of 0.005 in., the maximum opening being 0.009 in.

The crack interval was very erratic and ranged from approximately 1 ft. to 58 ft. The greatest concentration of cracks was in the portions of the pavement constructed earliest in the morning. In these locations there were areas from 200 to 300 ft. in length where the cracks averaged from 3 to 4 ft. apart.

Careful examination disclosed no cracks in the 187-ft. slabs. The opening that had occurred at the expansion joints between these slabs ranged from a minimum of 0.30 in. to a maximum of 0.60 in.

ACKNOWLEDGMENTS

The author wishes to acknowledge the very helpful assistance of Messrs. H. W. Giffin, H. E. Phillips, and R. H. Stelljes of the New Jersey State Highway Department in the preparation of this report.

AN EXPERIMENTAL CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN ILLINOIS

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SYNOPSIS

In the fall of 1947, the Illinois Division of Highways started construction of an experimental pavement consisting of eight continuously reinforced test sections, ranging in length from 3500 to 4230 ft., each section separated from adjacent ones by an expansion joint. Four of the sections are uniformly 7 in. thick and four are uniformly 8 in. thick.

One section of each thickness is reinforced longitudinally with 0.3, 0.5, 0.7, and 1.0 percent of steel, respectively. Round deformed rail-steel bars were used for longitudinal reinforcement. The transverse reinforcement consists of $\frac{3}{8}$-in. round deformed, intermediate grade, billet-steel bars at 12-in. centers in one half of each section and at 18-in. centers in the other. The pavement contains no center joint, the transverse reinforcing bars being continuous across the pavement.

The pavement was built directly on the natural subgrade, as graded, about 90 percent of which is composed of potentially "pumping" soils. A normal strength concrete with a cement factor of 1.4 bbl. per cu. yd. and a slump of $1\frac{1}{4}$ to $1\frac{1}{2}$ in. was used. The reinforcing bars were assembled on the subgrade into a continuous mat supported 3 in below the pavement surface. Concrete was placed in one lift and distributed and consolidated by a spreader and a vibratory finishing machine.

Provisions were made for measuring horizontal and vertical movements at various locations along the pavement; also for measuring the width of transverse cracks that develop. Unit strains in the longitudinal reinforcing bars are measured by SR-4 strain gages. Temperatures of the concrete and subgrade are measured with thermocouples.

Already numerous fine transverse cracks have developed, as expected, in the completed test sections. Unit tensile stresses of almost 40,000 lb. per sq. in. have been measured in the longitudinal reinforcement in the 7 in. section with 0.7 percent steel. The yield point of the steel is approximately 70,000 lb. per sq. in.

One of the major problems in highway engineering is that of improving pavements structurally so that they will meet more adequately the demand of present day and anticipated future traffic and provide longer serviceable life. The problem is made more difficult because heavy truck and bus traffic is constantly increasing in volume and weight.

In their efforts to improve pavements, highway engineers have increased pavement thickness, improved the subgrade and drainage, introduced joints at close intervals to reduce warping stresses, and adopted the use of steel reinforcement.

In Illinois at the present time the standard concrete pavement for trunk highways is 10 in. thick, laid on 6 in. of granular subgrade replacement. The pavement is divided by means of dowelled contraction joints into panels 100 ft. long, which are reinforced with one layer of 78 lb. wire fabric reinforcement placed 2½ in. below the surface of the pavement.

While this design is giving excellent service, highway engineers in this State, like those in others, are conscious of the necessity of continually improving pavement design to meet the increasing demands of traffic and to increase the service life of pavements.

INVESTIGATION UNDERTAKEN BY ILLINOIS

With these objectives in mind the Illinois Division of Highways early in 1946 decided to conduct an investigation of continuously reinforced pavement embodying the design features suggested by Mr. W. R. Woolley, Materials Engineer, Division 4, Public Roads Administration, in the paper which he has presented at this meeting (see p. 28). Mr. Woolley has covered adequately in his paper the experience background for this design. No further discussion of this phase is necessary except to emphasize that none of the previous continuously reinforced pavement experiments were of sufficient scope to develop definite conclusions as to the influence of slab dimensions on performance or to establish the amount of reinforcement required.

The investigation was undertaken with the approval and cooperation of the Public Roads Administration. It was decided that the investigation should include the construction of three experimental pavements to be located on highways in different parts of the State where the traffic would be extremely heavy and soil conditions favorable for pumping action.

Plans for two of these pavements were made simultaneously for the purpose of so coordinating them that the greatest possible scope would be developed in the investigation. Plans for the third will be made after the other two are built and significant developments occur.

It was planned to construct the first project on U. S. Route 66 just south of Springfield and, because of its close proximity to Bureau headquarters, its scope was made broader and more extensive than that of the second planned project. However, owing to difficulties encountered in securing additional right of way, letting of the contract for the Springfield project has been indefinitely delayed. In the meantime the contract for the second project was awarded and construction started this fall (1947). Time did not permit any significant changes to be made in the plans for this project, although some may have been desirable because of the change in order of construction of the two projects.

FIRST EXPERIMENTAL PAVEMENT CONSTRUCTED

The experimental pavement described in this paper is officially identified as F. A. Route 12, Section O-2, Fayette County. It is located just west of Vandalia and will become a part of U. S. Route 40, which carries a large volume of heavy truck traffic. It is estimated from origin and destination studies made in the vicinity of the project that the new pavement when opened to traffic will carry a total of 2100 vehicles per 24 hr., consisting of 1600 passenger cars and 500 commercial vehicles.

The experimental pavement is 22 ft. wide and 29,493 ft. or about 5½ miles long, and is divided into eight test sections, six of which are approximately 3500 ft. long and two about 4230 ft. The pavement contains no transverse joints except those separating the test sections and construction joints at the end of each day's run. At the construction joints the reinforcing steel was carried across the joint so that in effect the construction is continuous from end to end of each test section. The transverse reinforcing bars extended the full width of the pavement, creating continuous reinforcement in that direction also, and the customary center joint was omitted. The variables between test sections are thickness of pavement and amount of longitudinal reinforcing steel.

Four of the sections are uniformly 7 in. thick and four have a uniform thickness of 8 in. These thicknesses, chosen somewhat ar-

bitrarily, were though most likely to give the desired performance within economic limits.

The construction contract under which the pavement was built also included about 4 miles of standard 10-in. pavement, 22 ft. wide, with 78-lb. wire fabric reinforcement and contraction joints at intervals of 100 ft., thus affording a basis for direct comparison with the experimental sections.

AMOUNT AND DISTRIBUTION OF REINFORCING STEEL

Four percentages of longitudinal steel, based upon the cross sectional area of the pavement, were used with each thickness of pavement; namely, 0.3, 0.5, 0.7, and 1.0 percent. These percentages, it is believed, provide sufficient range to establish the proper amount of reinforcement required to induce closely spaced transverse cracks and to hold them tightly closed.

The full theoretical amount of steel required is about one percent. This is based upon the assumption that pavements, in their daily and seasonal movements, actually slide on the subgrade, and gives consideration to thermal stresses in the steel. However, there are considerable experimental data to indicate that actual sliding does not occur but that the subgrade moves elastically with the slab and actually offers much less resistance than is generally assumed. Available information on continuously reinforced slabs indicate that 0.5 percent of longitudinal steel is adequate. The percentages of steel included in this pavement, therefore, range from the full theoretical amount to an amount which may be somewhat inadequate.

The various percentages of longitudinal steel were obtained by using bars of different diameters and variable spacing. The bar sizes used were $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ in. Spacings varied from $4\frac{9}{16}$ to $6\frac{1}{4}$ in. Bar sizes were selected that would give maximum distribution of steel and spacings wide enough to prevent interference with the placing of concrete. The minimum free opening between bars, except at laps, is $4\frac{1}{4}$ in.; the maximum is $5\frac{5}{8}$ in.

Each section is divided into two subsections of approximately equal length in which the variable is the spacing of the $\frac{3}{8}$-in. transverse reinforcing bars. In one subsection the transverse bars are placed 12 in. center to center; in the other the spacing is 18 in. It is believed that the smaller amount of transverse steel will provide sufficient cross sectional area to maintain the tensile stress within safe limits, but there is a possibility that it may not provide the necessary bond strength and may eventually result in an undesirable opening of longitudinal cracks. The performance of the two amounts of steel may reveal whether this is true.

TYPE OF REINFORCING STEEL

The decision to use reinforcing bars assembled into mats on the subgrade was made because it appeared that this method of fabrication would provide the flexibility required by the variations in pavement thickness and percentage of longitudinal steel. Other types of reinforcement no doubt would be equally effective and probably could be installed at least as efficiently on a normal project where only one pavement thickness and one percentage of steel are involved.

TABLE 1
RESULTS OF MILL TESTS OF REINFORCING STEEL

Bar Size	No. of Tests	Yield Point	Tensile Strength	Elongation
Longitudinal Reinforcement				
in.		*psi.*	*psi.*	%
$\frac{3}{8}$	6	78,701	119,469	14.3
$\frac{1}{2}$	14	66,223	107,355	14.5
$\frac{5}{8}$	17	70,778	127,759	12.5
$\frac{3}{4}$	21	70,157	124,888	11.7
Transverse Reinforcement				
$\frac{3}{8}$	7	43,498	77,356	21.5

Reinforcing bars meeting the requirements of ASTM Designation A-16 for rail-steel bars were used for the longitudinal reinforcement because it is believed the higher tensile strength and yield point and the lower ductility of this steel will give better performance than lower strength, higher ductility steels. The special provisions permitted the use of other grades having similar properties, but rail-steel was furnished. The physical properties of the longitudinal reinforcing steel, as represented by mill tests, are given in Table 1.

The use of bars meeting the requirements of ASTM Designation A-15 for intermediate

grade, billet-steel bars was permitted for transverse reinforcement because supporting chairs could be welded to bars of this grade more effectively than to rail-steel bars. The physical properties of this steel also are given in Table 1.

All bars met the requirements of ASTM Designation 305-47T for the deformations of deformed steel bars for concrete reinforcement.

PAVEMENT BUILT ON "PUMPING" SOILS

Grading for the section was completed late in 1943, and the road was used by local traffic until construction was started on the paving contract in 1947. At the time the road was graded it was not known that an experimental pavement would be built and no special soils studies were made. Nevertheless, soils studies made prior to and during grading operations furnish adequate information as to the character of the soils in the sub-base and subgrade. During June and July 1946, a detailed soils survey was made on the graded section. Analysis of borings taken at close intervals along the section has identified the soils in the subgrade and established the location of transitions from one type to another within reasonably close limits.

Since one of the basic arguments for the use of continuous reinforcement is that it will eliminate pumping, the experimental pavement was constructed directly over the natural soil as graded, without granular replacement or special subgrade treatment of any kind. The subgrade is composed of soils of the following groups in the amounts given:

	percent
Silty clay and clay of the A-7-4 group	35
Silty clay loam, silty clay, and clay loam of the A-4 group	33
Clay loam of the A-4-2 group	11
Sandy loam of the A-2 group	11
Clay of the A-6 group	10

It is known that soils of the A-4, A-6, and A-7 groups will pump when water is present and heavy vehicles in sufficient number travel over a pavement built on them. Hence, at least 89 percent of the experimental pavement is built over potentially pumping soil.

CONSTRUCTION METHODS

The experimental pavement was constructed as a part of a regular contract section which also included, at either end, a considerable amount of standard uniform 10-in. pavement. Construction on the experimental portion was carried on in the orthodox manner with only such slight variations as were necessary to fit special features of the work. Conventional paving equipment employed by the contractor consisted of a form grader, form tamper, mechanical subgrader, 34-E dual drum paver, screw type mechanical spreader, finishing machine, and mechanical longitudinal float. The finishing machine was equipped with a vibrating front screed to improve consolidation of the concrete around the reinforcing steel.

Air entraining cement was used. The aggregates used were a siliceous sand containing some limestone, and a crushed limestone coarse aggregate. The concrete mixture was designed to give the strength normally obtained on paving jobs in Illinois. It was assumed that the increase in strength to be expected from vibrated concrete would be offset by the lower strength caused by the increased air content. Hence only minor changes were made in the concrete proportions when going from the standard to the experimental pavement.

The proportions per bag of cement used for the most part were 215 lb. of sand, 359 lb. of stone, and 5.25 gal. of water, which gave a cement factor of 1.4 bbl. per cu. yd. The slump varied from 1¼ to 1½ in.

Tests, using the pressure method, were made daily to determine the air content of the concrete. The air content varied considerably, ranging from a minimum of 2.1 percent to a maximum of 6.5 percent. A limited number of tests indicated some reduction in air content caused by spreading, vibrating and finishing, but the loss was small. There was also an indication that vibration caused the air to move upward through the concrete, a slightly higher air content being found in the concrete above the reinforcement than in that below it.

INSTALLATION OF REINFORCING STEEL

The longitudinal reinforcing bars were 30 ft. long; the transverse bars were 21 ft. 9 in.

long. These were assembled on the subgrade into a continuous mat supported 3 in. below the surface of the pavement by means of chairs welded to certain of the transverse bars. Since the paver was operated outside the forms, as required in Illinois, it was possible to lay out the reinforcement a sufficient distance ahead of the paver to prevent delays from this source.

Transverse bars equipped with chairs were placed on the subgrade at intervals of 3 ft. The chairs were spaced on the bars, by the producer, in multiples of the longitudinal bar spacing and arranged so that the legs of the chairs extended above the transverse bars and served as positioning stops for the longitudinal bars. This arrangement helped materially in spacing the longitudinal steel and reduced the time required to set it.

The transverse bars were placed on the subgrade first, and those with chairs positioned by means of templates. Workmen then placed bundles of longitudinal bars on the transverse bars. Other workmen set the longitudinal bars in place and tied them to the transverse bars with wire ties, beginning at one edge and working toward the other as shown in Figure 1. The transverse bars without supports were lifted off the subgrade and tied simultaneously with the others.

The longitudinal bar at the edge was tied to the transverse bars at every intersection; the others were tied at every other intersection. When the tying of one group of longitudinal bars had progressed part way across the pavement, work was started on another group. In this way several groups of workmen could tie steel at the same time.

Longitudinal bars were lapped 30 diameters and tied with two wire ties at the lap. The centerline of the laps was approximately perpendicular to the pavement centerline.

A view of the assembled reinforcement is shown in Figure 2.

Figure 1. Workmen Placing Longitudinal Steel

POURING CONCRETE

The concrete was placed in one lift, being deposited through the reinforcement, distributed and consolidated around the steel by the spreader, and further compacted and finished by the finishing machine which made two passes over the pavement. This procedure is shown in Figure 3. The vibrators were operated only during the first pass.

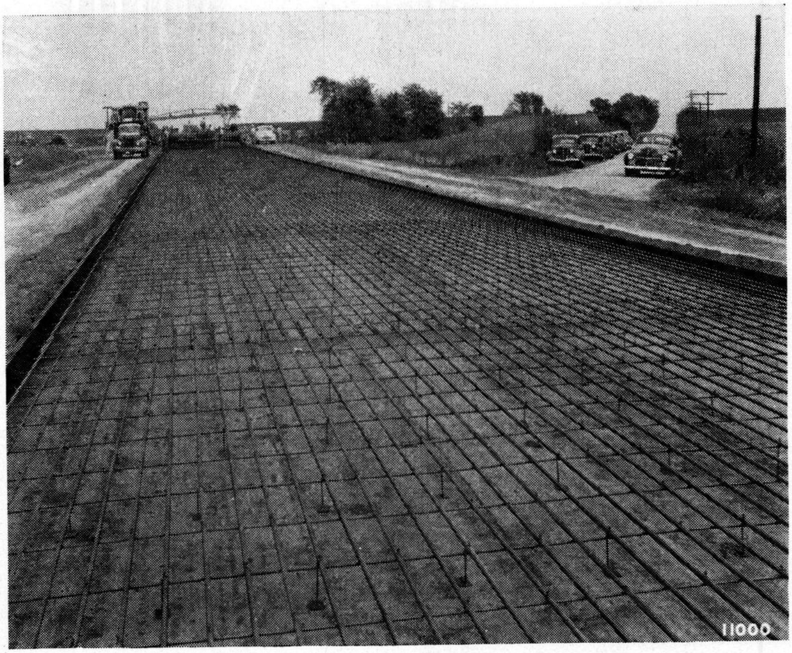

Figure 2. Completely Assembled Reinforcement

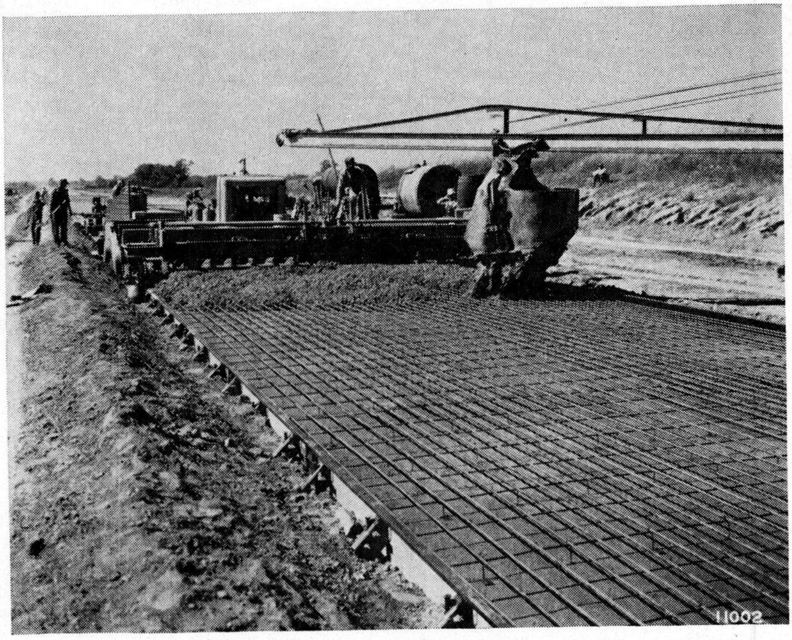

Figure 3. Placing Concrete on Reinforcement

This equipment placed the concrete very satisfactorily. Close observation during placing showed that the concrete flowed well through the openings in the reinforcing mat. Only a slight amount of honeycombed concrete occurred along the edges of the pavement and at the header boards placed at the end of a day's run, which seemed to be good evidence that the concrete was well consolidated.

The wood header board, which was installed at the end of each day's run, was removed when paving was resumed, the concrete being poured against the end of the adjacent slab. Header boards were placed not less than 5 ft. from a lap in the steel.

Figure 4. Measuring Horizontal and Vertical Movements From a Reference Monument

The eight test sections were separated from each other and from the adjacent 10-in. standard pavement by a 4-in. expansion joint poured with an asphalt filler. At five of the joints the ends of adjacent slabs rest on concrete sills. At three the subgrade for a distance of 10 ft. each side of the joint was replaced to a depth of 6 in. with granular material. The pavement at the remaining two was built on the natural subgrade without additional support or subgrade treatment.

REFERENCE MONUMENTS AND REFERENCE PLUGS

For the purpose of studying longitudinal and vertical movements of the pavement, reference monuments were constructed at each end and at intervals of 400 ft. from one end to the center of each test section. The top of each monument is about 3 in. below the surface of the pavement and is protected by a heavy concrete slab fitted with a locking type steel cover designed for easy access.

Measurements are made between the monuments and brass reference plugs set in the edge of the pavement approximately 24 in. on either side of the monument, using a specially designed vernier caliper-clinometer which will read to a thousandth of an inch horizontally and a hundredth of an inch vertically. A reference monument and the method of measurement are shown in Figure 4.

Near the middle of each test section thirty-six brass reference plugs were set at 10-in. centers along one edge of the pavement for the purpose of measuring the width of cracks that may develop in that region. The distance between plugs is measured with a 10-in. Whittemore strain gage.

Reference plugs for taking precise elevations were also placed at 200-ft. intervals along the opposite edge of the pavement from the monuments, and along transverse lines opposite each monument.

STRAIN GAGE PANELS

When the investigation was planned, it was realized that it would be very desirable to determine the stresses in the longitudinal reinforcing steel, particularly at transverse cracks where the steel would carry the entire tensile load. The only method which appeared to be adaptable to this work involved the use of electrical resistance wire strain gages, commonly called SR-4 strain gages. A study of these gages in the laboratory and a pilot installation in a pavement in the fall of 1946 were sufficiently promising to warrant the installation of gages in the experimental pavement. The most difficult problem encountered was that of providing a durable flexible water-proof coating over the gages.

It was originally planned to install gages in each test section but, for a number of reasons, they were installed in only two. It was desirable to have the gages on bars which would be subjected to fairly high stresses but not exceeding the yield point of the steel. Since 0.7 percent of steel appeared most nearly to meet this requirement, both the 7-in. and 8-in. test sections containing that percentage were chosen for the strain gage installations. The installations include 113 active gages, 53 in test section 3 and 60 in test section 6.

In addition, each of these test sections has six temperature compensating gages.

The gages were installed near the middle of the test section. Bars with gages previously attached and waterproofed were installed on the subgrade sufficiently in advance to permit circuits to be run, connected, and tested before the paver reached that location. Gages were located along eight transverse lines, $2\frac{1}{2}$ ft. apart, perpendicular to the center-line of the pavement.

In six of these lines, gages were placed in one traffic lane only; in the other two they were installed in both lanes. Gages were placed on selected bars along each line and, since the presence of waterproofing material at a gage destroys the bond for a length of about 4 in., those bars which did not carry a gage in that particular line were likewise treated with waterproofing material. With the bond thus destroyed on all bars along each line, the gages on the selected bars should reflect the average strain condition for all bars. The presence of the waterproofing, however, may result in a wider opening of cracks than will occur at other locations.

It was considered desirable also to eliminate, as much as possible, from the stress indicated by the gages, flexural stresses caused by curling of the pavement, and, for this reason, all the bars were placed with the gages on the side of the bars.

It is anticipated that eventually transverse cracks will develop along one or more lines of gages. But to assure one such crack and to obtain early measurements of strain at a crack, a depressed contraction joint was installed along one of the lines containing gages in both traffic lanes.

Circuits from the gages were carried along the subgrade to the edge of the pavement, thence through a metal conduit across the shoulder to a panel board housed in a weather-proof metal cabinet; shown in Figure 5. A No. 16 AWG solid copper wire with a polyvinyl covering, recommended for under-ground installation, was used for the circuits. A completely wired strain gage panel is shown in Figure 6.

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

Half batches of concrete were deposited within the area in which strain gages are located and spread and consolidated in the usual manner. As soon as the finishing equipment had advanced beyond the strain gages, a reading was taken on each gage to establish its initial or no-strain reading.

Unfortunately the weather caused work to be halted for the winter before the pavement was completed. The remaining three test sections, including the strain gage panel in test section 6, will be built as soon as construc-tion is resumed in 1948.

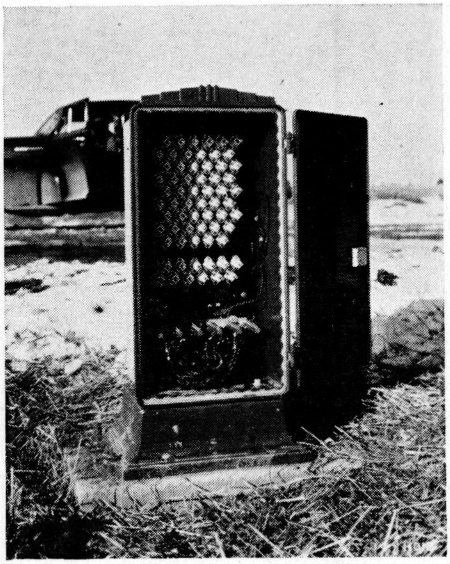

Figure 5. Panel Board and Terminal Box for Strain Gage Circuits

SCHEDULE OF OBSERVATIONS

The tentative minimum program of observa-tions to be carried on over an indefinite period of years, includes the following:

1. A detailed crack survey at least once each year on each section. Two or more sections will be selected for more frequent surveys in order to obtain information relative to the rate of crack development.

2. Measurements of horizontal and vertical movements of the pavement with respect to the reference monuments four or more times a day during one week in each of the four seasons of the year. This will include meas-urements across the expansion joints separat-ing the test sections.

3. Strain measurements in the reinforcing steel in the strain gage panels hourly during one week in each of the four seasons of the year. Temperature readings of the concrete and subgrade will be taken at the same time.

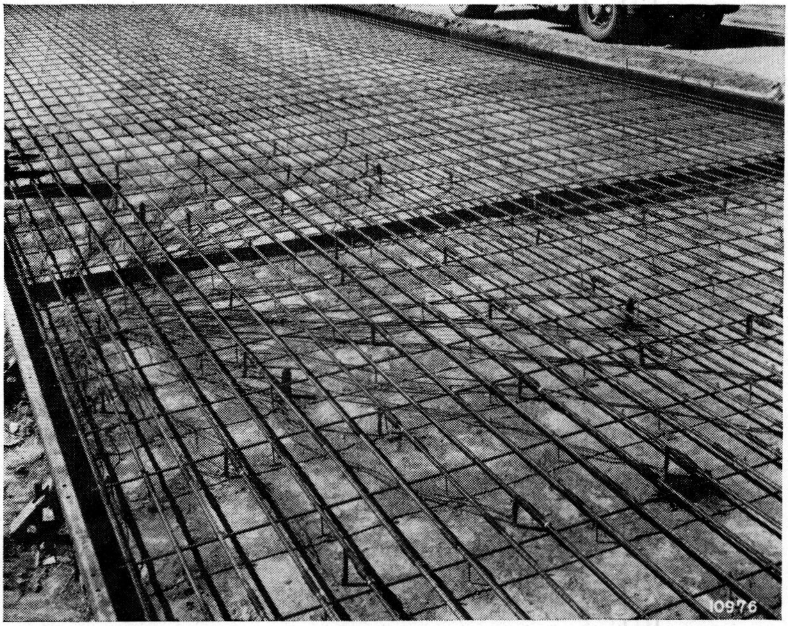

Figure 6. A Completely Wired Strain Gage Panel

4. Precise levels perhaps twice a year or as personnel will permit.

5. Condition survey of pavement once each year.

EARLY DEVELOPMENTS

Numerous fine transverse cracks have developed as expected in the test sections and are held so tightly closed that they are extremely difficult to find even in the morning when the pavement is contracted. In the afternoon when the concrete is expanded the cracks often close so tightly that they cannot be seen with the naked eye. The results of an early crack survey on the five completed test sections are given in Table 2.

For the first 3 days after the concrete was poured in the strain gage panel in test section 3 practically all of the gages indicated similar strains, with maximum daily variations between early morning and mid afternoon of about 150 microinches per inch. Tensile strains were indicated in the mornings when the concrete was contracted and zero strain, or sometimes a small amount of compressive strain, was measured in the afternoons. During this time the transverse crack over the depressed contraction joint was hardly visible except at the edges of the pavement.

TABLE 2

RESULTS OF CRACK SURVEY ON EXPERIMENTAL CONTINUOUSLY REINFORCED PAVEMENT

Test Section	Slab Thickness	Per-cent Reinforcement	Length	No. of Transverse Cracks[a]	Average Crack Interval	Min. Average Crack Interval[b]	Age at Time of Survey
	in.		*ft.*		*ft.*	*ft.*	*days*
1	7	0.3	3504	201	17.34	9.91	37
2	7	0.5	3504	189	18.44	10.00	34
3	7	0.7	3504	258	13.53	6.67	32
7	8	1.0	4231	204	19.56[c]	6.25	18
8	7	1.0	4233	498	8.48	4.76	22

Note: Curing paper left on most of Section 7 until the day before survey was made; removed from other sections in 3 to 4 days after pouring.

[a] Includes construction joints.

[b] Based on 100 ft. of pavement having largest number of cracks.

[c] Based on 3990 ft. of section.

On the afternoon of the third day the paper used for curing was removed from the pavement. The next morning the transverse crack. No significant changes have occurred in the strains indicated by the other lines of gages.

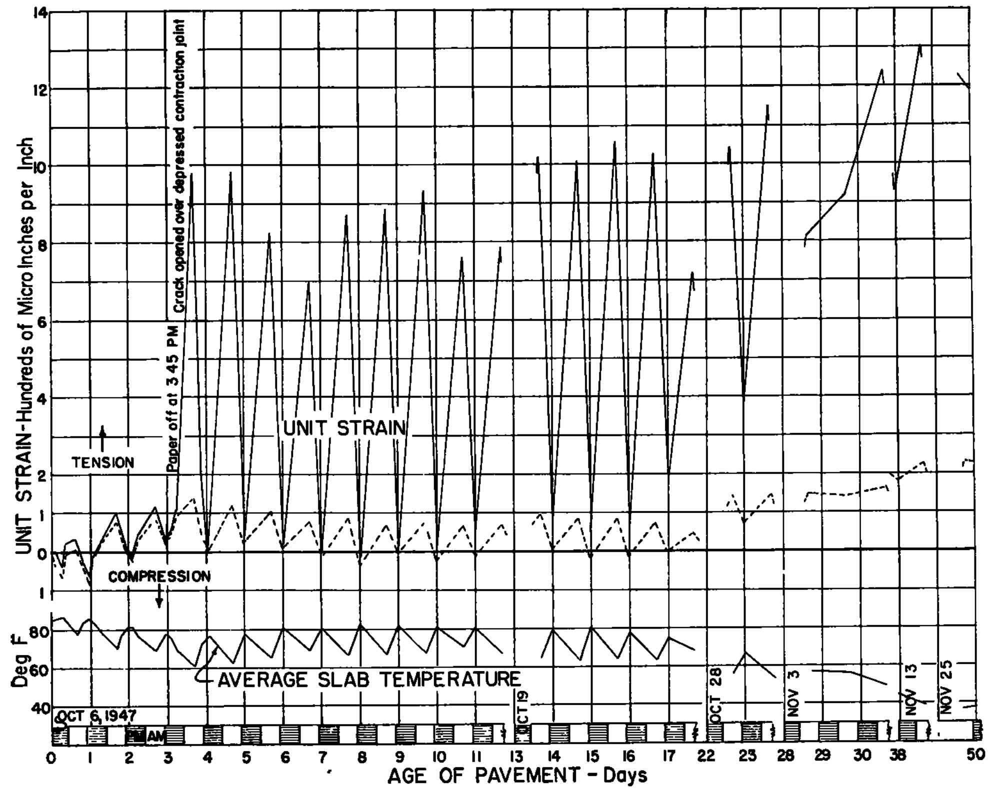

Figure 7. Unit Strain Measured in Longitudinal Reinforcement in Test Section 3—Section 3 contains 0.7 percent longitudinal reinforcement consisting of 42 ⅝-in. round deformed rail steel bars at 6¼-in. c. to c.—Solid line indicates average of 9 gages along transverse contraction joint—broken line indicates average of 35 gages along seven transverse lines all located some distance from cracks or joints. No readings were taken where gaps are shown.

crack over the depressed contraction joint had an opening of 0.010 in. and the gages along the joint showed unit strains of approximately 1000 microinches per inch, which is equivalent to a tensile stress in the reinforcing steel of approximately 30,000 lb. per sq. in. By 2:00 PM the crack was so tightly closed that it was hardly visible and the stress had fallen to approximately 600 lb. per sq. in.

Daily readings taken since then indicate the same trend, a relatively high tensile stress in the morning accompanied by a measurable opening of the crack, and a reduction of the stress in the afternoon with a closing of the

The strain readings taken from October 6 to November 25, 1947, are plotted in Figure 7. This chart compares the average strains measured by those gages which span the transverse contraction joint with those measured by all the other gages in the panel, the latter being located some distance from the joint and the single transverse crack which was then present within the area of gages. The maximum average unit strain measured at the transverse joint is 1310 microinches per inch, which is equivalent to a stress of 39,300 lb. per sq. in. The yield point of the

steel is about 70,000 lb. per sq. in. This illustrates clearly the need for adequate continuous reinforcement if transverse cracks are to be kept tightly closed.

It is much too early for observations and measurements to be too significant, but it is gratifying that they indicate no unexpected or undesirable developments and that the instruments and installations are working satisfactorily.

PROGRESS REPORT ON LOAD DEFLECTION TESTS DEALING WITH LENGTH AND SIZE OF DOWELS

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SYNOPSIS

In 1934 the Michigan State Highway Department started a comprehensive investigation for the evaluation of load transfer devices. The primary object of the investigation was to develop a test procedure for determining the relative efficiency of various types of load transfer devices on the market to the end that definite specifications for their selection and use could be developed. Two progress reports on the results of this work have appeared in the proceedings of the Highway Research Board.

The purpose of this paper is to present the results of another phase of this investigation dealing specifically with the development of a load deflection test procedure and its use in studying the mechanical characteristics and efficiency of dowels of varying length and diameter.

The paper describes the testing machine and test procedure followed in conducting the tests. It presents graphs showing the load deflection characteristics of dowels of three sizes: ¾ in., 1 in., and 1¼ in. and for lengths in each size of 10, 15, 20, 24, and 30 in. Data bearing on residual deflections and joint rigidity are also presented.

The work suggests the need of a recognized test procedure for evaluating load transfer devices. Further, the data indicate that the length of dowel beyond 10 in. has very little influence on deflection. However, there appears to be an optimum length of dowel for maximum performance irrespective of diameter. That length is between 15 and 20 in. A physical quantity for the measurement of rigidity is suggested and designated "joint modulus." It is evaluated by dividing the shear force in pounds by deflection in inches. For shear values within normal highway load ranges this value is approximately constant and may be assigned a definite average value for each type of load transfer device.

In 1934 the Michigan State Highway Department became vitally interested in the problem of evaluating load transfer devices and established a comprehensive investigation on this subject. The primary object of the investigation was to develop a test method for evaluating load transfer devices to the end that a definite specification for the selection and use of load transfer devices could be developed. Progress reports on the results of this work so far have been published previously in the proceedings of the Highway Research Board (*1*, *2*)[1].

[1] Italicized figures in parentheses refer to list of references at the end of the paper.

There is unquestionably a great need at the present time for such a test procedure because of the continual appearance on the market of new mechanical load transfer devices to replace the common dowel bar and also because it is imperative that we know the mechanical and physical characteristics of all types of load transfer devices and can predict with reasonable accuracy the performance of such devices under continual service, in order that they can be intelligently designed and properly spaced in a pavement joint.

The purpose of this paper is to present the results of a phase of this investigation on the evaluation of load transfer devices dealing