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CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

REPORT NO. 2

RESEARCH IN CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

LITERATURE SURVEY

by

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A. INTRODUCTION

The history of concrete pavements in the United States has followed the trend of local experience, and the increasing traffic needs that have developed with the increases of surface motor transportation.

The first concrete pavement, in the United States was constructed in Belletontaine, Ohio, in 1891[1.1]. This pavement consisted of plain concrete blocks, five to six feet square and six inches thick. It is logical to deduce from the type of construction prevelant in urban areas in that period, that the prototype of this first concrete pavement was pavement block construction. Thus the comparatively small slabs.

The first major use of concrete for rural roads was in Wayne County Michigan, in which four miles of highway were constructed in 1909, an additional 20 miles a year later, and then another 40 miles in 1911[1.8].

The design of pavements by 1915 had taken a radical change from the original concrete blocks. By this time the pavments of plain concrete had evolved to continuous slabs of full road width, the only joints being the daily construction joints. In general, these pavements were constructed with an 8 inch center thickness similar to the macadam roads of that period[1.3]. The pavement was constructed directly over the subgrade, which was constructed without thought of compaction. The loads that this type of pavement were subjected to were light, and subsequently the behavior was quite satisfactory. The crack pattern that mostly developed consisted of random longitudinal cracks near the center of the pavement, and transverse cracks spaced from 6 to 30 feet apart. As traffic loads increased, certain maintenance items became apparent, and of considerable concern. In the areas of poor subgrade soil conditions, the major maintenance problems were the result of pavement breaks at the edges, and at the juncture of the longitudinal and transverse cracks. In addition to the corner and edge breaks, there existed a great tendency for the joints to fault, disturbing the riding qualities and accentuating the tendency for breaking and spauling.

To alleviate the above problems of the earlier continuous plain slabs, the design of pavements was modified to a uniform thickness, usually 8 inches. This was of little affect, and the design was further modified to thicken the edges of the pavement. In addition longitudinal cracking was controlled by installing a longitudinal joint, with tie bars across the joint to hold the two slabs together. This principle which was generally accepted about 1922, is still in force in all types of concrete pavements. Today four thickness designs are used[1.3]. These are:

1. Pavements of uniform 8, 9, or 10 inch thickness
2. Pavements of 5, 6, 7 or 8 inch center thickness, and 7, 8, 9 or 10 inch edge thickness, with only the edge thickened.
3. Pavements of thickened edges, going from a 5, 6, or 7 inch center thickness to a 7, 9 or 10 inch edge thickness on a parabolic curve.
4. Pavements of thickened edge and center joint.

A further attempt to reduce corner breaks resulted in the inclusion of reinforcing bars at the edges of the pavement.

The increasing experience with concrete pavement design was matched by developments of the theory of concrete pavements. The first published attempt at a rational design procedure was developed by Older in 1920 and 1921[3.41], as a result of investigations conducted at the

Bates Test Road in Illinois, in 1920. The results of this test road, initiated the widespread use of thickened edge pavements in the United States. The rational design by Older was based on assumptions of a non-supporting subgrade and one-dimensional flexure in the pavement under a static load applied at a point near the corner of a pavement. These relationships were shortly supplanted by the work of Westergard(3.68),(3.71) who envisaged the pavement as an elastic slab resting on a bed of springs. The analysis of Westergard indicated that stresses in the pavement due to wheel loads were most critical when these loads were at the corner of a pavement; less critical when the load was applied at the edge; and still less critical when the load was applied at the center. This analysis, which came after the experience, justified the previous changes in design and enabled engineers to rationally calculate a pavement design. Had this analytical effort pre-dated the intuitive experimentation, a great deal of effort and financial expenditure may have been saved. Since 1925, Westergards work has been modified by experiment(3.63), (3.65), (3.67), (3.33), and by further theoretical studies by Westergard(3.79) Picketts(3.47), Burmister(3.13) and others. At the present time, there is a very large school of highway engineers who believe that the thickness design of rigid pavements has reached the point of economic stability, and that additional research is unnecessary. The primary motivation behind this thinking is the standardization of design that has been achieved, and the lack of knowledge of many highway engineers concerning the theoretical concepts behind a given thickness design. It is as yet truly unknown whether success at a particular design is not simply do to over designed pavements which although uneconomical maintain a long period of maintenance free life.

With the growing interest in soil mechanics in the late twenties and early thirties, it became apparent to highway people that thought had to be given to improvement of the subgrade. With the inclusion of joints in pavement design water was trapped under the joints and the dynamic action of traffic, literally pumped fluid mud from under the pavement. The loss of support caused by pumping created unsightly cracks and in many cases, outright failures of the pavement. At about the same time concern was developing over pavement pumping, frost heaving was being noticed as a major problem[(2.1), (2.7)]. The correction to both these problems lay in the same remedy. A six or eight inch compacted granular base course was installed between the pavement and the subgrade[(2.4), (2.5), (2.8)]. This base course consisted of free draining material which carried the water away and eliminated pumping. In addition the base course insulated the subgrade reduced the capillary potential and relieved the frost heaving problem. It wasn't until much later, after the theoretical work of Burmister and Picketts, that it was realized that the base course exerted an added influence on the strength of the pavement. The base course material being of higher quality than the subgrade acted as a strengthening element to distribute the pavement loads over the subgrade. Thus the system resulted in better pavement performance, with respect to subgrade support than had been achieved prior to that time. Although subgrade support was improved by external applications, many failures still occurred which could be attributed directly to the subgrade. The problem at hand was one of improving a given subgrade material such that a stable condition of maximum strength could be achieved. The first step in this direction was the establishment of moisture density relations for clay-soils[(2.50)]. This was first established in the laboratory, but its potential was so wide that it shortly evolved into field procedures[2.48], [2.49]. A given soil can reach a state of strength permanence if it is compacted artificially to a condition of optimum density, while containing an optimum amount of water. Thus, by field controls of moisture and compacting effort, subgrade soils can achieve an optimum, stable state. Methods of compaction control are used today, in all but a very few states, as standard procedure for all highway work.

By the time that the original concrete pavements built in the early part of this century were nine to twelve years old "blow-up" occurred at the transverse cracks during hot weather. It was believed that the buckling of the pavement was caused by expansion of the concrete. As a result the construction practice was changed such that expansion joints were installed at the daily construction joints. Subsequently the spacing of expansion joints was decreased to spacings of 100, 200 and 400 feet. In addition, steel dowel, load transfer devices were installed in pavement expansion joints where the spacing was 200 feet or less. Although this remedy helped reduce the buckling problem, it did not reducd the problem of transverse cracks. In fact, the cracks that now appeared were wider than in the older types of pavement. These cracks were attributed to tension occurring during the subgrade restrained contraction. As in the case of warping, the remedy was one of controlling the cracking. Thus contraction joints were installed between the expansion joints.

Although jointing pavements helped in the control of cracking, it did not eliminate transverse cracks and further it did not control the size of the random transverse cracks. The use of light steel reinforcing in the pavement was incorporated into the design as a means of holding the random transverse cracks to non-objectionable widths. Although the reinforcement improved maintenance problems, it did not eliminate them, as joint maintenance was still a major factor in all highway budgets.

Current conventional slab design calls for a six inch base course to be placed over a highly controlled compacted subgrade. The base course is constructed of special free-draining granular soil and is vibrated or rolled in place. In some instances the base course is water bound. The pavement slabs are 8 to 10 inches thick; and reinforced with steel percentages in the vicinity of 0.12 per cent of the cross-sectional area of the slab. The joints are currently spaced 30 to 100 feet apart and contain 1 1/14 inch dowels spaces 12 inches on center. These pavements are very strong, and for the first few years of service behave very well indeed. These pavements, however, are over designed at the interior and under designed at the joints. The result is that all the joints require almost constant maintenance and in many instances repair.

As a result of this apparent inadequacy in current design techniques, highway engineers have been looking for other methods of construction and design of pavements.

In 1921, a portion of the Columbia Pike, near Washington, D. C. was constructed with 350 foot reinforced slabs. The behavior of this section of highway was so outstandedly superior that highway engineers, in the search for better performing pavements, conceived of the idea of a continuously reinforced pavement in which no joints were installed. The first experiment of continuously reinforced pavements occurred in 1938, when a five mile experimental highway was constructed in Indiana. This was followed in 1947 by two additional experimental projects in

Illinois and New Jersey. In 1949, California conducted a similar experiment. As a result of these experimental projects the methods of design was adopted by the Fort Worth (Texas) Freeway Authority and in 1951 constructed a road in this manner. In 1956, Pennsylvania commenced an extensive investigation into the use of continuously reinforced pavements. This investigation consisted of the construction of two experimental highways, and a program of field service testing to evaluate the performance of these types of pavements.

This report is designed to summarize the published information on continuously reinforced concrete pavements, and to evaluate some of the findings, in terms of the experimental projects being conducted in Pennsylvania. In addition, an extensive selected bibliography is included which covers the important published work on rigid pavements in general.

B. CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

Continuously reinforced concrete pavements are designed to eliminate the poor joint performance that occur in conventional pavements. The basic principle underlying the continuous pavement is that a slab will invariably crack transversely, but that if these cracks can be held to small openings they will do no damage. Instead of putting joints in the pavement to force and control the cracking, the continuous pavement is permitted to crack randomly. The width at the crack is held small by the use of a high percentage of reinforcing steel which hold the cracks together.

In this section, all the current research projects and published theories of continuously reinforced pavements will be summarized. In all cases the opinions and conclusions are those of the respective authors of the cited reports and are not necessarily in agreement with the conclusions and theories of the author of this literature survey.

1. THEORY OF CONTINUOUS REINFORCED PAVEMENTS

Theories of behavior of continuously reinforced concrete pavements have to date, concentrated on the determination of the amount of reinforcement necessary to maintain a closed crack. The basic mechanism studied in this question has been the mechanism of temperature variations.

The first attempt at explaining the thermal behavior of long reinforced highway slabs was made by Woolley[6.16]. In this paper the behavior of a continuously reinforced slab was likened to a steel bar, whose ends were completely restrained from moving. Woolley further analyzed the behavior of a reinforced slab as follows.

As the temperature drops in a continuously reinforced slab, the major stresses in the slab are carried by the concrete, until such time as the slab cracks and then the steel carries the total stress at the crack. The steel stress is transfered to the concrete away from the crack, via the bond properties of the bars. The crack will open slightly on rupture which will tend to relieve the tension in the steel. As the temperature continues to drop the steel stress will increase in tension and elongate, thereby opening the crack. Amounts of steel inadequate to control the crack opening will permit excessive elongation and eventually rupture. As the crack opens, however, there will be higher stresses produced in the surrounding concrete and another crack will form. The formation of the additional crack will again relieve the stress in the steel, and continue the above process until the entire pavement, except near the ends will contain a system of narrow cracks, closely spaced.

In terms of the stresses produced, this theory postulated that, since the steel is stronger than the concrete the steel will always cause another crack, in the concrete, before it is overstressed.

On the above basis, certain recommendations are made with regard to the correct percentage of steel in a continuously reinforced concrete pavement. These design criteria are based on the transformed area concept of reinforced concrete, and are based on the desired design stresses in the steel. Assuming steel with a yeild point of 50,000 psi; concrete with a tensile strength of 350 psi, and allowing for a 100°F drop in temperature, the analysis requires 1.0 per cent of steel.

Despite this, it is recommended that continuously reinforced pavement be reinforced with 0.5 per cent of steel, based on field observations.

The mechanism of shrinkage is discussed by Woolley. The shrinkage is considered to be a tension process in the concrete in which the steel acts to distribute the stresses throughout the length of the pavement. Thus the steel in the pavement causes many small cracks in the concrete. This is contrasted to the few wide cracks in plain concrete.

Based on the general mechanics of behavior, Woolley in his theory, attempts to apply the shrinkage effects to crack the pavement at an early age and thus establish a highly cracked pavement before temperature changes can take place. He reasons that if the concrete is poorly cured, the shrinkage will take place before the concrete has had time to build up its strength, and the shrinkage stresses in the concrete will cause closely spaced, tightly closed cracks in the pavement, although there is a minimum of steel. In addition, after these cracks are formed they will act as miniature contraction joints during temperature flucuations. This phenomena of shrinkage cracks is used as an explanation of why the recommended reinforcement is one-half of that required theoretically.

The theory as advanced by Woolley forms in many ways the basis for present thinking on this subject. The one big point of dispute in the thinking is the attempt to combine the behavior of both a cracked and an uncracked slab. In doing this, he fails to consider the continuity conditions that must prevail at the crack, and therefore inevitably comes up with the contradiction that the theoretically required amount of reinforcement is twice that of the amount which he claims to have been successful in field experiments.

A purely analytical model for the behavior of a cracked pavement has been proposed by Friberg[6.7]. This model is shown in Figure 1.

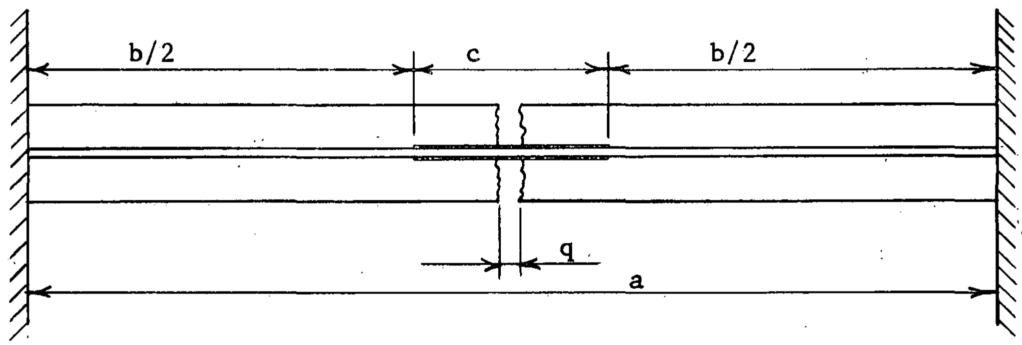

FIGURE 1

FULLY RESTRAINED CRACKED PAVEMENT

The assumptions involved in the analysis are:

1) The concrete and steel act elastically.

2) The steel in the immediate vicinity of the crack is unbonded.

3) The entire section is fully restrained against movement.

4) The length of the section is sufficient for bond to be fully effective.

5) The steel-bond slip conditions are fully reversible.

On the basis of the above assumptions the problem is analyzed following the conditions of equilibrium and continuity as follows:

1) Forces throughout the system must be in equilibrium.

$$\sigma_c + p\,\sigma_b = p\sigma \qquad (1)$$

where

σ_c = Stress in the concrete

σ_b = Stress in the steel away from the crack

σ = Stress in the steel over the unbonded length (c)

p = Net reinforcing percentage

2) The length of the concrete and steel in the fully bonded section (b) must remain equal, regardless of the temperature change and the common change in length.

$$\frac{\sigma_c}{E_c} b - e_c\, t\, b = \frac{\sigma_b}{E_s} b - e_s\, t\, b \qquad (2)$$

where

E_c = Modulus of concrete

E_s = Modulus of steel

e_c = Coefficient of thermal expansion of concrete

e_s = Coefficient of thermal expansion of steel

t = Temperature change

b = Fully bonded length

3) No over-all length change will take place in the section with changes in temperature.

$$\frac{\sigma_b}{E_s} b + \frac{\sigma}{E_s} c - a\, t\, e_s = 0 \qquad (3)$$

where

c = Length of unbonded portion

a = Total length of section

The simultaneous solution of these three equations for the stresses results as follows:

$$\sigma_c = \frac{ctp\, E_s\, e_c}{npa + c} \qquad (4)$$

$$\sigma_b = E_s\ e_s\ t - \frac{c\ t\ E_s\ e_c}{n\ p\ a + c} \tag{5}$$

$$\sigma = E_s\ e_s\ t + \frac{b\ t\ E_s\ e_c}{n\ p\ a + c} \tag{6}$$

where

$$n = \frac{E_s}{E_c} \tag{7}$$

Under the same conditions the change in the crack width is:

$$q = a\ e_c\ t - \frac{\sigma_c}{E_c}\ b \tag{8}$$

where

q = crack width

In particular if the thermal coefficients for steel and concrete are equal, then:

$$q = \frac{\sigma}{E_s}\ c \qquad (e_c = e_s) \tag{9}$$

Based on this theory, it is possible to make some further simplifying assumptions on the safe side and develop a design criteria for the percentage of steel. These further assumptions are:

1) The steel remains fully bonded except at the crack; i.e. $c = 0$.

2) The thermal coefficients for steel and concrete are equal;

$$e = e_c = e_s$$

Thus the percentage of steel can be determined from equation (6).

$$p = \frac{E_c}{\frac{\sigma}{et} - E_s}$$

For a set of conditions considered to be usual such as:

$E_c = 3,000,000$ psi

$E_s = 30,000,000$ psi

$e = 0.0000065$ in/in/^{o}F

$t = 100^{o}$F

and limiting σ to 50,000 psi, p comes out to be 0.64%.

The above theory thus is closer than others to the actual conditions. It lacks, however, two basic considerations. In the first place it does not consider the possibility or the consequences, either favorable or unfavorable, of the steel exceeding the yield point. The second factor that this theory fails to consider is the fact that strains in the steel are a more representative clue to the phenomena than the stresses. The width of the crack is the single major maintenance criteria. This is a strain phenomena, and should only be considered in this light.

The other assumptions that were used are either compatible with field observations or are necessary to maintain mathematically manageable expressions.

On the basis of analysis and observations of the field experimental projects currently in service, the Rail Steel Bar Association has recommended as of February 1957, the following design criteria for continuously reinforced concrete pavements:

1) For highways carrying moderate amounts of traffic:

   a. 7 inch thick pavement

   b. 0.6% longitudinal steel

   c. #6 longitudinal bars in the center depth of the slab spaced 10 inches on center.

   d. #4 transverse bars spaced 36 inches on centers

   e. 3-inch compacted granular base course

2) For highways carrying heavy traffic:

   a. 8 inch thick pavement

   b. 0.6% longitudinal steel

   c. #6 longitudinal bars at the center depth of the pavement slab, spaced 9 inches on center

   d. #4 transverse bars spaced 36 inches on center

   e. 3 inch compacted granular base course

A qualative theory of behavior has been presented by Jacobs as follows[6.8]. There are two areas of pavement to consider, these being:

1) The slabs of moderate length where the slab ends are not fixed and are free to expand or contract under the influence of temperature differentials.

2) The central portions of continuously reinforced concrete pavements which are completely restrained from moving.

In addition to the above areas of concern there are two temperature conditions, namely

1) The temperature is above the equilibrium temperature at which the concrete hardened.

2) The temperature is below the equilibrium temperature at which the concrete hardened.

Thus there are four conditions of stress to be considered.

1) With expansion in the end zones and relief from slab movement over the subgrade, there is no possibility of critical compression stresses and the magnitude of the expansion is limited by the uniform slab temperature above that at which the concrete hardened.

2) In the end zones as the temperature drops from equilibrium tension occurs in the slab, and is cumulative to the lowest uniform slab temperature that occurs except as relieved by movement over the subgrade.

3) A temperature increase in the fully restrained areas will result in full compressive stresses limited only by the temperature differential with respect to the equilibrium temperature.

4) A temperature decrease in the fully restrained areas will result in full tension stresses limited only by the temperature differential with respect to the equilibrium temperature.

The stresses in the end zones are modified by the ability of the slab to move in the end zones, to the extent that maximum stress can occur only at approaches to the completely restrained portion and cannot exceed the stresses in that portion.

The varied crack spacing is a natural result of the above theory. The crack spacing is widest near the end decreasing to a uniform minimum at a moderate distance from the end where restraint has built up to the point where there is no motion on the subgrade.

Since expansion and contraction are relieved by motion on the subgrade, the crack width will be lessened with motion. Thus the crack width will increase from the end toward the point of restraint where widths will be maximum.

In the spring of 1957, the Highway Research Board of the National Research Council organized a special subcommittee on continuously reinforced concrete pavements. The purpose of this committee was to recommend criteria for research in this area. As of May 1957, the informal recommendations were to investigate the following variables:

1) Base course thickness with a minimum of 3 inches.
2) Variations in the percentage of steel from 0.5 to 0.7 per cent.
3) Variations in the spacing of steel from 4 to 12 inches.
4) Variations in the thickness of the pavement from 7 to 9 inches.

2. RESEARCH IN INDIANA

In 1938 a cooperative research project was commenced on the study of concrete pavements. This project under the joint aegis of the Indiana State Highway Department and the U. S. Bureau of Public Roads, embraced a section of highway near Stilesville, Indiana and was for the specific purpose of studying the effects of varying amounts of continuous longitudinal steel reinforcement in pavement slabs of various lengths. The experimental sections ranged in length from 20 to 1,310 feet, were constructed with rail-steel bars, billet-steel bars and welded wire fabric of varying sizes, such as to provide percentages of longitudinal steel ranging from 0.07 to 1.82. A tabulation of the length, and reinforcement is presented in Table 1.

TABLE 1

LENGTH AND LONGITUDINAL REINFORCEMENT DETAILS
INDIANA PROJECT

Length of Section (ft)	Reinforcement Diameter (in)	Reinforcement Spacing (in)	Percentage	Yield Point psi	Ultimate Strength psi
a) Deformed Rail-Steel Bars					
600					
840	1	6	1.82	63,000	113,200
1,080					
1,320					
340					
470	3/4	6	1.02	64,400	113,300
610					
740					
150					
210	1/2	6	0.45	68,800	115,300
270					
330					
80					
120	3/8	6	0.26	66,700	93,600
150					
180					
40	1/4	6	0.11	60,300	84,600
50					
60					
80					
b) Intermediate Grade Deformed Billet-Steel Bars					
360					
600	1	6	1.82	46,900	78,000
840					
1,080					
200					
340					
470	3/4	6	1.02	49,100	78,500
610					
90					
150	1/2	6	0.45	51,400	78,600
210					
270					
50					
80	3/8	6	0.26	55,500	81,900
120					
150					

20 40 50 60	1/4	6	0.11	56,900	77,300

c) Cold-Drawn Welded Wire Fabric

140 190 250 310	0.3938 (No. 0000)	4	0.42	--	81,800
90 130 170 200	0.3938 (No. 0000)	6	0.28	--	80,300
80 110 140 170	0.3625 (No. 000)	6	0.24	--	89,100
60 80 100 120	0.3065 (No. 0)	6	0.17	--	83,700
30 50 60 80	0.2437 (No. 3)	6	0.11	--	81,000
20 30 40 50	0.1920 (No. 6)	6	0.07	--	88,700

This highway was constructed in September and October 1938. The pavement was a 9-7-9 thickened edge slab 20 feet wide. Three types of expansion joints were used in between sections. The Type I joint between slabs of intermediate length was similar to bridge expansion joints that were in use at that time. This joint consisted of angles anchored on opposite sides of the joint and a cover plate rigidly attached to one angle and permitted to slide in a key-way on the opposite side of the joint. The joint was designed to permit 1-1/2 inches of movement in either direction. Type II joints consisted of two Type I joints spaced 10 feet apart and were used where longer sections joined. Type III joints were conventional dowel units made with 3/4 inch plain round bars spaced 12 inches on center. The bar reinforcement was placed at the mid-depth of the slab and was mounted prior to pouring concrete. The wire fabric was placed by the strike-off method at 2-1/2 inches from the top of the slab.

The slab was poured directly on the subgrade soil. The subgrade soil has a plasticity index of from 14 to 26.

The research for this project was in terms of performance for the following purposes.

1) To determine the effect on performances of transverse cracking in terms of surface deterioration and concrete durability.

2) To determine the manner in which the various types and quantities of longitudinal reinforcement performed their structural function of holding cracks tightly and permanently closed.

3) To determine the susceptibility of the sections to pumping and the influence of pumping at the joints, cracks and edges

of the pavement.

4) To determine the maintenance requirements of each section.

5) To determine the effects of traffic on pavement behavior.

6) To determine the surface smoothness of the sections.

The performance of the pavement was based on period surveys, made shortly after construction and at the end of 1, 3, 5, 8, 10 and 15-1/2 years of service.

The observations included a detailed crack survey, level measurements, and measurements of horizontal movement of the end, quarter points and center of each section.

At the end of one year of service certain observations became available(6.1). In the first place, a large number of fine transverse cracks had appeared in the central portions of the long heavily reinforced sections, and the crack spacing was frequently less than 3 feet. In general, it was then observed that the crack spacing seemed to be directly related to the length of the section, and did not bear a relationship with the amount of reinforcing.

After two years of service the reported performance was as follows(6.2).

1) The cracking in the 1.82 and 1.02% reinforcement were tighter than in the sections with 0.45% reinforcements.

2) The average distance between cracks in the 600 foot sections decreased from 16 feet in March 1939 to 10 feet in November 1940.

3) The average distance between cracks in the 1070 foot sections decreased from 13 feet in March 1939 to 6 feet in November 1940.

4) The crack width was inversely proportional to the amount of steel.

5) All cracks remained tightly closed, and had little or no structural significance.

6) In the longer sections, 40% of the cracking after two years had developed during the curing period.

7) No longitudinal cracks had developed.

8) The daily length changes in the slabs longer than 750 to 850 feet were of the same magnitude. On this basis it was deduced that the central portions of slabs longer than 750 to 850 feet were completely restrained with respect to short-term temperature flucuations.

9) The long term length flucuations of slabs longer than 335 feet were reversible. From this fact, it was deduced that the initial crack widths were very fine and the reinforcements prevented progressive opening of the cracks.

10) During the first year of service the horizontal movements of the 1310 feet section showed that the quarter point movement was 10% of the movement of a free slab, while the end movement was 40% of that for a free slab. During the second year of observation the quarter point moved 33% of a free slab while the ends moved 54% of a free slab. The actual movements were:

1939: quarter point - 0.15 inches
ends - 0.80 inches

1940: quarter point - 0.42 inches
ends - 1.40 inches

11) The seasonal elevation changes were non-uniform, and were less than one inch. There was no indication that these changes affected the structural integrity of the slab.

12) After 18 months of service all surfaces were still very smooth.

At the end of five years of age, another survey was made of the highway and certain significant facts were reported(6.3).

1) The crack width measured at the pavement edges were found to be as follows:

0.07% steel - 0.009 inches
0.45% steel - 0.003 inches
0.82% steel - 0.002 inches

2) The average crack spacing in the 800 foot sections which was 8.5 feet at the end of one year was reduced to 6.5 feet at the end of 5 years. In the 1310 foot sections the spacing was reduced from 7 feet to 5 feet in the same period.

3) The maximum crack spacing occurred at the ends of the slab while the minimum spacing occurred in the central portion of the slab.

4) In the 1070 foot and 1310 foot lengths, it was found that the average crack spacing for the central 400 feet was the same. Thus it was deduced that the crack spacing in the central portions of a long slab was independent of the slab length for lengths above a certain minimum.

5) The effect of traffic on the formation of cracks was observed by noting the differences in crack formation between the main traffic lane and the passing lane. In the first year of service the main lane contained 2% more cracks than the passing lane. By the end of the third year this percentage increased to 4% and by the fifth year there were 6% more.

6) The maximum tensile stresses developed in the late Summer and Fall.

7) There were small changes in the pavement elevation which had no apparent effect on length changes or crack pattern.

A very complete survey and report was made at the end of 10 years of age[6.4]. These findings and the conclusions drawn are as follows:

1) The crack pattern for the 1310 foot section indicated the following average spacings:

Entire length - 4.2 feet

Central 700 feet - 3.3 feet

End 610 feet - 7.3 feet

2) The surface width of the cracks indicated that some rounding of the crack edges had occurred. The spawling of the surface cracks was measured to be 1/8 inch deep. Table 2 presents the average surface width of fifteen to twenty cracks in various portions.

TABLE 2

SURFACE WIDTH OF CRACKS IN CENTRAL PORTION OF SLAB
INDIANA PROJECT - 10 YEARS

% of Reinforcement	Width in Heavy Traffic Lane (in)			Width in Passing Traffic Lane (in)		
	Min.	Aver.	Max.	Min.	Aver.	Max.
0.26	0.09	0.117	0.15	0.02	0.038	0.07
0.45	0.07	0.104	0.18	0.02	0.038	0.06
1.02	0.03	0.078	0.15	0.02	0.032	0.05
1.82	0.02	0.053	0.11	0.01	0.020	0.03

The surface widths of the cracks in the central portion of the 1310 foot section averaged about twice the widths near the ends. The average crack width decreased directly with an increase in steel percentage..

The true width of the cracks in the central portions of the slabs was measured at the edge mid-depth. The average of five readings is presented in Table 3.

TABLE 3

CRACK WIDTH AT CENTRAL PORTION OF SLAB
INDIANA PROJECT - 10 YEARS

% of Reinforcements	Width in Heavy Traffic Lane (in)			Width in Passing Traffic Lane (in)		
	Min.	Aver.	Max.	Min.	Aver.	Max.
0.24	0.005	0.013	0.018	0.006	0.010	0.013
0.45	0.007	0.011	0.018	0.007	0.009	0.010
1.82	0.003	0.004	0.007	0.001	0.002	0.003

3) The slight effect of traffic was evidenced by the fact that during the second 5 years of service, 55% of the new cracks developed in the heavy traffic lane.

4) In the long sections, it was observed that 65% of the 10 year cracking had occurred during the first year.

5) The 10 year crack spacing for slabs longer than 1700 feet decreased uniformly with the distance from the ends of the section. The spacing in the central portions was uniform and averaged 2 to 2.5 feet. As a result it was concluded that the central portions maintained complete movement restraint.

6) The daily changes in length of all sections was independent of long term time variations.

7) In the 1310 foot sections the contraction movement at the quarter points was 1% of the unrestrained slab, while the contraction at the ends was 20% of a free slab. The expansion at the quarter points and ends was 2% and 20% respectively of an unrestrained slab.

8) Over an annual period there was a greater overall length change than over a short period of time. For an 84^{o}F annual temperature variation the net movement was as follows:

840 foot sections - 2.7 inches
1070 foot sections - 3.2 inches
1310 foot sections - 3.3 inches

9) The net growth of the pavement over a ten year period had the following range between February and August.

600 foot sections - 0.55" - 2.25"
1070 foot sections - 1.10" - 3.75"
1310 foot sections - 0.90" - 3.80"

10) None of the transverse cracks showed any evidence of inelastic deformation of the steel that would contribute to length changes.

11) The maximum change in elevation over a ten year period was 0.8 inches.

12) The occurrence of pumping at the end of 10 years, with a single exception was limited to the expansion joints. There was but a single instance of pumping in the transverse cracks where the steel exceeded 0.24%. Even where corner cracks had developed the reinforcement held them so tight that there was no evidence of pumping. A certain amount of edge pumping developed, but this was not due to cracking.

13) The smoothness of the pavement was not appreciably affected by the cracking. Between 1940 and 1949, the intermediate length was ob-

served to have an increase in roughness index of 51 while in the same period the long sections showed an increase of 40. In 1949, the greatest measured roughness index was 127. Indices ranging from 80 to 120 are classed as smooth riding.

Based on the above observations, the following conclusions were drawn by the investigators in question.

1) Small, non-uniform changes in pavement elevation have no apparent effect on the crack pattern.

2) The center portions of sections longer than 800 feet are in complete restraint under daily temperature flucuations. The center sections of slabs greater than 1310 feet are in complete restraint under annual temperature flucuations. The greatest tensile stresses probably occur in the fall.

3) Moderate progressive changes in length in the longer sections are assumed to be due to the progressive opening of cracks.

4) The average crack width in the heavily traveled traffic lane varied for the longer sections from 0.011 inches for 0.45% to 0.004 inches for 1.82% of reinforcement.

5) 67% of the cracking appeared during the first year. In subsequent years few cracks appeared during the winter months.

6) Based on the experience of this research, it was deemed that the better reinforcement in the completely restrained areas would result in a crack interval of from 2 to 2-1/2 feet.

7) The heavily traveled lane produced 6% more cracks than the lightly traveled lane. The cracks in the heavily traveled lane showed more superficial damage than the other lane.

8) Since the length changes and the crack pattern were symmetrical in the center portions, it was concluded that the pavement behavior was predictable.

9) Pumping developed at many expansion joints but were effectively absent from the transverse cracks.

10) The smoothness of the surface has been maintained. After 10 years the pavement is no rougher than some conventional pavements in the as constructed condition.

The results of the performance survey conducted 15-1/2 years after construction were reported in 1955[6.6]. The findings of this survey were as described below.

1) The shorter sections were comparatively free of cracking; but as the length of the slab increases the cracking becomes more frequent until in the central regions of the longest slabs the crack interval was often less than 2 feet.

2) The rate of cracking was very high during the first year of service being as much as 65% of total for 310 foot slabs and 40% for the central 400 feet of the 1310 foot slab.

3) The rate of cracking for the 310 foot slab showed a marked decrease after the third year with only 19% of the cracks occurring in the later 12-1/2 years of service.

4) The rate of cracking for the central 400 feet of the 1310 foot section showed an almost constant rate of cracking. 13% of the cracks formed in the later 5-1/2 years.

5) In the central portion of the 1310 foot section with 1.82% reinforcement the crack interval varied between 1 and 6 feet and averaged 2.3 feet. The largest decrease in crack interval occurred between the first and third years of life, with practically no change in the interval in the later 5-1/2 years.

6) The crack interval variations for the 310 foot sections reinforced with 0.42% of welded wire fabric showed an average spacing of 31 feet in the first year, 26 feet in the second year, 23 feet after 10 years and 22 feet at the end of 15-1/2 years.

7) Irrespective of crack incidence or rate of development, all sections, except those reinforced with the lightest amount of steel, have continued in service free from distress at the cracks other than inconsequential edge raveling and very limited damage caused by localized sealing and spalling.

8) Cracks in the lightly reinforced short and moderate length sections are generally wider than those in the more heavily reinforced longer sections.

9) For a given long section the cracks in the central portions are wider than those in the end regions.

10) The surface width of a crack tends to decrease with an increase in the percentage of steel in a linear manner.

11) The surface crack width tended to increase with pavement age. The maximum width at the end of 10 years was 0.11 inches while at the end of 15-1/2 years it was 0.13 inches.

12) The surface width of the cracks at the end of 15-1/2 years for the 1.82% section was 0.11 inches.

13) At the end of 15-1/2 years all sections containing percentages of steel 0.17% and greater had maintained cracks in a closed condition.

14) A number of open cracks, indicative of steel failures developed in sections containing 0.07 and 0.11% of steel.

15) After the steel failed, the cracks opened up and pumping and faulting developed rapidly and bituminous maintenance was shortly required to alleviate surface roughness.

16) The first steel failures were observed at the end of 8 years, while the vast majority of failures occurred between 10 and 15-1/2 years of age.

17) Pumping was noticed at the bridge type expansion joints early in the life of the pavement. Since these joints did not have effective load transfer the pavement deflections in these portions of the pavement are larger than elsewhere. After 5 years of service many of these joints were pumping, faulting was noticeable and flexural cracking was observed due to loss of subgrade support. At the end of 15-1/2 years, nearly all the bridge type expansion joints in the heavily traveled lane were subject to pumping, and in many instances flexural cracking, serious settlement and severe faulting of the slab ends had

developed. Very few of the companion joints in the passing lane were affected.

18) Seven years after construction, incipient pumping appeared at several of the conventional doweled expansion joints which separate the shorter sections. After 10 years pumping was observed at a number of these joints, but the action was still so slight that faulting was negligible and flexural cracking of the slab ends had not occurred. At the end of 15-1/2 years, however, the performance survey showed that pumping at the doweled joints in the heavily traveled lane had progressed to the point of causing considerable structural damage. Evidence of pumping was observed at more than half of the joints. Approximately one third had pumped to the extent that flexural cracking had developed in the slab ends and faulting, as much as 1/2 inch, was present at the outside corners of several. In contrast, pumping was completely absent and faulting was negligible at the same type of joint in the passing lane.

19) The daily construction joints in the sections containing welded wire fabric were formed by 5/8 inch deformed tie bars, 4 foot long, and spaced 2 feet on centers. Of 11 joints in the heavily traveled lane of this type, the 15-1/2 year survey showed that five joints were subject to pumping, one of which to the extent of flexural cracking. The widths of these joints ranged from tightly closed to 3/4 inch. Faulting had developed in nearly all the joints with a maximum of 3/4 inch. The majority of these joints were subject to moderate surface spalling requiring maintenance. In the passing lane pumping had not appeared at any of the joints, but widths were consistent with those in the heavily traveled lanes and slight faulting along with minor spalling had developed.

20) The construction joints in the sections containing bar reinforcement were formed by running the steel continuously through the joint. Twenty-two of the twenty-five joints of this type were found to be structurally sound, tightly closed and free from pumping. The three unsound joints of this type were open 3/8 to 1/2 inches indicationg that the steel had failed at the joint. The failed joints in the heavily traveled lane had pumped in sections of billet-steel reinforcements with #2, #3 and #4 bars. The failures occurred sometime after the tenth year of pavement life.

21) Pumping in various stages was noted at a number of the open transverse cracks that formed in the sections reinforced with the lighter amounts of steel. In contrast to the action of open cracks, neither pumping nor faulting had developed at the transverse cracks held closed by the continuous reinforcement.

22) Pumping along the free edges of the pavement in the mid length of a section was observed during the 10 year survey. The pumped areas were those in which the subgrade was soft, spongy and unstable to such an extent that the side forms had to be supported on mudsills of heavy planking during placement of concrete. At the time of the 15-1/2 year survey, a few additional locations showed evidence of edge pumping, these all being in the heavily traveled lane of relatively long sections. The action, however, had been slight and the pavement in these areas was in no immediate structural jeopardy.

23) The 10 year crack survey indicated that 53% of the transverse cracks then present in both lanes of the pavement had formed in the heavily traveled lane. A similar survey at the end of 15-1/2 years indicated that

51% of the cracks in the central portion of the 1310 foot section were in the heavily traveled lane.

24) The surface crack widths in the heavily traveled lanes were approximately three times wider than cracks in the passing lane.

25) The roughest pavements in the heavily traveled lane were in the very short, lightly reinforced sections, although the basic pavement roughness is somewhat less than conventional pavements.

Based on the above observations, the investigators drew the following conclusions.

1) The central sections of long slabs maintain themselves in a condition at high stress with continued bond between the steel and the concrete.

2) The crack interval in the central portion of long slabs is independent of the slab length and dependent on the amount of steel.

3) Load-transfer devices had considerable value in resisting and delaying the action of pumping, as compared to the bridge type expansion joints.

4) Shear bars in the daily construction joints did not develop sufficient bond or lacked the strength to perform properly their intended function of maintaining adjoining slabs in close contact.

5) The appearance of pumping at the cracks shortly after the reinforcing steel ruptured is indicative of the structural weakness resulting from this type of failure and emphasized the importance of maintaining all cracks in a closed condition.

6) Considering the prevalence of pumping and faulting elsewhere in the pavement, the lack of same for fine transverse cracking, indicates the watertight character and inherent structural strength of cracks maintained in a closed condition by reinforcing steel.

7) The repetition of traffic loads has exerted some influence on the development of transverse cracks, particularly in the shorter sections. The influence is slight, however in the restrained central portions of the long slabs.

8) The need for prevention of pumping is greater for the heavily traveled lanes than for the passing lanes of divided highways.

9) Sections of any length can be reinforced with continuous steel in sufficient amount to maintain all transverse cracks in a tightly closed condition, without adverse effect on the concrete.

10) Up to a given length, the length of a slab containing adequate steel will have a pronounced effect on the crack pattern. Slabs up to 150 feet long will contain comparatively few variably spaced cracks.

11) The rate of crack development and the crack frequency will have little effect on the service life of adequately reinforced pavements.

12) The surface width of cracks held closely by continuous reinforcements will increase gradually with time by raveling and superficial wearing due to exposure and traffic. This condition, however, will not impair the durability of the concrete and will require little maintenance.

13) Cracks held closed by continuous reinforcement will be highly resistant to pumping, practically free of faulting and will seldom

develop surface roughness.

14) Percentages of steel of 0.07% for welded wire fabric and 0.11 for reinforcing bar appear to be structurally incapable of performing the intended function of holding transverse cracks permanently closed in sections subjected to heavy traffic.

15) Continuously reinforced pavements will be less susceptive to the damaging effects of pumping than rigid pavements of conventional design. Continuous pavements, however, will not be immune from pumping action at the longitudinal free edge of the pavement.

3. RESEARCH IN ILLINOIS

In the fall of 1947 the Illinois Division of Highways started construction of a continuously reinforced experimental highway. This highway consisted of eight test sections, in which the pavement thickness was varied between 7 and 8 inches and the amount of reinforcement was 0.3%, 0.5%, 0.7%, and 1.0%. The variations in the reinforcement was accomplished by using #3, #4, and #6 deformed rail steel bars with spacings that varied between 4-1/2 and 6-1/4 inches on centers. Each section was divided into two subsections of approximately equal length between which the variable was the spacing of the #3 transverse bars. In one subsection the transverse bars were placed 12 inches on center, while in the other the spacing was 18 inches. Mill tests on the rail steel bars indicated a yield point of from 66,000 to 78,700 psi and an ultimate strength varying from 107,300 to 124,700 psi. The elongation varied from 11.7 to 14.5%. The length of six of the continuously reinforced section was 3,504 ft., while the length of the two sections containing 1.0% of steel was 4,233 feet.

The pavement was placed directly on the subgrade. This was contrary to the usual practice in Illinois, but was done to test the arguments that continuous reinforcement will eliminate pumping. The subgrade was composed of so-called "pumping" soils in which 35% were of the A-7-4 AASHO classification. Of the remainder of the soil types, 33% was of the A-4, 11% of the A-2 group and 10% of the A-6 group. It is known that A-4, A-6 and A-7 soils are susceptible to pumping. Thus 89% of the subgrade is in a potential state of pumping.

The longitudinal reinforcement consisted of bars 30 feet long placed on chairs, at the mid-depth prior to pouring the pavement. All longitudinal bars were lapped 30 diameters. The concrete was placed in one lift. At the end of a days pour the construction joint was formed by running the reinforcement through a headerboard construction joint. The adajacent slab test sections were separated by a 4 inch expansion joint with asphalt filler.

The instrumentation used to measure the response of this pavement were as follows.

1) In order to study the longitudinal and vertical movements of the pavement, reference monuments were constructed at each end and at intervals of 400 feet from one end to the center of each test section. For 24 inches on either side of the reference monuments brass plugs were set in the pavement, so that an inclinometer could be used to measure pavement deformation.

2) Near the middle of each test section 36 brass plugs were set in the pavement 10 inches apart. These plugs were set along the edge, and were for the purpose of measuring crack widths that were expected to develop

in these areas. The movement of these plugs was measured with a Whittemore gage.

3) Pavement elevations were determined from reference plugs placed at 200 foot intervals along the pavement edge. Plugs were also placed along transverse lines opposite each monument.

4) In order to measure the stress in the longitudinal steel several bars were mounted with SR-4 gages. These gages were placed in the middle of the 7 and 8 inch sections containing 0.7% of steel. 413 active gages were used divided approximately equally between each test section. The gages were located along eight traverse lines 2-1/2 feet apart, perpendicular to the center-line of the pavement. In order to eliminate the effects of flexural stress on the gage readings the gages were mounted on the side of the bars. In order to assure that at least one line of gages were at a transverse crack, a depressed contraction joint was installed along one of the lines of gages.

5) Thermocouples were installed at various levels in the concrete and subgrade to measure temperature.

All the 7 inch sections and an 8 inch section containing 1.0% reinforcement were constructed in the Fall of 1947. The remaining 3 sections of the 8 inch pavement were constructed in April and May of 1948.

The initial program and schedule of observations planned were as follows.

1) A detailed crack survey at least once each year on each section.

2) Measurements of horizontal and vertical movements of the pavement with respect to the reference monuments. These measurements were planned to be taken four or more times a day during one week in each of the four seasons of the year. These measurements were also planned to include the width of the expansion joints separating the test sections.

3) Strain measurements in the reinforcing steel were planned to be made hourly during one week in each of the four seasons of the year.

4) Temperature readings of the concrete and subgrade were planned to be made hourly during one week in each of the four seasons of the year.

5) Precise levels were planned for twice a year.

6) Once a year it was planned to make a condition survey.

Early observations were reported for those sections constructed in the Fall of 1947[6,9]. These cracks were so tight that they were extremely difficult to find even in the morning. After 37 days a crack survey was made on the 7 inch 0.3% section. The average crack interval was 17.34 feet with a minimum interval of 9.91 feet. For the 7 inch 0.5% section at the end of 34 days the average interval was 18.44 feet and the minimum was 10.00 feet. After 32 days in the 7 inch, 0.7% section the average interval was 13.53 feet, with a minimum of 6.67 feet. After 22 days the 7 inch 1.0% section had an average crack interval of 8.48 feet with a minimum of 4.76 feet. After 18 days, in the 8 inch, 1.0% section, the average crack interval was 19.56 feet, with a minimum interval of 6.25 feet.

2) On the afternoon of the third day after pouring curing paper was removed. The following morning a transverse crack at one of the depressed contraction joint opened to a width of 0.010 inch and the crack gages indicated strains of approximately 1,000 micro-inches per inch (micros). This was equivalent to a stress of 30,000 psi. By 2:00 p.m. that afternoon the above described crack was closed so tight that it was hardly visible. At this time the steel stress had reduced to 600 psi. During this period strains registered away from the crack were never higher than the equivalent of 4,000 psi.

3) Strain readings in this initial period were taken almost daily between October 6, 1947 to November 25, 1947. The trend of these readings indicated relatively high strains in the morning, accompanied by a measurable crack opening, followed by a reduction of strain in the afternoon and a closing of the crack. No significant changes in the strains away from the crack occurred during this period.

4) In the early period of observation the maximum average unit strain was 1310 micros which is equivalent to a stress of 39,300 psi. The slab temperature change in the same period was from a contructed temperature of 82^{o}F to a minimum of 39^{o}F.

The performance of the Illinois pavement was reported on in 1950[6,10]. This report covered an age of the test sections of less than 3 years, with 5 sections going through 3 winters and the other three going through 2 winters. The reported results up to this time are as follows.

1) During the first winter the maximum average stress at a crack in the 7 inch slab was 62,400 psi, which occurred on January 21, 1948. The reported stress was an average of 9 gages, none of which varied more than 4% from the average. This stress was in the vicinity of yielding. During the reported period the gages away from the crack indicated, stresses that were never higher than 10,000 psi, indicating that so long as the concrete is structurally continuous it will carry a large part of the total stress. As an example of the efficiency of the bond, one line of gages showed no measurable effect from a transverse crack only 10 inches away.

2) The maximum stress measured in Januray 1949, for the 8 inch, 0.7% section was 42,000 psi. This section was poured in April 1948, and at this time was subjected to total temperature change of 45°F.

3) The SR-4 gages in both sections became erratic in a relatively short time, and as a result it was not possible to obtain strain measurements for more than one cycle of seasonal contraction. The gage failures were believed to be caused by moisture, despite the waterproofing efforts.

TABLE 4

LONGITUDINAL STEEL STRESS MEASUREMENTS - ILLINOIS

Date	A.M. or P. M.	Stress (psi) Tension	Compression
a) 7-inch pavement, 0.7% steel			
Oct. 7, 1947	A.M.	4,500	
	P.M.		3,000
Oct. 10, 1947	A.M.	30,000	
	P.M.		200
Nov. 1947	A.M.	37,000	
	P.M.	28,000	
Dec. 1947	A.M.	34,000	
	P.M.	28,000	
Jan. 1948	A.M.	62,000	
	P.M.	38,000	
Apr. 1948	A.M.	17,000	
	P.M.	5,000	
b) 8-inch pavement, 0.7% steel			
Apr. 6, 1948	A.M.	4,000	
	P.M.		2,500
Apr. 27, 1948	A.M.	28,000	
	P.M.	2,000	
June 1948	A.M.	3,000	
	P.M.		3,000
Sept. 1948	A.M.	27,000	
	P.M.	2,000	
Jan. 1949	A.M.	42,000	
	P.M.	25,000	

4) There seemed to be a relationship between the crack interval and the amount of longitudinal steel. The relationship is shown in the following table.

TABLE 5

CRACK INTERVAL MEASUREMENTS - ILLINOIS

Date	Average transverse crack interval (ft.)			
	0.3% Steel	0.5% Steel	0.7% Steel	1.0% Steel
a) 7-inch pavement				
Mar. 1-4, 1948	21.6	14.9	11.5	9.0
Dec. 1-2, 1948	19.6	13.3	9.2	6.7
Sept. 26, 1950	15.9	9.9	7.6	5.6
b) 8-inch pavement				
Mar. 1-4, 1948	--	--	--	11.6
Dec. 1-2, 1948	19.5	18.2	14.2	9.1
Sept. 26, 1950	14.2	10.7	8.9	7.0

5) The crack interval distribution followed a pattern somewhat different than in Indiana. Beginning at the ends, the frequency increased at a fairly uniform rate to a maximum at a distance ranging from 300 to 500 feet from the ends of the sections, from whence it decreases uniformly for several hundred feet over the central 1600 to 2000 feet of the section. In general the crack frequency is lower for the thicker pavement and this trend intensifies as the amount of reinforcement increases.

TABLE 6

FREQUENCY OF TRANSVERSE CRACK - ILLINOIS - 3 YEAR SURVEY

% Steel	Average Crack Spacing (ft.) 200-500 ft.	800-900 ft.	(Dist. from Ends) Center
a) 7-inch pavement			
0.3%	8.3	13.3	13.3
0.5%	4.8	10.0	9.1
0.7%	5.0	7.1	6.3
1.0%	4.0	4.5	4.5
b) 8-inch pavement			
0.3%	7.7	16.7	11.8
0.5%	5.6	7.1	11.4
0.7%	5.9	7.5	7.2
1.0%	4.2	5.0	7.2

6) The crack width is a function of the steel stress and effectiveness of the bond between concrete and steel in the immediate vicinity of the crack. The cracks in the 0.3% sections are much wider than the others and the cracks in the 1.0% section are the tightest. The only spalling that existed at this time showed up in the 0.3% steel sections. The crack widths vary seasonally, being relatively wider in winter when the steel is under greater tension. The cracks were becoming progressively wider with age and do not close tightly in the summer when the pavement expands, but appear to be remaining further open with each successive summer. There seems to be an inverse relationship between the average crack width and the percentage of steel.

TABLE 7

AVERAGE CRACK WIDTH - ILLINOIS - 3 YEAR SURVEY

Date	Slab Temp. °F	Average width (in.) in terms of Steel % 0.3%	0.5%	0.7%	1.0%
a) 7-inch pavement					
July 28, 1948	94	0.010	0.006	0.005	0.003
Feb. 2, 1949	36	0.017	0.009	0.008	0.004
July 7, 1949	105	0.013	0.014	0.009	0.008
Feb. 27, 1950	46	0.020	0.015	0.012	0.007
Aug. 30, 1950	82	0.021	0.015	0.013	0.007
b) 8-inch pavement					
July 28, 1948	94	0.005	0.004	0.004	0.003
Feb. 2, 1949	36	0.015	0.010	0.090	--
July 7, 1949	105	0.013	0.012	0.014	0.004
Feb. 27, 1950	46	0.017	0.014	0.014	0.006
Aug. 30, 1950	82	0.016	0.012	0.011	0.007

7) Soil stains had been observed in a number of cracks, particularly in the 8 inch section with 0.3% of steel at a time when the crack width averaged 0.015 inches.

8) The change in length seemed to vary inversely with the amount of steel; the more steel the greater the contraction in the winter and the less the summer expansion. All sections experienced a permanent length increase. By Aug. 30, 1950, the length had increased between 2 1/4 and 3 1/2 inches for all sections. In the 7 inch pavement there was a net decrease in length over the first two winters, the second winter having

less contraction than the first. The third winter showed a net increase in length. The greatest length increase occurred in the 7 inch section between the first and second summer of service. The same trend was apparent for the 8 inch section. The winter contractions becamse greater as the amount of steel increased. The sections with the most steel showed the least summer expansion.

9) The general increase in length was attributed to foreign matter entering the cracks and rendering them ineffective in absorbing expansion and contraction.

10) No appreciable movement was initially observed in the central 2700 test of the 3500 foot sections, and in the central 3400 feet of the 4200 foot sections. All the movements were produced in the end 400 feet. The later observations indicated that the length of the stable central section was becoming smaller due to infiltration of incompressible material.

11) Transverse cracks were for the most part invisible to motorists, even when traveling at moderate speeds.

12) Since the pavement was built without a longitudinal joint, longitudinal cracking developed, as expected.

13) At a construction joint near the center of the 8 inch slab with 0.3% steel an 8 foot length in one lane developed four closely spaced cracks that started pumping in October 1940, five months after construction. Even with the pumping, no cracks appeared in the slab on the other side of the construction joint. The same cracks extending to an adjacent

lane were not nearly so critical. The construction ahead of this joint was delayed nine days, due to rain, and when it was resumed the subgrade was soft and uncompacted. In addition to these causes the concrete was found to be of poor quality. About 1000 feet behind this joint, in the same lane and same section, three feet of concrete between two cracks started pumping and was rapidly deteriorating. In May 1950, the poor portion of the slab for 8 feet ahead of the joint was removed in the first case, and 3 feet of concrete between cracks in the second case was removed. The subgrade in both instances was wet and unstable and was replaced with sand. The removed sections were replaced with high quality concrete.

14) Other than the above mentioned failures, no pumping has occurred at any of the transverse cracks, although moderate pumping has been observed at the expansion joints between sections.

The authors of the 1950 report offer the following conclusions and interpretations of the test observations.

1) In view of the high steel stresses measured in the 0.7% steel, it may be necessary to provide more steel than has been indicated by earlier experiments.

2) The maximum tensile stresses in the steel occur in the winter and are proportional to the temperature drop and inversely proportional to the amount of steel. (Note: This is entirely in agreement with Friberg's theory).

3) It appears that the critical stress in the steel will not occur where the concrete is unbroken, a condition which assists the steel in carrying stress.

4) The theory of continuous reinforcement assumes that the occurrence of frequent transverse cracks will afford stress relief and absorb a large portion of the daily and seasonal length changes of the slab. Thus, if the pavement contains adequate steel, frequent cracks are induced by an alternate interchange of stress between the steel and the concrete. When the concrete cracks, the steel assumes the full stress at that location, but by virtue of bond, the stress is quickly transferred to the concrete and another crack eventually develops. The behavior of all test sections has been according to theory in the respect that large numbers of transverse cracks have developed in each section and new cracks are continually developing.

5) The progressive widening of the transverse cracks was considered to be due to three contributing factors. In the first place, incompressible material entering the cracks may prevent them for closing fully. Secondly, the steel bars may have a somewhat higher thermal expansion coefficient than the concrete and the greater expansion of the steel may tend to keep the cracks from closing. Thirdly, the permanent shrinkage of the concrete may be sufficient to produce a permanent opening at cracks. It is the authors conclusion that the first factor is largely responsible for the progressive crack opening.

6) The fact that the winter contraction of entire slabs is inversely proportional to the amount of steel (less contraction with more steel) is interpretated as being due to the fact the the cracks in the lighter reinforced sections, being wider, take up a relatively larger part of the total contraction.

7) It is suggested that overall performance would be improved by a combination of continuous reinforcement and a thin granular base course.

A report of performance was made after the pavement was in service 3 1/2 to 4 years. (6.11) This report covered an inspection made in Septembef 1951. The findings were as follows:

1) The crack interval was tabulated for the sections of 7 inch pavements excluding the end 150 feet. This exclusion was made to eliminate the undue influence of the unrestrained ends.

TABLE 8

AVERAGE CRACK INTERVAL (7-INCH PAVEMENT)-ILLINOIS-4 YEAR SURVEY

Date	Average Crack Spacing (ft.)			
	0.3% Steel	0.5% Steel	0.7% Steel	1.0% Steel
Mar. 1948	18.7	16.9	13.7	12.6
Dec. 1948	13.0	11.5	8.5	7.6
Sept. 1950	10.0	8.0	6.5	5.7
Sept. 1951	7.8	5.9	5.0	4.6

2) The crack frequency increases at a uniform rate to a maximum at a distance from 200 to 500 feet from the ends. After this point the frequency decreases irregularly for a few hundred feet until it stabilizes and remains fairly uniform over the central 1600 to 2000 feet.

3) There is a substantial decrease in crack width with increase in steel percentages and an increase in crack width with age.

TABLE 9

AVERAGE CRACK WIDTH (7-INCH PAVEMENT)-ILLINOIS-4 YEAR SURVEY

	Crack width (in.)			
Date	0.3% Steel	0.5% Steel	0.7% Steel	1.0% Steel
July 1948	0.010	0.006	0.005	0.003
July 1949	0.013	0.014	0.009	0.008
Aug. 1950	0.021	0.015	0.013	0.007
Aug. 1950	0.023	0.017	0.013	0.009

4) The crack widths in the 8-inch pavement were generally similar to those in the 7-inch pavement.

5) The section that was repaired in 1950 was in a sound condition and there has been no progress of the deterioration.

In July 1952 a careful survey was made of the Illinois project.(6.8) This survey included the visible condition of the pavement, the crack frequency at the ends, quarter points and construction joints. The results of this survey are as follows:

1) Presence of oil smudges indicated a pavement dip of one inch spanned over 75 feet. The changes in elevation were so gradual that it had no visible effect on spacing or width of cracks. High speed driving produced only slight sensations of changes of level. The settlement was believed to be caused by consolidation settlement of a moderate fill.

2) The crack spacing was measured in the slabs excluding the end 300 feet of each section.

TABLE 10

CRACK SPACING-ILLINOIS-1952

Location	% Steel	Spacing (ft.) Maximum	Minimum	Average
a) 7-inch pavement				
West 300-400 ft.	0.3	9	4	4.6
West 800-900 ft.	0.3	15	7	8.5
Center 200 ft.	0.3	15	11	12.5
East 800-900 ft.	0.3	14	7	7.7
East 300-400 ft.	0.3	14	6	8.3
West 300-400 ft.	0.5	6	3	4.8
West 800-900 ft.	0.5	14	5	8.3
Center 200 ft.	0.5	11	3	7.1
East 800-900 ft.	0.5	11	4	7.1
East 300-400 ft.	0.5	8	2	4.8
West 300-400 ft.	0.7	10	4	5.9
West 800-900 ft.	0.7	9	4	5.3
Center 200 ft.	0.7	9	3	5.9
East 800-900 ft.	0.7	12	3	5.9
East 300-400 ft.	0.7	6	2	4.0
West 300-400 ft.	1.0	10	3	5.3
West 800-900 ft.	1.0	7	3	4.2
Center 200 ft.	1.0	8	2	4.7
East 800-900 ft.	1.0	9	2	5.3
East 300-400 ft.	1.0	9	3	4.5
b) 8-inch pavement				
West 300-400 ft.	0.3	12	4	8.3
West 800-900 ft.	0.3	15	8	14.3
Center 200 ft.	0.3	14	9	12.5
East 800-900 ft.	0.3	14	6	9.1
East 300-400 ft.	0.3	12	2	6.3
West 300-400 ft.	0.5	14	3	5.5
West 800-900 ft.	0.5	14	3	7.7
Center 200 ft.	0.5	14	3	8.3
East 800-900 ft.	0.5	10	3	6.7
East 300-400 ft.	0.5	8	3	4.5
West 300-400 ft.	0.7	8	3	5.5
West 800-900 ft.	0.7	10	3	5.9
Center 200 ft.	0.7	8	2	6.3
East 800-900 ft.	0.7	14	3	7.1
East 300-400 ft.	0.7	9	2	5.5
West 300-400 ft.	1.0	10	4	5.0
West 800-900 ft.	1.0	12	3	7.7
Center 200 ft.	1.0	10	4	6.6
East 800-900 ft.	1.0	10	3	6.2
East 300-400 ft.	1.0	10	2	5.9

3) The crack spacing decreased with increasing steel percentage in a reasonably similar pattern in each section. The widest spacing is at the section ends. The closest spacing is at 300 to 400 feet from the ends, and the spacing again widens in the following 500 feet. The remaining 2,000 feet of center has uniform spacing.

4) The transverse crack orientation varied from straight and continuous across the pavement, to straight and discontinuous at the longitudinal crack to sloping or curving cracks.

5) In the heavier reinforced sections there was a slight grouping tendency where two to four transverse cracks were spaced 2 to 2.5 feet apart and the group was spaced 9 to 12 feet.

6) The width of transverse cracks decreased with an increase in percentage of steel. There was evidence that for the same amount of reinforcement the crack width in the 7-inch pavement was slightly wider than in the 8-inch pavement.

7) 60% of the expansion joints showed indications of pumping.

8) Eleven of 27 construction joints showed indications of pumping. These joints also indicated faulting between 1/8 inch and 3/8 inch.

The Illinois pavement was reexamined in October 1952 to ascertain a comparison between summer and fall conditions.

Even in the cool early morning no crack was observed with more than 1/64th of an inch of surface width. All performance conditions were as observed the previous summer.

The conditions found in the examination after five years seem to justify the following conclusions.

1) After 5 years of service under heavy traffic on a critical soil and no special subgrade preparation, all sections were sound.

2) There was no visible departure from the original grade line, and at all speeds the driving conditions were excellent with no bumps or other effects from the transverse cracks.

3) None of the approximately 4300 cracks showed evidence of structural unsoundness caused by raveling, spalling, faulting, or pumping.

4) Although slight distress was noted at the expansion joints and the daily construction joints, no distress was found at the transverse cracks which were held tight by the longitudinal reinforcement.

5) There was no evidence that the longitudinal steel has exceeded its elastic limit. There was every reason to believe that sections with 0.5% or more of steel would continue to maintain the structural integrity of the transverse cracks.

6) Expansion joints are dangerous and unwarranted. It is desired that special care or improved design and treatment of expansion joints be developed.

7) Extensive longitudinal cracking has occurred even though these cracks are uniformly sound and are not in any stage of deterioration, even at intersections with transverse cracks. Since longitudinal cracking is inevitable for wide slabs, the omission of a formed longitudinal joint is not warranted.

The report of a 10 year survey will be presented at the January 1958 meeting of the Highway Research Board. It was found in the survey that construction joints in the 0.3% and 0.5% sections had failed and that longitudinal steel failures had occurred in the 0.3% section at some of the transverse cracks.

4. RESEARCH IN NEW JERSEY

A continuously reinforced experimental pavement was constructed in New Jersey in September and October 1947. These sections were constructed on a highway which was heavily traveled by large trucks. The highway was a dual lane road of two 12 foot lanes without tie bars. This feature was a departure from other experiments in Indiana, Illinois, and California. The subgrade soil that had as its largest size component the 3/8 inch size and between 18 and 51% passing the number 200 sieve. The plasticity index of the subgrade was in no case greater than twelve. The subgrade was uniformly in cut with an average of 2 feet of excavation, and was prepared by use of a pneumatic-tired roller and a 3-wheel 10-ton roller. The highway rested on a free draining granular base course, the thickness of which varied from 12 inches under the 10-inch pavement to 14 inches under the 8-inch pavement. The 8-inch pavement contained 0.90% of longitudinal steel and was 5,430 feet long. The 10-inch pavement was 5,130 feet long and contained 0.72% of longitudinal steel. The longitudinal steel consisted of welded wire fabric, fabricated into mats 16.25 feet long, and 6 feet wide. The 3/8" longitudinal cold drawn wires were spaced 3 inches on center, while the No. 5 transverse wires were spaced 12.2 inches apart. The ultimate tensile strength of the wire was 84,600 psi. The reinforcement was placed in two layers by the strike-off method, the layers being 3 inches from the top, and 3 inches from the bottom of the pavement respectively. The concrete had a 28 day compressive strength of 5042 psi and a 5 week tensile strength of 300 psi. The ends of the experimental pavement were formed by a 3/4" doweled expansion joint. The daily construction joints were formed with steel running continuously through the joint a minimum of 6 feet. The pavement was cured with membrane curing compound.

The research conducted on these pavements was for the following specific purpose.(6.14)

1) To determine if the reinforcement was capable of preventing the opening of transverse cracks to non-objectionable widths.

2) To determine the magnitude of the daily, seasonal and annual changes in the over-all pavement length.

3) To determine if a central zone in the pavement remains at a constant length.

In order to accomplish the above purposes the following instrumentation was installed.

1) To determine the width of cracks with the central portions of the pavement, brass gauge plugs were installed in the pavement 10 inches apart in various locations. As many as 6- consecutive plugs were installed along the longitudinal lines on the pavement. Initial readings were taken as soon as the concrete had hardened.

2) Concrete reference monuments were installed along sides of the pavement to determine the magnitude of longitudinal movement of the pavement. These monuments were located on either side of the pavement establishing a series of transverse lines across the pavement.

3) Special plugs were installed in the pavement both longitudinally and transversely and in series to determine the changes in length that occur in specific areas. These plugs were spaced 20 inches apart and were measured to an accuracy of 0.0001 inches.

4) Gauge plugs were installed at all transverse joints in the pavement.

5) Permanent temperature wells were spaced longitudinally throughout the pavement.

6) In particular areas, pavement levels were taken at 5 foot intervals along both edges of the pavement lanes. The level measurements were taken to the nearest estimated 0.001 feet.

A supplementary experiment was performed in New Jersey, consisting of a series of slabs 187 feet long. These slabs had additional reinforcing in the central portions and were gauged with plugs in various portions of the slab. The reinforcement for this section consisted of a single line of welded wire fabric of No. 00 gage wires 6 inches on centers for the end 34 feet. The steel in the central 119 feet consisted of a single line of steel of the same type as in the continuously reinforced section, resulting in a percentage of 0.36%.

The following early developments were reported:

1) The concrete poured early in the morning achieved a temperature 10 to 20 degrees higher than that placed in the afternoon, with the greatest differential occuring on warm sunny days. Consequently, transverse cracking occurred first in the sections poured early in the day and these sections had the closest crack spacing. The temperature of the afternoon placed concrete remained constant over night and thus the end of the pavement was in the as constructed condition on the second morning.

2) A survey in early November 1947 when the slab temperature was approximately 48° F indicated that the ends of the 10-inch slab had contracted between 0.29 and 0.40 inches with the greatest movement occurring at the end of lanes constructed during the warmest weather. The contraction 200 feet from the ends averaged 0.10 inches and no movement had occurred 500 feet from the ends. The 8-inch section had contracted 0.22 to 0.28 inches at the ends. Practically no movement had occurred 200 feet from the ends.

3) In the 10-inch section in November 1947 the average crack width was 0.01 inches. This was in an area where the pavement temperature reached 105° F within 12 hours after construction. The crack width in the 8-inch section averaged 0.008 inches with a maximum of 0.013 inches.

4) In November 1947 the construction joints in the 10-inch section were open an average of 0.01 inches with an maximum of 0.03 inches. In the 8 inch section the construction joints were open an average of 0.005 inches with a 0.009 inch maximum.

5) In early November 1947 the crack interval was very erratic ranging from approximately 1 to 58 feet. The highest crack frequency was in the sections constructed early in the morning. In these areas there were lengths 200 to 300 feet in which the crack interval averaged 3 to 4 feet.

6) There were no cracks in the 187 foot slabs. The joints between these slabs had opened from 0.3 to 0.6 inches.

A complete survey of the New Jersey continuously reinforced pavement project was made in October 1950, representing 3 years of service. The following observations were reported.(6.15)

1) The 1950 traffic on this highway consisted in part of a daily volume of 1,750 tractor-semitrailer units with a legal gross weight of 30,000 pounds and a single axle load exceeding 30,000 pounds, depending on tire size.

2) The longitudinal movements of the pavement indicated that for the 8-inch pavement the maximum end movement was a contraction of 0.38" which took place in January 1950 on a day of 41° F. temperature. The minimum movement was 0 .04" of contraction measured on October 1950 when the temperature was 73° F. In general, the movement 200 feet from the ends was between 1/2 and1/10 of the end movement. The total maximum movement of the central portion (excluding end 200 feet) was 0.12" in one instance, but the general trend of data indicated this movement to be between 0.02 and 0.07 inches. The movements appeared to be seasonally reversible. The trends for the 10-inch pavement movement were similar in all but one respect. This respect was that the magnitude of the end movement was greater having a maximum of 1.0 inches in January 1948 when the temperature was 35°F. Thus, the total change in length of the pavement measured in October 1948

was 1.23 inches for the 8-inch pavement and 1.82 inches in the 10-inch pavement. There was some difference in the time of readings for both sections, and the end joint measurements indicate that the change of length of all sections was essentially the same, and was approximately 2 inches.

3) In general, the end joints had undergone an increase in width. This increase varied between 1/4 and 3/4 of an inch. This change in width was not a change in length of the test slab, but from movements of the adjacent 56 foot slabs. The causes are believed to be the infiltration of foreign matter in the joints.

4) The ends of all the continuous slabs showed an appreciable uncracked distance. The end encracked distance of the 8-inch pavement varied from 64 feet on one end to 114 feet on the other end. The uncracked distance for the 10-inch pavement varied from 161 to 177 feet.

5) The central portions of all sections contain a very large number of transverse cracks, that began within a matter of days after construction. The cracks are extremely erratic and follow no particular pattern. Some cracks extend across the full land width while others originate at the edge and then terminate in the middle of the lane. Still others originate at the edge and then branch into two cracks which may or may not extend the full lane width. A fourth type originate and terminate within a lane and do not extend to either edge. For these reasons it was not deemed possible to establish a true crack interval and crack width.

6) The findings in this report established a crack interval which was an average interval with the difficulties fully kept in mind.

TABLE 11

AVERAGE CRACK INTERVAL IN CENTRAL PORTION OF SLAB-NEW JERSEY-1950

	Crack Interval (ft.) 8" Pavement		10" Pavement	
Date	Inside Lane	Outside Lane	Inside Lane	Outside Lane
Nov. 1947	7.7	6.6	10.7	6.2
Oct. 1950	3.9	3.5	6.2	4.0

7) The total number of cracks has almost doubled in 3 years of service.

8) The crack interval in the outside heavily traveled lanes is smaller than the lightly traveled inside lanes.

9) The crack spacing is greater in the thicker pavement.

10) The cracks in the central portions of the slab are not uniformly spaced ranging from 6 to 20 feet. There are, however, only six locations in the central section of the pavement in which the crack interval exceeds 12 feet.

11) Most of the cracks are variable in width.

12) The maximum measured crack width in the 8-inch pavement was 0.013 inches in the inside lane and 0.019 inches in the outside lane. Under the same conditions the average crack width was 0.011 and 0.015 inches respectively. The temperature differential was 40° F.

13) The maximum measured crack width in the 10-inch inside lane was 0.024 inches for a 50° F temperature change. The corresponding average crack interval was 0.014 inches.

14) The maximum measured crack width in the 10-inch outside lanes was 0.030 inches for a 79° F temperature change. The corresponding average crack interval was 0.021.

15) The crack width has increased with age and service. In general the crack width has remained practically constant between January and October, 1950.

TABLE 12

CRACK WIDTH IN CENTRAL PORTIONS OF SLAB-NEW JERSEY-1950

Date	Temp.	Crack Width (in.) Max.	Crack Width (in.) Average
a) 8" Pavement-Inside Lane			
Oct. 15, 1947	70	0	0
Nov. 14, 1947	51	0.010	0.008
Feb. 24, 1948	45	0.008	0.007
Jan. 19, 1950	35	0.012	0.010
Jan. 20, 1950	30	0.013	0.011
Feb. 27, 1950	36	0.013	0.011
Oct. 3, 1950	84	0.012	0.010
Nov. 6, 1950	75	0.011	0.010
b) 8" Pavement-Outside Lane			
Oct. 7, 1947	75	0	0
Nov. 14, 1947	51	0.011	0.010
Feb. 24, 1948	45	0.010	0.008
Jan, 19, 1950	35	0.018	0.014

CRACK WIDTH IN CENTRAL PORTIONS OF SLAB-NEW JERSEY-1950 (cont.)

Date	Temp.	Crack Width (in.) Max.	Average
Oct. 4, 1950	69	0.018	0.014
Nov. 14, 1950	56	0.019	0.014
Nov. 30, 1950	44	0.019	0.015
	c) 10" Pavement-Inside Lane		
Sept. 26, 1947	76	0.	0
Nov. 13, 1947	52	0.012	0.008
Jan. 15, 1948	30	0.016	0.012
Feb. 10, 1948	26	0.022	0.011
Feb. 24, 1948	46	0.011	0.006
Jan. 20, 1950	30	0.024	0.014
Oct. 3, 1950	80	0.016	0.011
Dec. 1, 1950	45	0.023	0.013
	d) 10" Pavement-Outside Lane		
Sept. 11, 1947	105	0	0
Nov. 13, 1947	52	0.020	0.015
Feb. 10, 1948	26	0.028	0.016
Feb. 24, 1948	46	0.019	0.012
Jan. 19, 1950	36	0.029	0.020
Feb. 27, 1950	31	0.030	0.021
Oct. 3, 1950	80	0.026	0.015
Oct. 18, 1950	70	0.025	0.016

16) A comparison was made of the cracked and uncracked portions of the slab. On July 31, 1950 at a temperature of 95° F. the average 10-inch gage length had increased 0.0051 inches in the cracked section, while the same gage length had decreased 0.0028 inches in the uncracked section. A similar measurement was made on Dec. 1, 1950 with a slab temperature at 46° F. At this time the 10-inch gage length had increased 0.0013 inches in the cracked section, and decreased 0.0011 inches in the uncracked section. Based on these measurements it was determined that adding the change in gage length, an average measure increase in crack width of 0.0013 inches is actually an increase in 0.0024 inches.

17) There was no apparent failure of the reinforcement at any of the cracks.

18) The lapping of reinforcement has had no apparent effect on the location and/or crack width.

19) In general, there is no major deterioration at the cracks.

20) Minor spalling has occurred in the heavily traveled outside lane.

21) Many of the cracks divide into two cracks and form a small cracked island. It is feared that these islands will shatter and dislodge causing the necessity of patching maintenance.

22) There is no pumping at the cracks, although water has been observed to rise from the transverse cracks following a rain.

23) There has been minor spalling at the construction joints.

24) There has been no apparent faulting of construction joints.

25) The maximum construction joint opening is 0.041 inches.

26) Serious cracking has occurred on the new side of the construction joints for a distance of 6 to 10 feet.

27) A failure occurred at a section in the outside lane of the 10-inch section, approximately 1425 feet from one end. This failure started with heavy cracking which became serious in the spring of 1949. By September,1949, the failure had advanced to a point where the upper 3 inches of concrete had become shattered and had broken loose. This was cold patched. By December, 1950, the faulty area increased in size to 8 feet square and the concrete underlying the patch appeared to be shattered into squares of 2 feet or less. This area shows pumping of the base course fines and large deflections under traffic.

28) Changes in pavement elevation have been limited to less than 1/4 inch for the most part. A few localized areas have shown settlements of 5/8 inch over a 50 foot area.

29) The supplementary 187 foot slabs have remained relatively uncracked, with 8 of the 16 slabs being uncracked and only two slabs having 5 cracks which was maximum. These cracks are not located in any portions of the slabs, but are spread throughout. All the cracks are narrow and free of ravelling.

Based on the above observations, certain interpretations and conclusions were drawn by the author of the report. These interpretations and conclusions are as follows:

1) For a 500 foot slab the central 3500 feet is of constant length at all times.

2) The overall length changes are much less than for an uncracked free slab. Based on the New Jersey 100 foot slabs the extrapolated length change would be 26 inches which is far more than actually occurred.

3) Excluding tensile stresses due to shrinkage, the tensile stresses in the central portion of the slab is primarily a function of the temperature differential between as constructed temperature and the temperature in question.

4) The erratic behavior observed may be partially explained by the differences in the construction temperature.

5) In the 3 years of service under consideration, only one winter may be described as severe with a minimum recorded pavement temperature of 25°F. The other two winters had pavement temperatures averaging well over 32° F.

6) The severe cracking on the new side of the construction joints is due to the older, stronger concrete pulling the weaker, newer concrete apart.

7) The observed pavement failure was probably due to concrete of inferior quality.

8) The riding qualities, with the exception of a few "dips" appears excellent.

9) The crack openings appear in many cases to be excessive and indications that the pavement will not have long term serviceability.

10) The crack ravelling in the outside lanes is excessive.

11) The crack pattern is such as to encourage ravelling.

12) The use of extended steel in forming a construction joint is questioned in view of the serious cracking on the new side.

13) The failure of a section indicates that the lack of quality concrete is likely to have serious consequences.

14) The condition of the outside lanes, tend to indicate that under the heavy volume of truck traffic that exists, the pavement is not capable of satisfactorily carrying the load on a long term basis.

15) The more heavily reinforced 8-inch section appears to be in better condition than the 10-inch section with lighter reinforcement.

16) Cold weather appears to be the factor which is predominate in destroying a continuously reinforced pavement.

17) It is questioned whether a continuously reinforced pavement has structural strength equivilant to a conventional pavement. Under the same conditions in New Jersey conventional pavements have performed excellently with similar riding qualities.

The pavement was resurveyed in June 1952, at which time crack spacing and the condition of the pavement was noted.(6.8)

TABLE 13

CRACK INTERVAL AS OF JUNE 1952-NEW JERSEY

Distance from End (ft.)	Crack Interval (ft.) Outside Lane Range	Outside Lane Average	Inside Lane Range	Inside Lane Average
a) 8-inch Pavement				
0	30		32	
150	5-7	5.5	6-9	6.5
300	2-7	2.5	3-8	3.9
Quarter pt.	2-6	3.0	3-7	3.5
Center	2-7	3.0	3-7	4.0
Quarter pt.	2-6	3.0	3-8	4.0
300	3-6	3.5	5-9	6.0
0	105		117	
b) 10-inch Pavement				
0	60		60	
200	3-12	10.0	6-15	12.5
400	3-8	5.5	5-15	8.3
Quarter pt.	1-6	4.5	3-9	6.0
Center	1-6	4.8	3-9	6.1
Quarter pt.	2-7	5.0	4-10	6.1
300	4-7	5.5	5-9	7.5
0	90		90	

The 1952 servey was summarized as follows:

1) After 5 years of concentrated traffic with the exception of three small localized areas of disintegration there is no evidence of more than nominal crack deterioration in the tire tracks.

2) A moderate number of cracks appear wider than previous research pavements indicated.

3) All cracks appear structurally sound and are closely spaced thus minimizing future temperature stresses or movement.

4) There is no visible elevation change and at all speeds the riding is agreeably smooth.

5. RESEARCH IN CALIFORNIA

In May and June 1949 a section of continuously reinforced concrete pavement was constructed in California. The highway used was one in which all types of traffic acrued an average daily traffic of 12,000 vehicles of all types.

The project consisted of a four lane divided highway with two 12 foot lanes 8 inches thick tied together by 5/8 inch diameter tie bolts 30 inches long spaced 30 inches on centers. The pavement lanes were joined by a tongue and groove joint, and tie bars.

The subgrade conditions were an adobe of variable composition. The liquid limit varied between 36 and 43 and the plasticity index ranged between 16 and 23. The CBR varied between 2 and 20 with 1.7 to 10.8 per cent swell. The base course consisted of a 10-inch layer of gray volcanic ash (tuff). The tuff had an average optimum dry density of 65 pcf at 25 to 30% moisture content and a CBR ranging from 150 to 200. In order to attach the base course to the subgrade the top 4 inches of subgrade was treated with 3.5% of cement to prevent washing, shifting, pumping, and step-offs.

The structural properties of the concrete are presented in Table 14.

TABLE 14

STRUCTURAL PROPERTIES OF CONCRETE-CALIFORNIA PROJECT

Age	Traffic Lane	Passing Lane
	a) Compressive Tests	
3 days	2,180	1,525
10 days	3,367	2,835
28 days	4,397	4,138
1 year	6,105	5,135
	b) Tension Tests	
1/2 day	105	41
1 day	136	72
2 days	184	123
3 days	215	155
10 days	295	255
28 days	327	284

The test section was one mile long, and joined the standard pavement by means of 4-inch unrestrained expansion joints.

The longitudinal reinforcements consisted of two types. The first type was a billet steel bar with an average elastic limit of 54,090 psi and an average ultimate strength of 84,120 psi. 1/2 mile of pavement was reinforced with #4 bars of this type spaced 4 inches on centers resulting in 0.62% of steel. The second type of steel had an average elastic limit of 71,935 psi, and an ultimate strength of 121,250 psi. This type was laid for 1/2 mile and consisted of #4 bars spaced 5 inches on center, providing 0.5% of steel. All bars had the old type of deformations. The steel was placed in 30 foot lengths at pavement mid-depth. The splices were staggered and lapped 40 diameters (20 inches). The longitudinal steel was continuously carried through the construction joints and through the transition between the sections of differing reinforcements. One construction joint was continuous across both lanes and one joint crossed the outside lane only, making 3 joints across the 12 foot lanes in the test section.

The pavement was cured by a light gray pigmented solution using specifications similar to those used by the Bureau of Reclamation.

The paving was started on May 23, 1949 and finished on June 8, 1949. The temperature varied between 83 and 100^{o} F.

The test instrumentation was installed as follows:

1) Longitudinal movements of the pavement were recorded by means of reference plugs set in the pavement at the ends; 100,200,300, 400, 600 and 800 feet from the ends and 100 feet on either side of the center. These plugs were placed on both pavements. The control points were rods driven 2 feet into the subgrade housed in a steel box. Measurements were made with calipers and micrometers to an accuracy of 0.01 inches.

2) Brass plugs were installed on either side of the expansion joint and across several performed cracks, with the intention of measuring their opening. Measurements were made with a micrometer to an accuracy of 0.002 inches..

3) SR-4 gages were installed on selected reinforcing bars at two locations in each section in each lane, making a total of 8 groups. The gages were attached to the bars after

the loose scale and deformations had been removed. Once the active gages were attached to the bar, the gage was calibrated by stressing the bar by increments to just below the yield point. This procedure was followed for all gaged bars. Three types of strain gage panels were installed. The first consisted of a system of 24 gages in both lanes placed in pairs on 12 bars at a formed weakened plane joint. In addition 5 lines of 6 gages apiece were placed 30 inches apart away from the joint one gage per bar also spread across both lanes. The second type of panel consisted of five weakened plane joints spaced 5 feet apart. The center joint was gaged with 24 gages placed in pairs on 12 bars while two other joints on one side of the center were each gaged with 6 gages, one per bar. Halfway between the gaged joints 6 SR-4 gages were attached to the steel, one per bar. The third type of panel had a single preformed crack with ten gages placed in pairs on 5 bars and two lines of gages away from the crack at 5 feet and 10 feet, each having 6 gages, one per bar.

4) Iron constantan thermocouples were installed, one inch up from the bottom of the pavement, on the reinforcing steel at the center of the pavement and one inch down from the top of the slab. These gages were installed in various locations in each test panel.

5) In addition to the above, a crack survey, roughness measurement and a condition survey were periodically performed on the pavement.

A survey and report after 1 1/2 years of service reported the following observations.[6.12]

1) During the first 6 months with a low temperature of 42° F. the maximum longitudinal movement varied between 0.22 and 0.29 inches of contraction at the ends of the slab. At the end of one year the pavement had lengthened on a high of 101° F. with a maximum end movement ranging from 0.35 to 0.39 inches. The movement decreased within 250 feet of each end to a maximum of 0.03 inches that prevailed throughout the center portion without respect to season or type of reinforcements.

2) At an early age prior to the formation of the majority of the intermediate shrinkage cracks the maximum measured opening occurred at the interior construction joint and was 0.04 inches. The maximum opening of the preformed crack at the same age was 0.017 inches. The crack widths of all of the early formed cracks decreased with further intermediate cracking even with a 40° drop on temperature.

3) The average readings taken by use of the SR-4 gage is presented in Table 15.

TABLE 15

AVERAGE STEEL STRESS-CALIFORNIA PROJECT

(months) Age	Temp. °F	Stress (psi) At Crack	Between Crack
	a) 0.62% Section		
0	96	0	0
1	66	30,000	4,000
2	72	31,000	4,000
3	100	10,000	5,000
6	40	50,000	21,000
7	35	51,000	19,000
13	101	44,000	35,000
	b) 0.5% Section		
0	100	0	0
1	67	28,000	4,000
2	72	38,000	5,000
3	78	37,000	32,000
6	42	82,000	45,000
7	38	83,000	46,000
13	101	80,000	60,000

The readings taken at 13 months were very high and deemed to be unreasonable as the calculated stress based on calibrations below the elastic limit was frequently much above the ultimate strength of the bars. At ages less than 3 months the readings of pairs of SR-4 gages opposite each other on a single bar were in agreement within 10%. By 6 months of age these pairs varied by as much as 100%. Coupling this fact with the drop in insulation resistance of the gage, it was stated that gage readings after 3 months were unreliable and indicated stresses far in excess of the actual.

4) By the end of one year, the anticipated numerous shrinkage cracks had developed and except for 140 feet at each end the average crack interval was 4.5 feet. Fifteen feet on

the new side of the construction joints showed a much closer crack spacing. Differences in the crack spacing for different steel percentages was minor and varied from 4 feet, 200 to 500 feet from the end to 4.7 feet in the central portion. There was a slight increase in spacing of cracks in the 400 feet interval where the two sections join. The average crack interval stabilized at the end of seven days and was practically constant from that time until the end of one year. At the end of one year the greatest average crack interval was in the passing lane of the 0.5% section this being 5.5 feet. The traffic lane of the 0.62% section had an interval of 4.2 feet while the interval in the passing lane was 4.5 feet.

5) Roughness measurements taken with a profilograph at the age of one year indicated that the roughness of the continuous pavement was 1/4 the roughness of the adjacent conventional pavement.

Based on the above observed results, the reporter drew certain conclusions which are:

1) The cracks which have a high incidence are not at the end of one year very wide. However, a few cracks have developed objectionable characteristics and it was feared that others would follow.

2) It was recommended that the continuity of the reinforcement be broken at intervals of 100 to 200 feet to avoid numerous shrinkage cracks.

3) The effect of the time of day and/or order of construction was apparent from the crack development at construction joints. Even though the reinforcing steel is carried through the joint continuously, the crack incidence is much less at the end of a days pour, just before reaching the joint, than during the first hour of the succeeding day when the incidence is somewhat greater than the average.

4) Up to 1 1/2 years the crack incidence was not materially affected by difference in the steel percentage.

5) Up to 1 1/2 years the crack development was not appreciably affected by differences in concrete strength.

A detailed field survey was made of the California pavement in July, 1952, after three years of concentrated heavy traffic. The reported findings of this survey are as follows[6.8]:

1) There was no visible change in pavement elevation.

2) None of the approximately 1200 cracks in the mile long pavement showed any sign of distress or progressive deterioration other than normal slight edge wear in the wheel tracks in the outer traffic lane.

3) There is no significant difference in crack spacing or crack interval between the 0.5 and 0.62% of steel.

4) The crack spacing is recorded in Table 16.

TABLE 16

CRACK SPACING AFTER THREE YEARS-CALIFORNIA PROJECT

	Outside Lane		(Spacing--ft.)		Inside Lane	
Area	Max.	Min.	Aver.	Max.	Min.	Aver.
a) 0.62% Steel						
300'-400' from End	10	1.5	3.6	10	1.5	3.7
Center	7	1.5	4.0	8	2.5	4.4
Junction	8	2.0	4.4	8	3.0	5.0
b) 0.5% Steel						
Junction	8	1.5	4.8	7	4.0	5.9
Center	10	1.5	5.3	12	3.0	7.1
300'-400- from End	7	1.5	4.3	9	4.0	4.3

5) The two construction joints across the 12 foot outside heavily traveled lanes exhibit a crack incidence of 14 to 18 inches in the first 20 feet of concrete laid the second day.

6) There is no surface evidence of transition in terms of crack pattern from one section to another.

7) The outer traffic lane in the 0.62% section contains 8% more cracks than the inner lane. The outer lane, 0.5% section contains 16% more cracks than the inner lane.

The conclusions drawn from this survey are as follows:

1) The continuous reinforcement has resulted in the anticipated closely spaced transverse cracks which continue to be held so tightly closed as to present no structural importance.

2) Neither the high strength steel at 0.5% or the milder steel at 0.62% of reinforcement have given evidence of stresses exceeding the elastic limit.

3) The riding surface is excellent and there is no indication that any of the many tight cracks may become a deterioration hazard.

6. TEXAS PAVEMENT

In 1951 a section of the Fort Worth Freeway was constructed using continuously reinforced concrete. This highway was unique in that it was not a research project but was a preferred means of construction to serve heavy traffic with a minimum of interruption for maintenance and repair. The construction occurred over undulating surface topography principally in cut which in some cases was of considerable depth. Moderator fills were required in some areas. The soil is a silty clay which has shown susceptibility to pumping the older pavements. The pavement rested on a compacted granular base course.

The pavement was uniformly 8 inches thick and was reinforced with rail steel deformed #6 bars spaced 8 inches on center, providing 0.7% of longitudinal reinforcing. The concrete was designed for a 28-day compressive strength on 5,000 psi.

The highway consisted of 2 twenty-four foot roadways separated by a curbed median strip. Expansion joints were provided in the slabs at approaches to structures. The result was two slabs 619 feet long and 15 additional slabs ranging from 1961 to 8263 feet in length.

Construction was completed in September 1951. The construction temperature ranged between 90° and 100° F.

In July 1952 a survey was made of the pavement after approximately one year of service. The results of this survey are as follows:[6.8]:

1) The first crack appears at some distance from the expansion joint.

2) The crack interval decreases rapidly with distance from the joints until at 200 to 300 feet, the spacing ranges from 2 to 21 feet and averages 4.5 feet. The quarter point spacing ranged 3 to 12 feet and averaged 5 feet. The spacing near slab ends ranged from 3 to 11 feet and averaged 6 feet.

3) The crack width was approximately 0.007 inches.

4) There was very little edge wear or chipping of cracks and even that which has occurred is inconsequential.

5) All examined construction joints were sound, tight and exerted no influence on the spacing of adjacent transverse cracks.

6) There were visible elevation changes.

As a consequence of the 1952 survey the following conclusions were drawn.

1) High strength reinforcement with good bond properties will cause many closely spaced transverse cracks early in the pavement life.

2) Cracks are minute, have no structural significance and require no maintenance or attention.

3) Daily and seasonal variations of moisture are of no consequence to the structural stability of the pavement and the cracks are no indications of deterioration.

4) The continuously reinforced sections are impressively smooth riding.

7. SUMMARY

Based on the previous research papers dated up to 1952 a complete summary of the significant features of research in continuously reinforced concrete pavements was made. The results of this study are(6.8):

1) Reinforced slabs longer than 600 feet will have a central portion in which numerous fine, tightly closed transverse cracks will appear during the curing period. This central portion will exclude the end 150 to 250 feet of the pavement slab.

2) In the end portions of the slab, the first crack will occur at some distance from the end with rapidly decreasing spacing between subsequent cracks until a somewhat uniform spacing will occur around 250 feet from the ends.

3) The number of cracks will increase due to temperature change and possible traffic until the spacing will stabilize after about five years. After five years new cracks will appear only occasionally between the wider spaced cracks.

4) The percentage of longitudinal reinforcement is a factor of crack spacing in the restrained portion. The spacing is closer for the heavier percentages. A summary of crack spacing as of 1952 for all projects is presented in Table 17.

TABLE 17

CRACK SPACING - 1952

Project	% Reinf.	Slab Thickness (in)		Crack Spacing (ft.) Min.	Max.	Aver.
Indiana	1.82	9-7-9	(outer)	1.5	12.0	3.8
Indiana	1.82	9-7-9	(inner)	2.0	12.0	4.3
Indiana	1.02	9-7-9	(outer)	1.5	11.0	4.9
Indiana	1.02	9-7-9	(inner)	2.5	11.0	5.4
Illinois	1.00	8		2.0	18.0	7.8
Illinois	1.00	7		2.0	9.0	5.0
New Jersey	0.90	8	(outer)	2.0	7.0	2.9
New Jersey	0.90	8	(inner)	3.0	8.0	3.7
New Jersey	0.72	10	(outer)	1.0	8.0	4.9
New Jersey	0.72	10	(inner)	3.0	15.0	6.1
Illinois	0.70	8		2.0	20.0	6.8
Illinois	0.70	7		3.0	9.0	4.0
Texas	0.70	8		3.0	9.0	4.0
California	0.62	8	(outer)	1.5	11.0	4.0
California	0.62	8	(inner)	1.5	10.0	4.4
California	0.50	8	(outer)	1.5	10.0	4.6
California	0.50	8	(inner)	3.0	12.0	5.8
Illinois	0.50	8		2.0	15.0	6.3
Illinois	0.50	7		3.0	15.0	7.3
Indiana	0.45	9-7-9	(outer)	3.0	12.0	9.5
Indiana	0.45	9-7-9	(inner)	5.0	12.0	10.0
Illinois	0.30	8		4.0	15.0	11.4
Illinois	0.30	7		2.0	18.0	10.6

5) In pavements longer than 600 feet the crack spacing and crack width decrease in direct proportion to the increase in total tensile strength of steel in the pavement cross-section. Other physical factors being equal, the greater the strength of the steel the closer the transverse cracks and the tighter they are held. Different quantities of the same grade or differing grades in the same quantity have also shown this effect.

6) All of the thousands of transverse cracks examined up to 1952 were held tightly closed and structurally sound. Since this survey it has been reported that steel failures for lightly reinforced sections has occurred and the cracks have opened up.

7) In most projects the crack pattern is similar in terms of their characteristics such as width vs. location, behavior under seasonal and daily temperature changes, and behavior under traffic.

8) The transverse expansion and construction joints are a potential source of weakness.

Based on a thorough study of all projects, a group of behavior assumptions and recommendations for application were submitted.

1) Where sufficient reinforcing steel is provided the cracks should be structurally sound and require no maintenance.

2) A thin compacted, free-draining granular base course will give the pavement added support and control pumping.

3) The critical construction joint situation may be relieved by additional reinforcement via bonded dowels at the daily construction joints.

4) With proper protection of exposed longitudinal steel, continuous reinforcement may be joined effectively in subsequent pavement at any future date.

5) Frequent transverse cracks as close as 2 feet with an average of possible 40 to 48 inches will provide the most stable crack width condition. A 6 foot crack spacing should produce a maximum crack width not exceeding 0.04 inches. Service conditions have indicated this width is an upper bound due to the influence of reinforcement.

6) The research has favored the use of 0.5 to 0.7% of longitudinal steel in a seven or 8 inch uniform slab depending on subgrade and traffic conditions.

7) Where there is a combination of frequent, heavy trucks and silty clay subgrade with a possibility of subsurface water accumulation, 3 inches of compacted free-draining granular base course under 8 inch concrete reinforced with 0.7% longitudinal steel should assure sound continuous service and withstand repeated loads in excess of current legal loads common in most states.

8) A fabricated longitudinal joint is recommended.

9) Recommended transverse steel consists of #4 bars spaced 24 inches on centers.

10) The highest strength commercial grade of reinforcing steel is preferable.

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