special Mesh Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

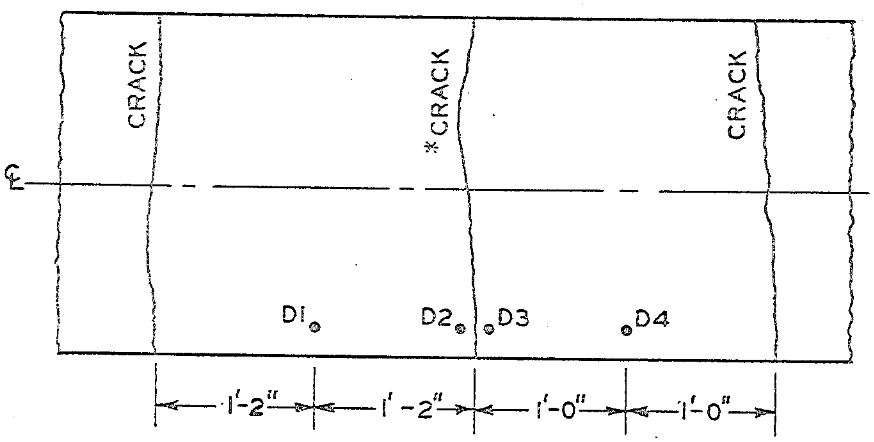
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.017	0.017	0.017	0.018	0.017
D2	0.019	0.019	0.019	0.019	0.019
D3	0.018	0.019	0.019	0.019	0.019
D4	0.014	0.014	0.014	0.014	0.014

*Crack Opening - 0.008

Figure 48. Deflectometer locations and deflection data for Test Area No. 2, Sta. 854+71 eastbound roadway.

TEST AREA NO. 3
Special Mesh Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

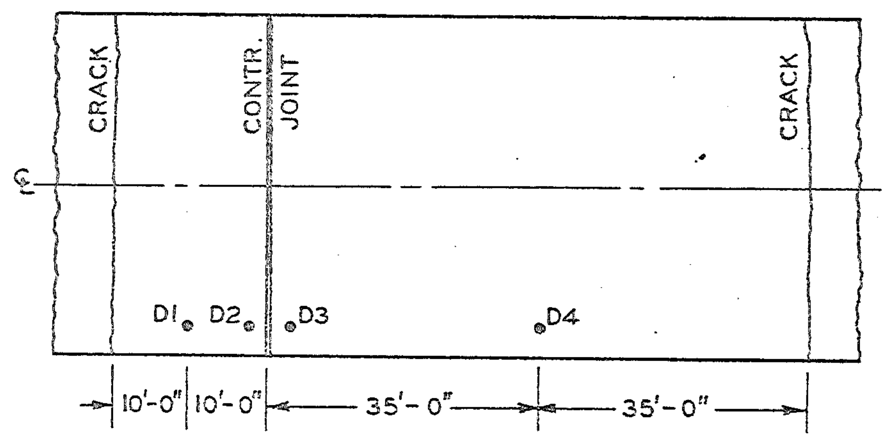
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.033	0.032	0.032	0.032	0.032
D2	0.037	0.038	0.038	0.037	0.038
D3	0.040	0.040	0.040	0.039	0.040
D4	0.037	0.036	0.036	0.036	0.036

*Crack Opening - 0.028

Figure 49. Deflectometer locations and deflection data for Test Area No. 3, Sta. 862+59 eastbound roadway.

TEST AREA NO. 4
Standard Mesh Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

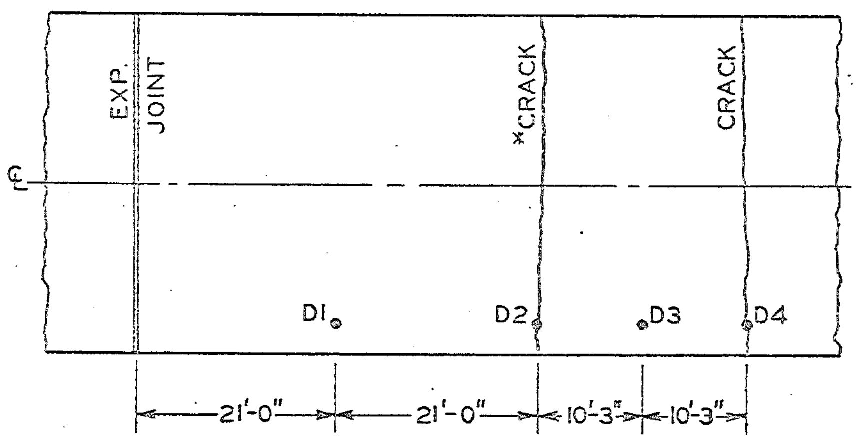
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.016	0.017	0.016	0.016	0.016
D2	0.042	0.042	0.042	0.042	0.042
D3	0.042	0.042	0.042	0.043	0.042
D4	0.018	0.018	0.017	0.017	0.018

Joint Opening - 0.625

Figure 50. Deflectometer locations and deflection data for Test Area No. 4, Sta. 982+12 eastbound roadway.

TEST AREA NO. 5
Bar Mat Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

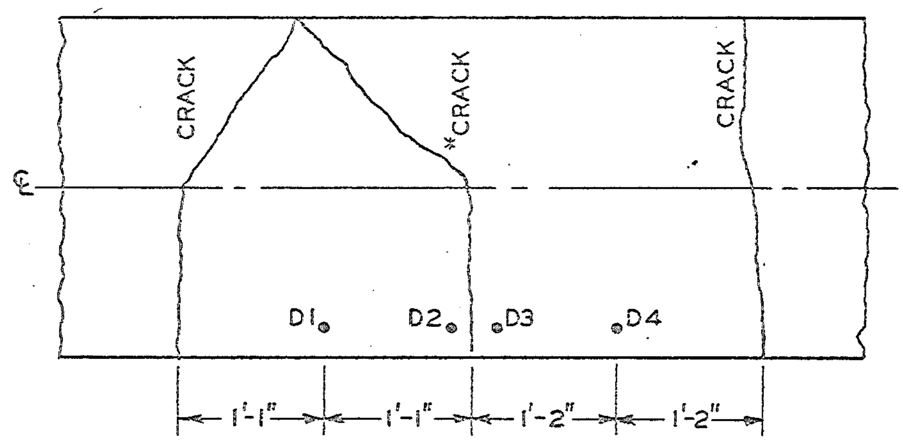
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.014	0.014	0.014	0.014	0.014
D2	0.012	0.013	0.013	0.013	0.013
D3	0.014	0.015	0.014	0.014	0.014
D4	0.014	0.015	0.014	0.015	0.014

*Crack Opening - 0.002

Figure 51. Deflectometer locations and deflection data for Test Area No. 5, Sta. 996+82 eastbound roadway.

TEST AREA NO. 6
Bar Mat Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

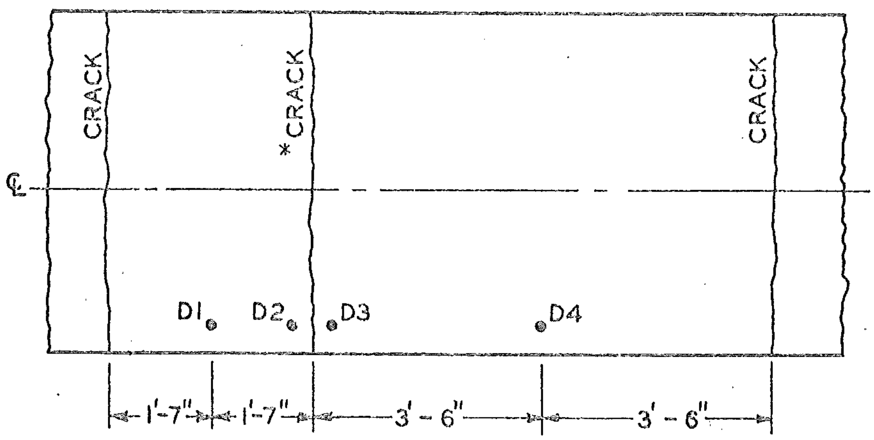
Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.016	0.016	0.016	0.016	0.016
D2	0.016	0.016	0.016	0.016	0.016
D3	0.016	0.016	0.016	0.016	0.016
D4	0.016	0.016	0.016	0.016	0.016

*Crack Opening - 0.024

Figure 52. Deflectometer locations and deflection data for Test Area No. 6, Sta. 1006+25 eastbound roadway.

TEST AREA NO. 7
Bar Mat Reinforcement

DEFLECTOMETER LOCATION



LOAD DEFLECTION DATA

Location	Trial 1	Trial 2	Trial 3	Trial 4	Avg
D1	0.019	0.019	0.018	0.019	0.019
D2	0.020	0.020	0.020	0.020	0.020
D3	0.019	0.019	0.019	0.019	0.019
D4	0.016	0.016	0.016	0.016	0.016
	*Crack Opening - 0.028				

Figure 53. Deflectometer locations and deflection data for Test Area No. 7, Sta. 1007+25 eastbound roadway.

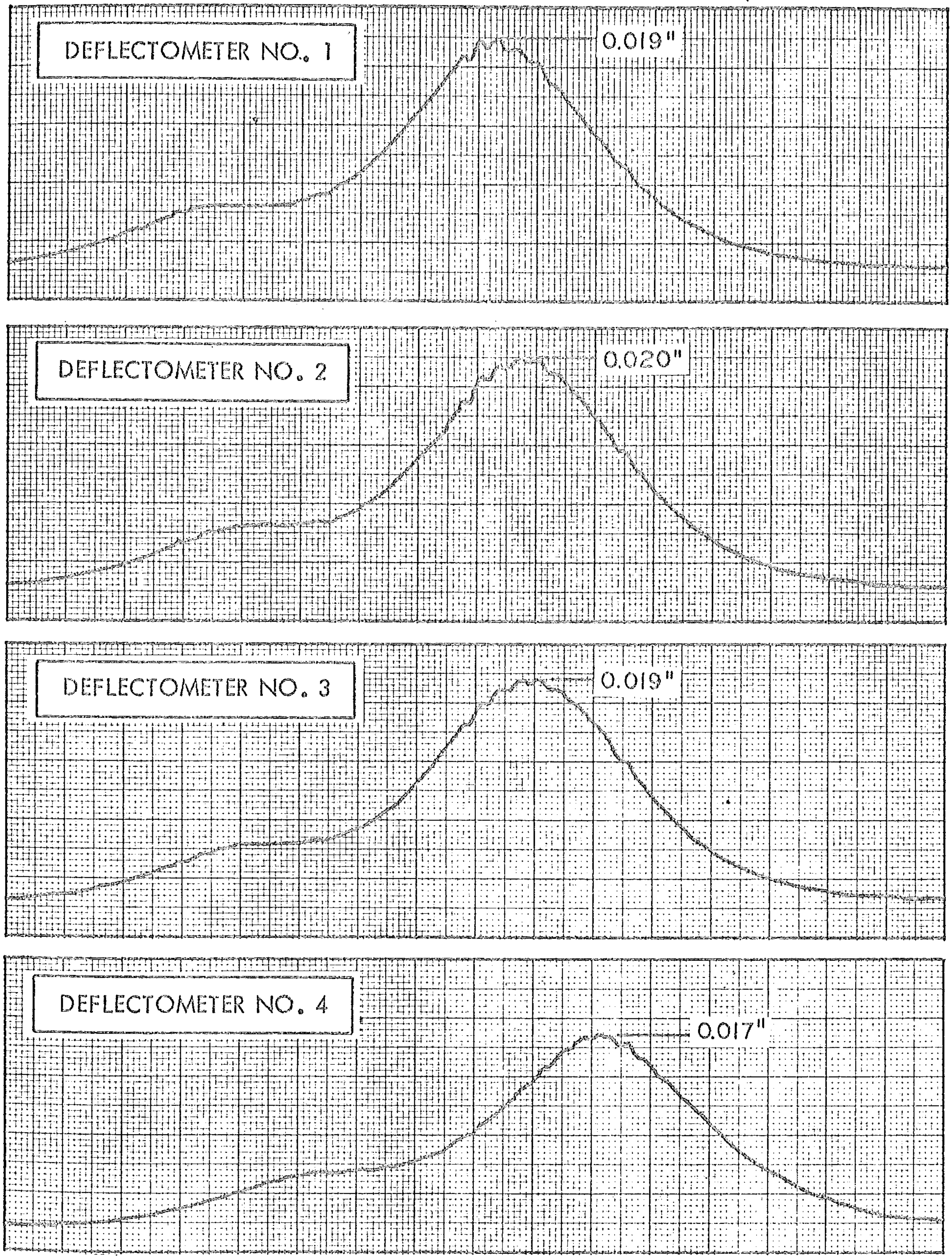


Figure 54. Typical load deflection trace from Trial 4, Test Area No. 7 at creep speed.

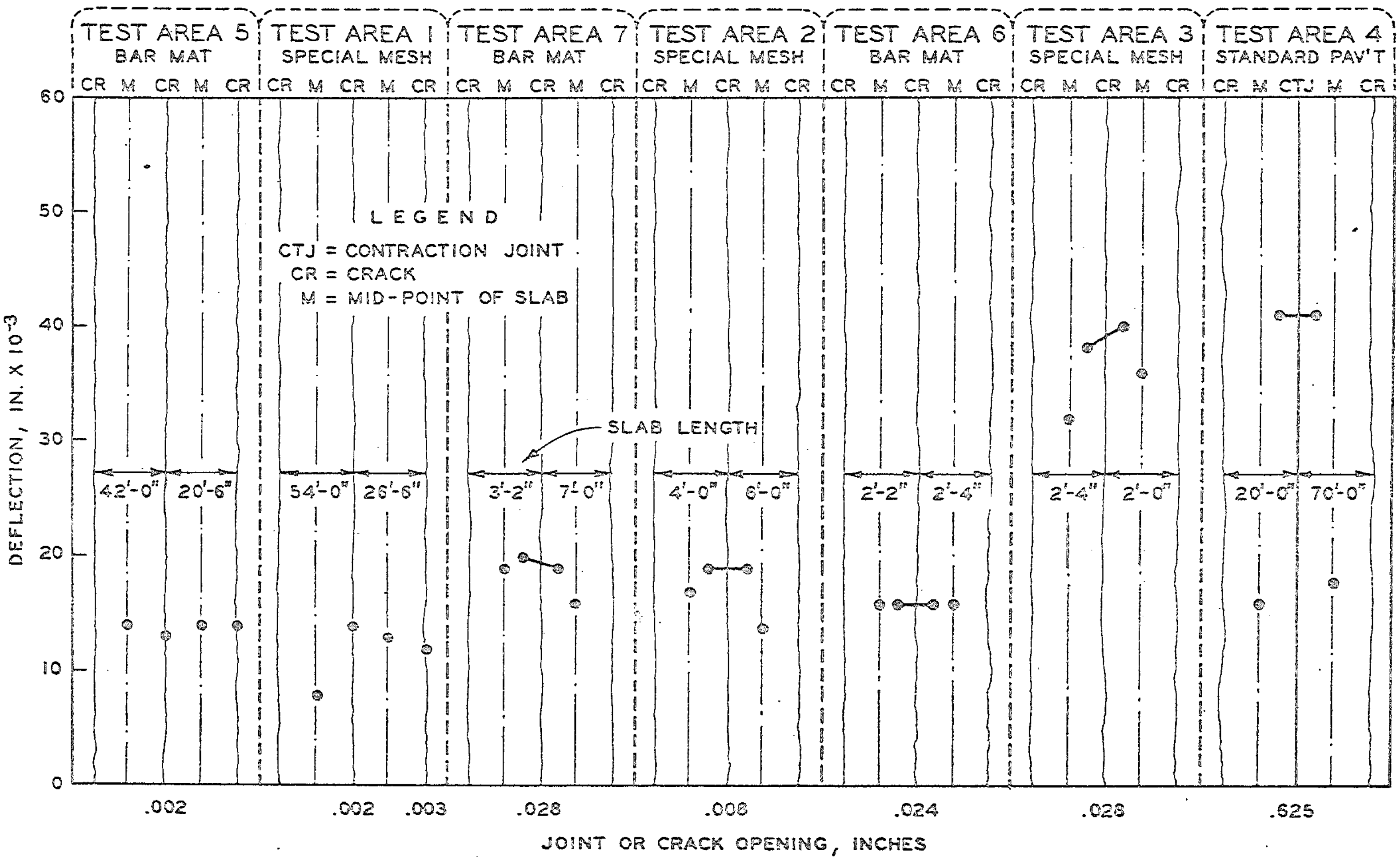


Figure 55. Average maximum deflection at various points along pavement edge during night tests at creep speed.

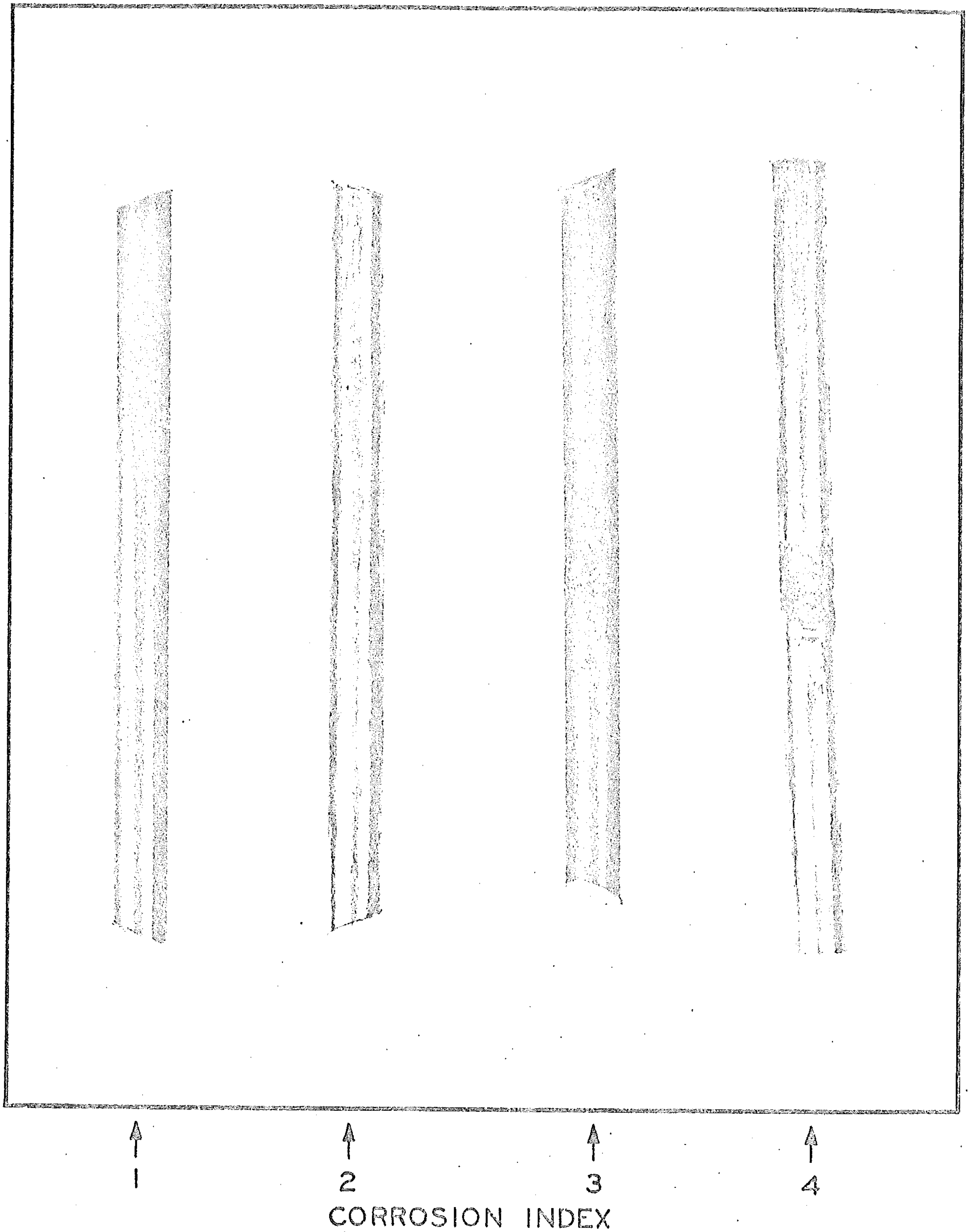


Figure 56. Steel samples showing the first four stages of corrosion.

REINFORCEMENT CONDITION

To maintain the continuity of a continuously reinforced pavement for its design life it is necessary that the reinforcing steel remain structurally sound. As previously mentioned, steel removed from repairs performed in 1962 was--in some cases--excessively corroded at crack locations. Although the corrosion had occurred at wide cracks within the failed area, it generated concern as to the condition of the steel at cracks of normal width. As a result several cores have been taken periodically for the purpose of checking the condition of the steel. A set of twelve cores were taken in 1963, six in 1966, and eight in 1968 through typical cracks throughout the experimental pavement. Periodic coring will be continued to monitor the condition of the steel reinforcement.

Unfortunately it is difficult to develop a meaningful procedure for determining the degree and effect of corrosion that has occurred on reinforcing steel. In an attempt to establish consistency for rating and comparing the corrosion attack the corrosion scale given below was devised. Samples of plain wires removed from cores are shown in Figure 56 to illustrate the different indices of corrosion. These samples were wire brushed before photographing.

Corrosion Scale

Index	Definition
1	No corrosion. Discoloration of the surface of the bar may have occurred.
2	Mild corrosion. Scattered minute pitting of the bar surface.
3	Moderate corrosion. Concentrated pitting of the bar surface with pit depths up to approximately 1/32 in., and/or the beginning of slight uniform reduction of the bar diameter.
4	Severe corrosion. Concentrated pitting of the bar surface with pit depths greater than approximately 1/32 in., and/or uniform reduction of the bar diameter up to approximately 25 percent.
5	Critical corrosion. Deep concentrated pitting of the bar surface, and/or uniform reduction of the bar diameter from over approximately 25 percent to failure of the bar.

TABLE 5
CORROSION CONDITION OF REINFORCEMENT

Date Cored	Crack Spacing Interval, ft	Reinforcement Type	Core No.	Crack Width at Surface	Crack Width at Steel	Corrosion Index
September 1963	8-12	Bar Mat	3	.070	.020	3
			9	.050	.010	2
		Mesh	6	.045	.015	2
			12	.045	.010	2
	3-8	Bar Mat	1	.045	.015	2
			8	.025	.010	2
		Mesh	4	.035	.020	4
			10	.025	.015	4
	1-3	Bar Mat	7	.015	.010	2
			2	.005	----	1
		Mesh	5	.025	.010	3
			11	.010	.010	4
March 1966	8-15	Bar Mat	3	.040	.013	2
			4	.033	.009	2
			6	.037	.008	2
		Mesh	1	.028	.008	2
			2	.048	.010	1
			5	.052	.009	2
March 1968	7-15	Bar Mat	5	.047	.015	3
			6	.043	Not measured	2
			7	.068	Not measured	2
			8	.045	.005	1
		Mesh	1	.048	Not measured	3
			2	.055	.010	3
			3	.047	Not measured	1
			4	.055	.008	1

The corrosive condition of the three sets of steel samples obtained from the sample cores is given in Table 5 along with adjacent crack spacing in the area cored, width of crack at surface, and crack width at the steel. Each steel sample was rated independently by three different persons. Since the rating scale is subject to interpretation the three ratings did not agree in all cases. Eleven samples were given the same index and for the remaining 15 samples two of the three observers were in agreement. For these samples the index number given in the table is that which was assigned by the two observers.

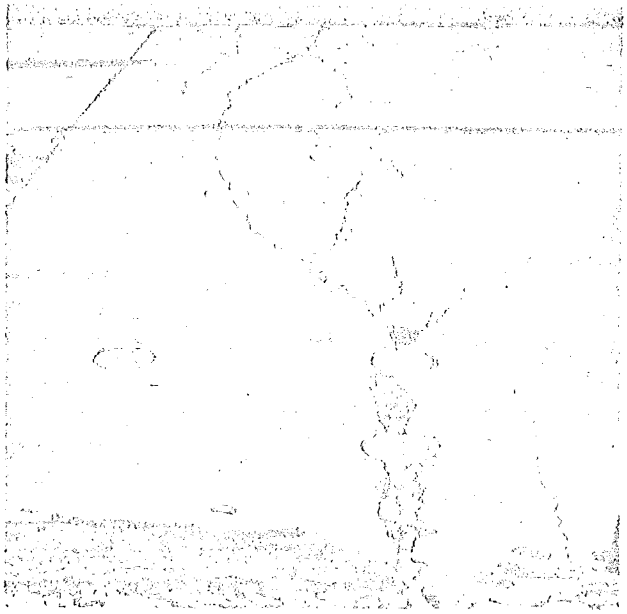
From the Table it is evident that corrosion of the steel does exist, but it is encouraging to note that apparently there has been little increase in the rate or degree of corrosion since 1963. Three of the samples removed after ten years showed no corrosion, two exhibited minute pitting and three had concentrated pitting at the crack location. These limited data presented are insufficient to make any predictions as to the length of time the reinforcement will continue to perform its function. However, on the basis of the 10 year condition as determined by the samples taken, it appears that it would be several years before reinforcement failure due to corrosion would take place.

FAILURE AND REPAIR

During the ten-year period since construction of the experimental pavement a total of 15 failures have occurred. Thirteen of these failures were in the continuously reinforced sections and two in the standard pavement section. The location and type of reinforcement at each repair area are given in Table 6.

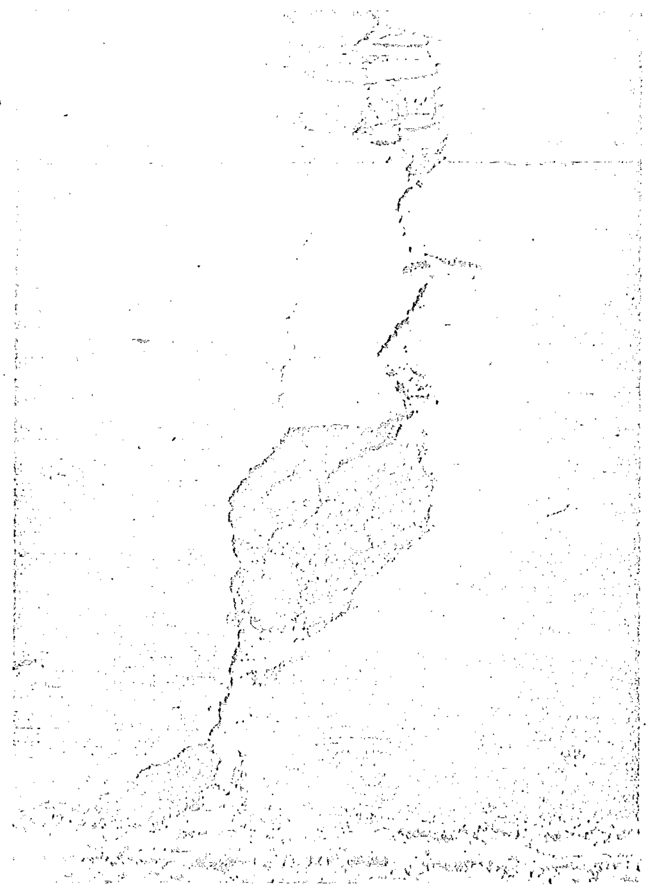
Repairs to re-establish the continuity of the pavement were made in Areas 1 through 5 in May 1962, in Areas 6 through 10 in September 1963, in Areas 11 through 13 in October 1965, and in Areas 14 and 15 in June 1968. Figure 57 illustrates a typical failure in each reinforcement type of the continuously reinforced sections.

Before a failure was repaired, cores were taken through the distressed pavement; and during repair, portions of the slab to be replaced were carefully removed in order to determine the cause of failure. As a result, the causes of each failure were determined and are discussed as follows:

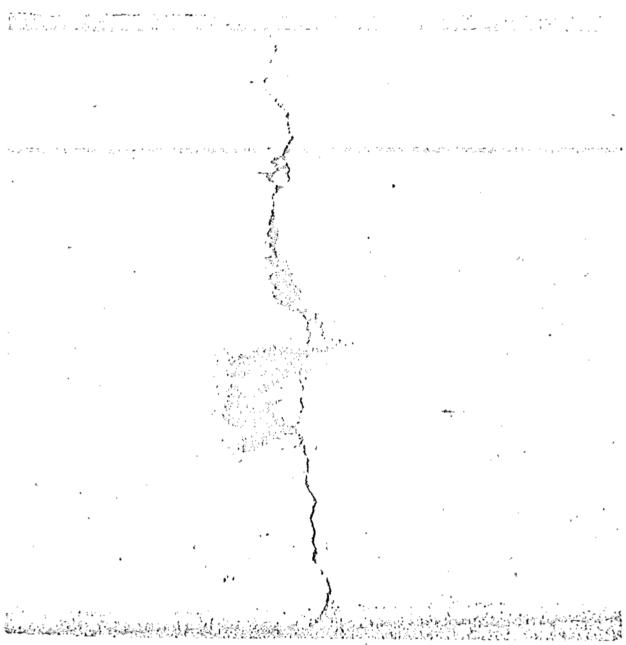


Bar mat reinforced section. Sta. 1017+03 eastbound.

Plain welded wire mesh reinforced section. Sta. 1045+40 westbound.



Plain welded wire mesh reinforced section. Sta. 1071+90 westbound.



Plain welded wire mesh reinforced section. Sta. 1070+25 westbound.

Figure 57. Typical failure conditions in continuously reinforced pavement sections.

TABLE 6

PAVEMENT FAILURES DURING THE TEN-YEAR OBSERVATION PERIOD

Area No.	Station	Roadway	Lane	Reinforcement Type
1	1071+90	Westbound	Traffic & Passing	Plain welded wire mesh
2	1045+70	Westbound	Traffic	Plain welded wire mesh
3	875+90	Eastbound	Traffic & Passing	Plain welded wire mesh
4	1017+03	Eastbound	Traffic & Passing	Deformed bar mat
5	1044+66	Eastbound	Traffic & Passing	Deformed bar mat
6*	1071+90	Westbound	Traffic	Plain welded wire mesh
7	1045+40	Westbound	Traffic & Passing	Plain welded wire mesh
8	976+87	Eastbound	Traffic & Passing	Standard mesh
9	990+82	Eastbound	Traffic & Passing	Standard mesh
10*	1044+66	Eastbound	Traffic	Deformed bar mat
11	1046+00	Westbound	Traffic	Plain welded wire mesh
12	1070+25	Westbound	Traffic & Passing	Plain welded wire mesh
13	936+72	Eastbound	Traffic & Passing	Plain welded wire mesh
14	1000+80	Westbound	Traffic & Passing	Plain welded wire mesh
15	1001+70	Westbound	Traffic & Passing	Plain welded wire mesh

* The failures at area 6 and 10 occurred in previously repaired areas.

Bar Mat Failures

Two failures (Areas 4 and 5) occurred in the first reinforcement lap following a transverse construction joint and were caused by poor consolidation of concrete in the area and a discontinuity in the steel reinforcement (the reinforcement mats had inadvertently been placed 2 in. apart in the vertical plane of the lap). As a result of the concrete being poorly consolidated and the reinforcement discontinuity, unusually wide cracks developed in these areas shortly after construction. Severe spalling along the cracks required periodic patching with bituminous cold patch material until the distressed pavement was replaced.

The third failure (Area 10) occurred in the replaced traffic lane pavement at Area 4 during the first night after concrete pouring. It was caused by insufficient bond development in the reinforcement steel splice and resulted in the formation of a wide single crack near the center of the reinforcement lap.

Plain Welded Wire Mesh Failures

Eight failures in the sections containing this type of reinforcement occurred at laps. These failures resulted from an insufficient lap length and

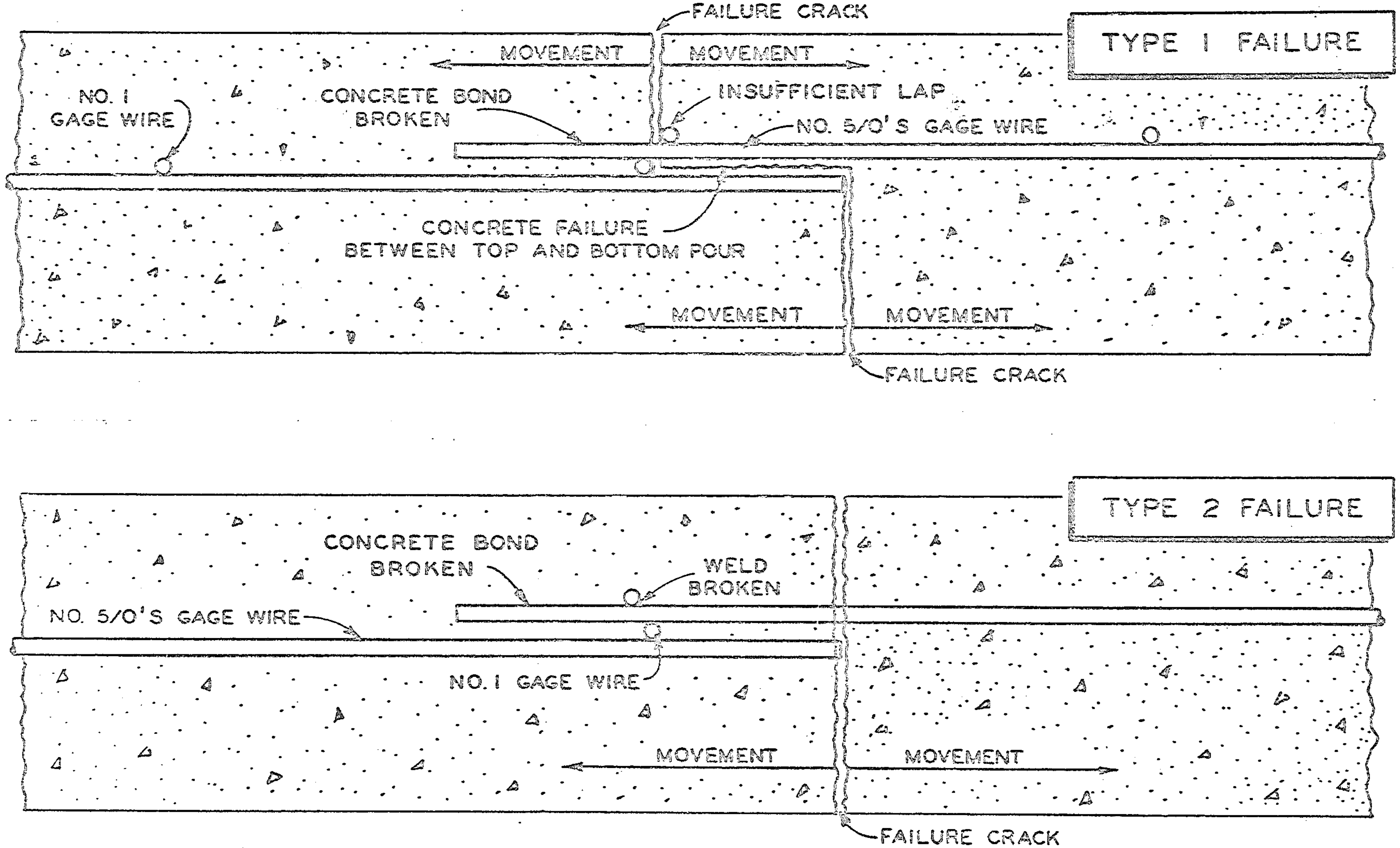


Figure 58. Two types of failure in wire mesh reinforced pavement.

are characterized by either one or both of the two types of failures shown in Figure 58, and described as follows:

Type 1 Failure is characterized by a vertical crack extending from the slab surface to the depth of the reinforcing steel; then by a horizontal crack or separation of the concrete between the steel mats to the end of the lower mat, where a vertical crack extends to the bottom of the slab. Because of the insufficient lap the bond resistance of the plain wire is broken and movement is possible as shown in Figure 58. Poor consolidation of the two concrete layers, as was observed in the areas where this type of failure existed, facilitates the formation of the horizontal crack at the steel level.

Type 2 Failure is distinguished by a vertical crack extending the full depth of the slab through a section at the end of the mat lap. The weld at the crosswire connection in the lap fails as well as the bond along the longitudinal steel extending through the vertical failure crack; the slab is then free to move (Fig. 58). Cores taken in the slab at sections through the failed crosswire connection indicate that the presence of moisture causes rusting of the reinforcement steel in the weld area, which undoubtedly increases the possibility of weld failure. Poor consolidation between the two concrete layers in the lap area apparently allows moisture to progress along this plane as water seeps down through the vertical crack. Improper welding or rusting of welds causes some of the cross-wire connections to fail before the maximum forces induced by shrinkage and temperature are developed. As a result, the stress normally taken by the failed welds is transferred to the remaining cross wire connections, causing them to be overstressed and a progressive failure occurs across the slab.

A brief description of each failure follows:

Area 1 - The failure crack originated at the outer edge of the traffic lane and it was found that a Type 2 failure had occurred in the outer 8 ft of this lane whereas the inner 4 ft had failed in the manner described for a Type 1 failure. In the passing lane the reinforcement had fractured at the failure crack. This steel fracture resulted from increased stress induced in the steel because the traffic lane was ineffective in resisting tensile forces. The steel was heavily rusted at the point of failure, indicating that corrosion had weakened the reinforcement prior to complete failure.

Area 2 - The failure was confined to the traffic lane and was categorized as a Type 2 failure. Extremely poor consolidation of the two concrete layers in the lap area was noted; thus, moisture entering through the vertical crack would have had easy access to the crosswire welds, which undoubtedly increased the possibility of weld failure by rusting of the steel.

Area 3 - A Type 1 failure was observed to have occurred in the traffic lane, where the failure crack originated. In the passing lane seven longitudinal wires had fractured completely. Fourteen wires near the original point of failure were severely corroded and the remaining 25 wires in the mat were rusted to a lesser extent.

Area 6 - The failure occurred in the May 1962 traffic lane repair patch (Area 1). A wide single crack developed in the center area of the patch during the night after concrete pouring, indicating that the bond strength of the reinforcement lap at this location did not have sufficient capacity to resist the ensuing tensile forces induced in the slab by shrinkage and decreasing temperature.

Area 7 - In the width comprising the first 17 longitudinal wires from the traffic lane edge a Type 1 failure had occurred. At the passing lane edge 21 wires had fractured at the failure crack. Although 12 wires in the remaining width of the roadway had fractured, it appeared that the failure in this area was a Type 2 failure.

Area 12 - Except for 14 wires fractured in the center of the passing lane, failure was characterized as a Type 1 failure. It appeared that the traffic lane and part of the passing lane failed in this manner first, causing overstressing of the steel in the center portion of the passing lane. As a result, these steel wires eventually fractured.

Area 14 - From the surface appearance of the distressed area and from coring through the crack, it was determined that a Type 1 failure had occurred in both lanes. During removal of the failed area it was noted that the concrete had not been sufficiently consolidated to obtain satisfactory bond between the top and bottom pours.

Area 15 - At this location it was found that a Type 2 failure had occurred in both lanes. In addition to poorly consolidated concrete in both lanes, the steel mats were placed 1 in. apart in the vertical plane of the lap.

At Station 1046+00 westbound roadway (Area 11) failure occurred in the traffic lane as a result of the presence of a large clay ball in the concrete. Spalls developed around the clay pocket and periodic repair, first with Embeco mortar and later with bituminous material, was required until repair with concrete was made in 1965. There was no break in the steel continuity at this location, but a few wires in the clay ball area had rusted through.

A blow-up occurred on June 27, 1965 at the construction joint located at Station 936+92 eastbound roadway (Area 13). Based on evidence gathered during repair, it was concluded that initial poor consolidation of the concrete on the morning side resulted in a plane-of-weakness along the steel level. Moisture entering through the joint, in addition to rusting the steel, seeped into the poorly consolidated concrete and caused further weakening and deterioration. The strength of the bottom concrete layer finally was reduced to the point that it offered little resistance to the induced compressive forces and failure occurred.

Standard Mesh Failures

The failure at Area 8 occurred at the third point of a 99-ft slab and was located almost directly over a 15 in. culvert. The reinforcement had fractured in both lanes. Because the contraction joints on either side of the failure were performing satisfactorily it appeared that loss of subgrade support rather than induced temperature and shrinkage stresses was responsible for the failure. At Area 9 the failure occurred at the approximate midpoint of the slab. The reinforcement had fractured in both lanes. The failure crack originated five months after construction of the pavement, and subsequent settlement of the subgrade required mud-jacking of one end in 1962. Stresses induced by volume change followed by loss of subgrade support, resulted in this failure.

The causes of the failures in the continuously reinforced pavement sections indicate that good workmanship is of the utmost importance in constructing this type of pavement. It is also apparent that the concrete vibration normally supplied by conventional paving equipment is not sufficient to obtain satisfactory consolidation of the concrete when the slab is poured in two lifts.

Repair Procedures

The basic requirement in replacing a section of continuously reinforced pavement is that continuity of the steel be maintained throughout the replaced area and the immediately adjacent original pavement. To insure this, the end limits of the repair areas were set at a minimum of 3 ft from a lap in the existing pavement so that 3 ft of reinforcement could be left intact through the end limit at each end of a repair in order that the replacement steel could be securely lapped with the existing reinforcement. Before establishing the limits of the area to be replaced, the laps in the reinforcement were located by taking cores through the failed pavement.

At the end limits of the concrete area to be replaced, a sawcut 1-1/2 in. deep was made to provide a neat straight joint across the pavement. The reinforcement was then exposed and cut 3 ft inside the sawed limits and the freed center portion of the distressed slab was broken and removed by mechanical equipment. In the areas where the existing reinforcement was left intact the concrete was removed by use of air hammers and hand tools. The longitudinal replacement steel consisted of No. 5 deformed bars in the bar mat reinforcement section and No. 4 deformed bars in the wire mesh reinforced sections. The bars were placed adjacent to and lapped 3 ft with each existing bar or wire at each end of a repair patch. All transverse steel was No. 3 deformed bars and was spaced at approximately 2-ft centers. A high-early-strength transit-mixed concrete was used in the repair patches. The concrete finishing operations were done by hand, and the patches were allowed to cure for three days before being opened to traffic.

When the continuity of a continuously reinforced roadway is lost across its entire width, the free ends at the failure are subjected to relatively large daily longitudinal movements due to temperature and moisture fluctuations. As a result it is difficult to develop sufficient bond strength in the lap joining the existing reinforcement and the replacement steel. In cases where only one lane fails, repair experience indicates that the remaining lane is capable of resisting the normally expected daily temperature-induced tensile forces.

The May 1962 repair procedures required the replacement steel to span the full length of the patch and that each No. 5 replacement bar be welded to an existing bar with 4 2-in. long flare groove welds equally spaced along the 3 ft lap. The No. 4 bars used in the welded mesh repairs were to be similarly welded but only three welds per bar were required. By specifying welding of the laps the reliance on early bond strength development to re-establish the continuity was eliminated. However, at two repairs where both lanes had failed, the bars welded in place bowed up vertically when unexpected hot weather occurred, and in order to return the steel to the correct elevation before concrete pouring, it was cut in the center of the patch and spliced. As previously mentioned, a large single crack developed at the splice location in these two repairs, indicating bond failure of the splice. Welding of the steel lap at the remaining locations was omitted since only one lane had failed or was partially effective in resisting induced forces. Instead the steel was only lapped for the 3-ft length at each end of a patch. The bond strength of these laps coupled with the resistance to movement of the other lane was sufficient to re-establish the continuity of the pavement.

As a result of the experience gained from the 1962 repairs, the following procedure to be followed in repairing failures in the continuously reinforced sections was recommended:

1. Where both lanes fail the replacement steel in the lane to be repaired is first welded to the existing steel with two 1-in. flare groove welds per bar. The welding operation at one end of the repair patch is postponed until just prior to concrete pouring to minimize the effect of temperature induced stresses. In the lane repaired last and at repairs where only one lane has failed the replacement steel is lapped 3 ft with the existing reinforcement.

2. Concrete pouring should take place during the late evening hours from 8 p.m. to 12 p.m. to minimize the temperature drop in the cooling cycle, and to give the concrete a six to eight hour cure before the concrete goes into compression in the warming cycle of the following day. By exercising these precautions the pavement continuity at the areas repaired in 1963, 1965, and 1968 were established without difficulty.

The two failures in the standard pavement were repaired by removing a 2-ft section and installing a standard contraction joint at each location. These areas have since continued to function in a satisfactory manner.

RELATIVE COST

A comparison of the initial construction costs for continuously reinforced and jointed concrete pavement for eight yearly periods when continuously reinforced pavement has been built in Michigan is given in Table 7.

As noted, the 1958 and 1961 costs are based on 9-in. uniform standard pavement with a 99-ft joint spacing and 8-in. uniform continuously reinforced pavement with 0.6 percent steel. The 1964 through 1970 costs are based on 10-in. uniform standard pavement with a 72-ft joint spacing and 9-in. uniform continuously reinforced pavement with 0.7 percent steel.

These yearly cost comparisons are based on the jointed pavement that was the standard at that time, and which was built in the same construction zone where the continuously reinforced pavement projects were located. The 1958 and 1961 construction seasons represent rural two-lane Interstate construction, whereas the remaining years represent urban multiple-lane Interstate construction located all in the Detroit area.

All costs include pavement reinforcement, joint construction and material, anchor lugs, and wide-flange or multiple expansion joint relief sections.

Based on these cost comparisons, the initial construction cost of continuously reinforced concrete pavement has averaged about 25 percent greater than standard jointed concrete pavement. It is also of interest to note that the cost per lane mile of construction for either jointed or continuous pavement has risen about 30 percent over the six-year period from 1965 through 1970.

Unfortunately, a realistic economic comparison between these two types of concrete pavement cannot be made since the total maintenance cost per lane mile per year for maintenance of pavement is not available and there is no quantitative standard to which concrete pavements are continuously maintained. It is of interest to note, however, that on two maintenance contracts let in 1970 for the removal and replacement of deteriorated joints in 12 and 14 year old pavement, the cost per square yard ranged from $25 to $35, or some three to four times the initial construction cost for the same year.

TABLE 7
INITIAL PAVEMENT COST

Year[1]	Standard Pavement		CRC Pavement	
	Lane Mile	Sq Yd	Lane Mile	Sq Yd
1958	$33,877	$4.82	$41,536	$ 5.90
1961	33,440	4.74	41,817	5.94
1964	45,196	6.42	52,025	7.38
1965	48,012	6.82	65,050	9.23
1967	55,970	7.95	71,808	10.20
1968	63,660	9.05	80,115	11.36
1969	64,910	9.21	79,341	11.33
1970	64,641	9.18	82,227	11.68

(1) 1958 and 1961 costs are based on 9-in. uniform standard pavement with 99-ft joint spacing and 8-in. uniform CRC pavement with 0.6 percent steel.
1964-1970 costs are based on 10-in. uniform standard pavement with 72-ft joint spacing and 9-in. uniform CRC pavement with 0.7 percent steel.

REFERENCE

1. An Experimental Continuously Reinforced Concrete Pavement in Michigan, G. R. Cudney, Highway Research Board Bulletin 274, 1960.