In these instances, the use of strip photography can effect a considerable saving of time and expense. An application of this type is frequently encountered in by-pass locations where the high cost of right-of-way in urban and suburban areas often makes it necessary to consider alternate locations in order that the most feasible and economical location can be adopted. Likewise, where the choice of a highway location is contingent upon the clearing of timber, a reasonably accurate estimate of size and number of trees involved can be made from a strip photograph covering the location.

Another application of strip photographs to location surveys is in obtaining topographical information. When done in the field this work consists mainly of the location of fence lines, access drives, trees, etc., and only moderate accuracy is required. With the use of strip photographs this same work can be performed with corresponding accuracy and considerably less cost. Those familiar with field survey work will no doubt appreciate the benefits of this use of strip photographs. Other applications of aerial strip photography to location work include reconnaissance surveys for the location of power transmission lines and pipe lines and any other installation which requires a narrow, cross-country right-of-way.

A COMPARATIVE STUDY OF DATA FROM THE COOPERATIVE INVESTIGATION OF JOINT SPACING

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SYNOPSIS

In 1945 the Highway Research Board published a special bulletin containing progress reports from each of the six States participating in the cooperative investigation of the spacing of expansion joints in pavements with closely spaced contraction joints. Also included in the bulletin is the related study of the structural effectiveness of transverse joints of the weakened-plane type. The reported data are presented in this paper as a summary analysis with the main objective of ascertaining common developments.

Since construction of the experimental pavements in 1940 and 1941, all States have made measurements and observations of the following: (1) temperature variations; (2) changes in the widths of the joints; (3) changes in pavement elevation; and (4) condition of the pavement and joints. However, because of the short time that the pavements have been in service, the present comparative study of the data has been limited to the reported daily, seasonal and progressive or permanent changes in the widths of expansion and contraction joints.

The most important developments observed in the study of the movements at the joints are: (1) expansion joints close progressively with time, the amount of permanent closure being greatest during the first annual cycle; (2) closure of expansion joints apparently continues until all available expansion space has been used up; (3) contraction joints open progressively with time, the rate of opening being greatest during the early life of the pavement; (4) the magnitude of the permanent opening of contraction joints decreases with an increase in the spacing of expansion joints, other conditions being equal.

In the related study of the structural behavior of weakened-plane contraction joints it was found that: (1) in the presence of interfacial pressure, much less than might develop in pavements under conditions of restrained expansion, weakened-plane joints are very effective in reducing critical stresses; and (2) without interfacial pressure, aggregate interlock is an uncertain means of stress control regardless of type and size of coarse aggregate.

Some measure of the structural effectiveness of the contraction joints in the experimental pavements was provided by comparing their observed openings with the relations established from the joint efficiency study mentioned above. It was concluded from this comparison that contraction joints of the weakened-plane type are, on the whole, not effective in controlling critical stresses caused by loads acting at their edges.

Early in 1940 the Public Roads Administration sponsored a cooperative investigation for the purpose of studying the effects of varying the spacing of expansion joints in concrete pavements containing closely spaced weakened-plane contraction joints. Three of the six participating State Highway Departments, Kentucky, Michigan and Minnesota constructed experimental pavements embodying such features in 1940. California, Missouri and Oregon built similar pavements the following year. Construction was, as nearly as possible, in accordance with the standards of each State. Also, as part of the investigation, a study of the structural efficiency of transverse joints of the weakened-plane type was conducted by the Public Roads Administration at the Arlington Experiment Farm, Virginia.

The pavements in Kentucky, Michigan and Minnesota are described in *Proceedings*, Highway Research Board, Vol. 20 (1940), and those in Oregon and Missouri in Vol. 21. A description of the California pavement is included in the 1945 report of that State which together with progress reports describing the developments and trends that have become evident during the early life of all these pavements were published in 1945 in a special bulletin of the Board,[1] together with the related study of the efficiency of weakened-plane joints.[2] The purpose of this paper is to present a summary analysis of the reported data.

Certain details of the design features of the experimental pavements are shown in Table 1. This table includes only those parts of the pavements from which data were taken for the summary analysis. Several States incorporated in their pavements additional designs that are of interest but do not lend themselves to comparative study.

As observed in Table 1 there are some differences in the construction and design features of the pavements. Briefly, the most important differences are:

1. The amount of expansion space was held constant for the various expansion joint spacings in the pavement of each State except in that of Michigan. In this State the expansion space was varied with the distance between the expansion joints by separating the longer subsections with either one or two short relief slabs. This method provided a single 1-in. wide joint at the ends of the 120-ft subsections, two at the ends of the 480- and 900-ft subsections and three at the ends of the 2,700-ft subsections.
2. The width of expansion joints was 1 in. in the pavements of Minnesota, Missouri and Kentucky and ¾ in. in those of California and Oregon.
3. Plain round dowels were used in the expansion joints of all of the pavements except in Missouri, where the Translode type of load transfer device was employed.
4. Expansion joint fillers were of a plastic character in all cases except in the California pavement in which a redwood board filler was used.
5. The methods employed in forming and sealing the weakened-plane contraction joints varied in each State.

These design differences naturally result in some variations in the behavior of the experimental pavements, but it is doubtful if the differences are or will be of such character as to have an important bearing on the main objective of the investigation.

In Table 2 are presented data relating to the construction of the different pavements and to the time at which the basic set of measurements were made to determine joint-width changes. It will be noted that: (1) with the exception of the California pavement and part of that in Michigan, all were laid during the summer months; (2) methods used in curing the pavements varied; and (3) the time at which the basic joint-width measurements were obtained ranged from immediately after the concrete had taken its initial set to several months after the pavement was laid. The

[1] Highway Research Board, Research Report No. 3B (1945).

[2] Also in *Public Roads*, April–May–June, 1945.

locations of the pavements were purposely selected to give relatively light grades and flat curves.

Since construction of the experimental pavements all States have made measurements and observations of the following: (1) Daily and seasonal variations in temperature; (2) Daily, seasonal and progressive or permanent changes in the widths of the expansion and contraction joints; (3) Changes in elevation of the pavement, especially with respect to faulting at the joints; and (4) The general condition of the pavements and joints.

sents the average of several expansion joints. In this and subsequent figures the lengths of the stippled bars indicate the changes in width that occurred in the joints during an annual cycle. Since it was not always possible to make measurements at joints under extreme temperature conditions, the annual width changes presented are not necessarily maximum for the yearly cycle.

In spite of the previously mentioned differences in design and time of basic measurements, it is indicated in Figure 1 that: (1) expansion joints close progressively with time

TABLE 1
DESIGN DATA ON THE EXPERIMENTAL PAVEMENT INCLUDED IN THE COMPARATIVE STUDY

State	Cross Section	Expansion Joints			Contraction Joints		Reinforced Section[a]	
		Width	Filler	Load Transfer	Type	Load Transfer	Panel Length	Weight of Reinforcement
	In.	*In.*					*Ft*	*Lb per 100 sq ft*
Michigan	9–7–9	1[b]	Preformed bituminous fiber	Dowels	Flexplane ribbon	Dowels	60[c]	60
Minnesota	9–6–9 7 unif.	1	Preformed bituminous fiber or granulated cork	Dowels except as noted[d]	Grooved, copper water seals and latex-oil mixture in majority	No dowels except as noted[e]	60	
Missouri	9–7–9 9 8–7 8–9 8	1	Preformed bituminous fiber	Translode base	Grooved, pressure injected Tarvia XC	do.	60	70
Kentucky	9–7–9 7 unif.	1	Preformed bituminous fiber	Dowels except as noted[d]	Bituminous fiber strip (sealed)	do.	60	70
California	9–7–9 8 unif.	¾	Redwood strips	do.	Grooved, poured blended asphalt	do.	60	70
Oregon	9–7–9 8 unif.	¾	Preformed bituminous fiber	do.	Asphalt impregnated felt strip	do.	60	

[a] 120-ft spacing of expansion joints
[b] Either 1, 2 or 3 one-in. wide joints, depending on length of subsection
[c] Divided by dummy joint with reinforcement continuous through joint
[d] No dowels in uniform-thickness section. This section has expansion joints at 120-ft intervals
[e] Dowels in reinforced section and in either one or two of the sections having expansion joints at 120-ft intervals

Because of the relatively short time that the pavements have been in service, conclusions cannot be drawn regarding items 3 and 4; hence, the following summary analysis will be confined to a discussion of the changes in the widths of expansion and contraction joints.

EXPANSION JOINTS CLOSED PROGRESSIVELY

A comparison of the annual and progressive changes in the widths of the expansion joints of the non-reinforced pavements is shown in Figure 1. Annual joint-width changes were computed from data obtained in the winter and summer of each year and are with respect to the basic set of measurements taken at the time indicated in Table 2. Each value repre-

the amount of permanent closure being greatest during the first annual cycle; (2) after the first annual cycle, subsequent permanent closure of expansion joints is relatively small and tends to diminish progressively with time; (3) the magnitude of the permanent closure of expansion joints appears to increase with an increase in their spacing, but the influence of spacing is slight for intervals greater than approximately 400 ft; and (4) the magnitude of the permanent closure of expansion joints decreases slightly as the spacing of contraction joints increases, other conditions being equal (See data from Michigan and Minnesota).

The great variation in the magnitude of the closure of the expansion joints in subsections

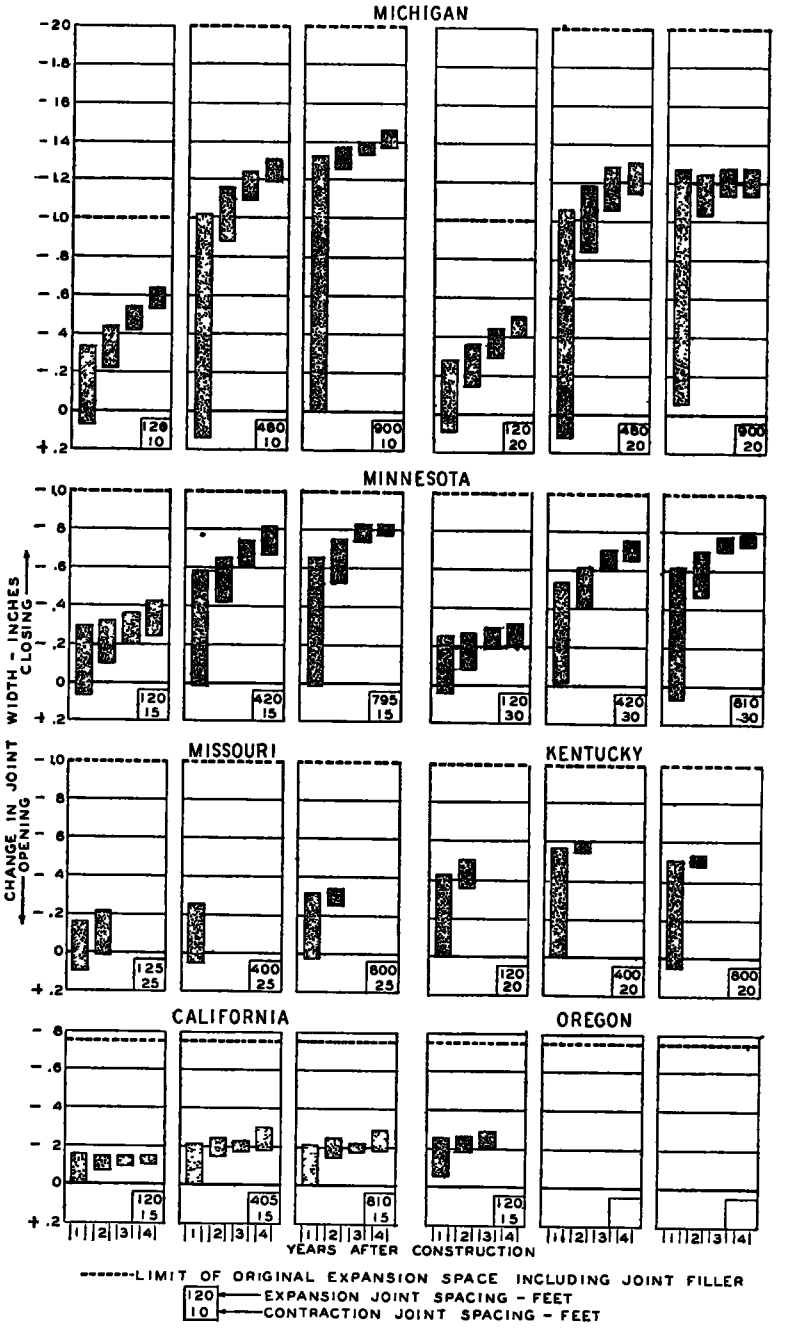

Figure 1. Annual and Progressive Changes in the Width of the Expansion Joints—Non-Reinforced Sections

of comparable lengths probably results from combinations of several factors, which include: (1) amount of available expansion space; (2) restraint to movement offered by joint fillers

and load-transfer devices; (3) amount of permanent opening of the contraction joints that develops as a result of the accumulation of foreign material in the joints or by relative displacement of the joint edges which prevents a close meshing of the joint faces; (4) differences in climate; (5) amount of expansion that develops beyond the length at which the concrete hardened; (6) time and condition under which the basic measurements were made; and (7) differences in the magnitude of subgrade resistance.

The influence of available expansion space is obvious in comparing the data from Michigan with those from California. In this connection California was the only State that used a nonplastic joint filler. The comparatively small movements found at the expansion joints of the Missouri pavement may be due to the type of load-transfer device that was used. As stated earlier, this State employed the Translode type of device, whereas the other States used conventional dowels. The Missouri pavement also had three special sections with expansion joints spaced at 125-ft intervals in which the load-transfer devices were omitted. At the end of the second summer following construction the reported average closure of the expansion joints of these special sections was nearly 50 percent greater than that for the similar sections with load transfer. Thus, there is some reason to suspect that the load-transfer devices used in the expansion joints of the Missouri pavement restrained progressive movements at the joints.

TABLE 2
CONSTRUCTION DATA ON THE EXPERIMENTAL PAVEMENTS

State	Length	Time of Laying Concrete		Method of Curing	Time of Base Measurements at Joints
		Year	Month		
	Mi				
Michigan	10 7	1940	July 31 to Oct. 25	Burlap and straw for a total of 7 days	Immediately after completion of each section
Minnesota	8.1	1940	Aug. 6 to Sept. 20	Impermeable fiber filled paper for 72 hr	Early in Oct 1940
Missouri	7 0	1941	June to early Aug	Transparent membrane sprayed on pavement	Aug. 1941
Kentucky	6 3	1940	July 8 to Aug. 16	Burlap and Sisalkraft paper for a total of 4 days	Nov 27, 1940
California	5 7	1941	Sept. 20 to Oct. 29	Moist earth for 8 days	Feb 1942
Oregon	3 8	1941	June 10 to July 7	Wet cotton mats for 72 hr	Immediately after concrete had taken initial set

CONTRACTION JOINTS OPENED PROGRESSIVELY

The annual and progressive changes in widths that occurred in the contraction joints of the non-reinforced pavements are shown, in a comparative manner, in Figure 2. These width changes are with respect to the basic set of measurements obtained at the time indicated in Table 2; and represent, as closely as can be determined from reported data, the amount that the average joint, in a given section, changed in width. The differences in the positions of the annual joint-width changes with respect to their base line is explained by the fact that the basic measurements were made during the upper range of annual temperatures in Missouri and Oregon, at intermediate temperatures in Michigan and Minnesota and at the lower range of annual temperatures in Kentucky and California.

In Michigan and Oregon the basic measurements were obtained before an appreciable number of the contraction joints had fractured so that the observed changes in the widths of the joints in the pavements of these two States are indicative of the true or total joint openings. In the other States the basic measurements were made after an undetermined number of the joints had fractured; hence, the true joint openings are not known. At the end of the first year, the magnitude of the joint movements as related to the basic measurements indicated that practically all, if not all, of the contraction joints had fractured.

In the discussion of Figure 1 certain factors were mentioned which influenced the per-

manent closures of the expansion joints. Because of the interrelation of the permanent or progressive changes in the widths of expansion and contraction joints the factors mentioned also influenced the permanent openings of the contraction joints. However, despite these influences, the data of Figure 2 clearly indicate that: (1) contraction joints open progressively with time, the rate of opening being greatest during the early life of the pavement; (2) the progressive opening of contraction joints continues for a longer period of time in pavements with closely spaced expansion joints than in those having expansion joints spaced at greater intervals; (3) the magnitude of the permanent opening of contraction

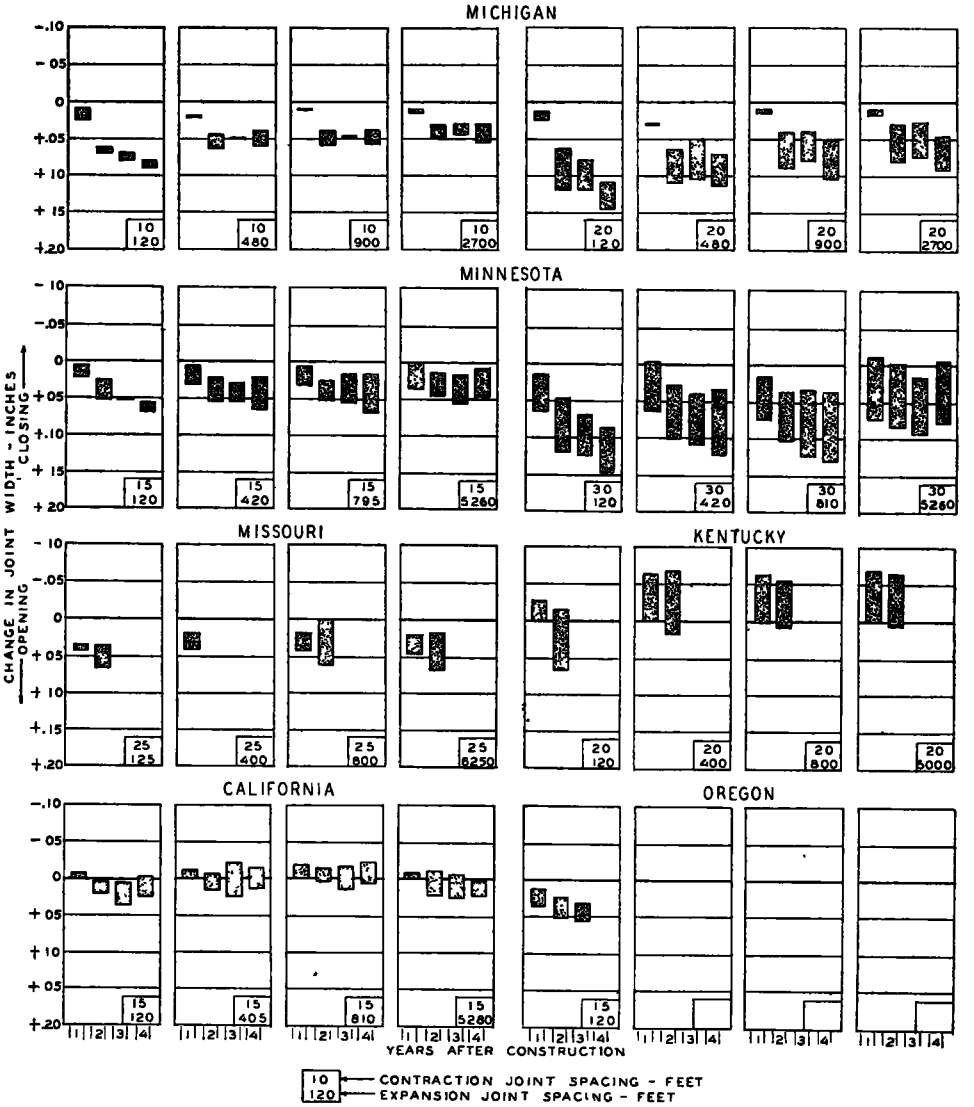

Figure 2. Annual and Progressive Changes in the Width of the Contraction Joints—Non-Reinforced Sections

joints decreases with an increase in the spacing of expansion joints; (4) the amplitude of the annual joint-width change of contraction joints increases with an increase in their spacing, other conditions being equal (see data from Michigan and Minnesota); and (5) the spacing of expansion joints, with the possible exception of those at 120-ft intervals, appears to have little, if any, influence upon the amplitude of the annual joint-width change of contraction joints.

The progressive changes in the widths of the contraction joints that have occurred in each pavement in the interior of the longest section without expansion joints and in the sections with expansion joints at 120-ft intervals are presented in Figure 3. These progressive changes were determined from summer observations and are with respect to measurements made during the first summer following construction of the pavements. The values for the longest sections are the averages of a number of centrally located contraction joints, whereas those for the sections with expansion joints at 120-ft intervals are averages of all contraction joints between the expansion joints.

At the time of the basic measurements vir-

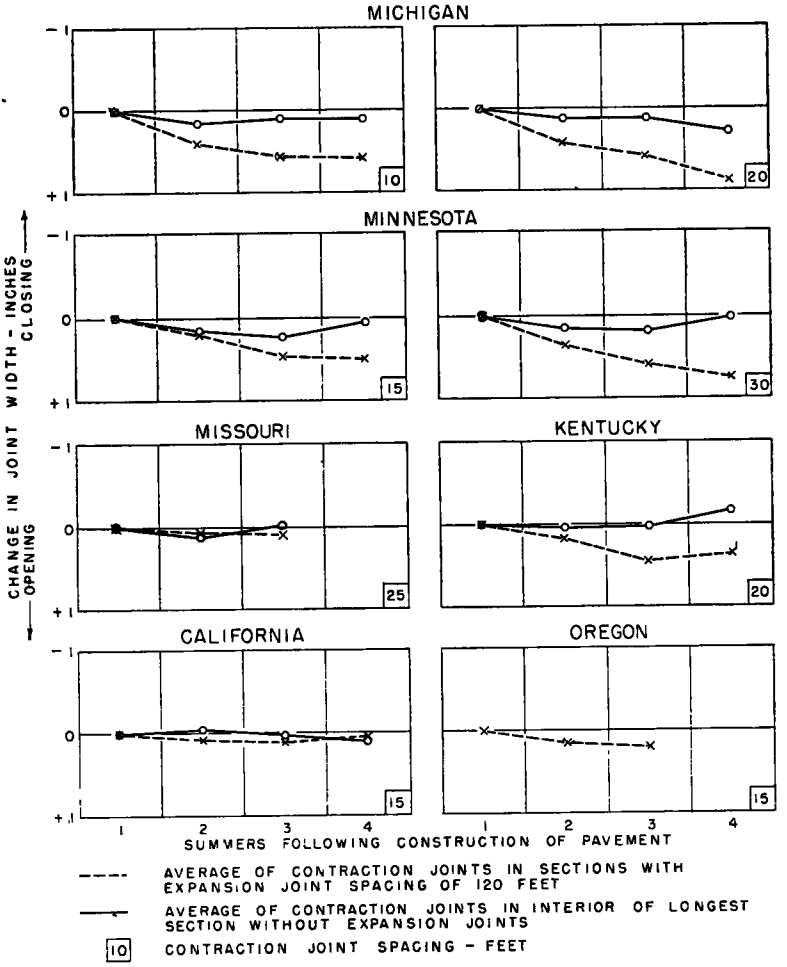

Figure 3. Progressive Changes in Contraction Joint Openings with Respect to Measurements Made During the First Summer Following Construction of the Pavement

tually all of the contraction joints had undoubtedly fractured. Thus the joints were opened a small amount when these measurements were made so that their actual openings are somewhat greater than those indicated in the figure.

The data of Figure 3 substantiate the indications of the previous figure in showing that contraction joints, unless restrained, open progressively over a period of several years in pavements with expansion joints spaced at 120-ft intervals. This same action is evident near the ends of sections with greater expansion joint spacings and may continue until all available expansion space has been depleted.

Changes in the widths of the contraction joints were probably influenced to a slight degree by variations in the temperature and moisture content of the concrete and by warping of the slabs at the time that measurements were obtained. This may explain the somewhat erratic trend of the data of Figure 3. Also, the small width changes shown for the joints in the central part of the longest sections may have been caused in part by the variables just mentioned. As the measurements are continued the effects of these variables will tend to become less evident in the data.

It appears from the data of Figure 3 that after the first summer following construction the observed differences in the magnitude of the progressive openings of the contraction joints in comparable-length subsections of the different pavements can be explained mainly by variations in: (1) amount of available expansion space; (2) restraint to movement offered by the joint fillers and load-transfer devices; and (3) spacing of the contraction joints. Considering the 120-ft spacing of expansion joints, the greatest permanent openings of the contraction joints were observed in the Michigan, Minnesota and Kentucky pavements which had 1-in. wide expansion joints, and the least in the pavements of California and Oregon with ¾-in. joints. However, the permanent opening of the contraction joints in the Missouri pavement, which also had 1-in. expansion joints, is more nearly comparable to that observed in the California and Oregon pavements with the ¾-in. joints. As remarked before, it is believed that the type of load-transfer device used in the expansion joints of the Missouri pavement restrained progressive joint movements. In reference to the California pavement, it is probable that the nonplastic joint filler restrained movements at joints more than the type used in the other States. The effect of the spacing of contraction joints is observed in the data from Michigan and Minnesota pavements. These data indicate that the permanent opening of the contraction joints is slightly greater for the greater spacing of such joints.

It is possible that some of the differences in the amount of progressive opening that have been observed in the contraction joints of the various pavements were caused by differences in the forces or conditions tending to cause the joints to open. For example, the rate at which the joints progressively open may be influenced by the magnitude of the daily and seasonal changes in the widths of the joints, by volumetric changes in the concrete, frost action and other factors.

To summarize, the data of Figures 1, 2 and 3 indicate that, in pavements which are divided into short slab lengths by means of intermediate contraction joints between the expansion joints, a temperature rise causes the individual slab units to be moved over the subgrade toward the expansion joints as the pavement expands beyond the length it acquired when it hardened. Such action tends to cause a permanent closure of the expansion joints. After the pavement has attained its maximum length from volumetric changes, the tendency is for the individual slab units to expand and contract about their individual centers. However, if during periods when the contraction joints are open, they are, for some reason, restrained from completely closing again, as for example by infiltrated material, a second rise in temperature will tend to cause the individual slab units to be again translated over the subgrade as the pavement expands. This leads to further closure of the expansion joints and the action may continue until all available expansion space has been dissipated.

MOVEMENTS AT THE EXPANSION AND CONTRACTION JOINTS OF THE REINFORCED SECTIONS COMPARED

The annual and progressive changes in the widths of the expansion and contraction joints

in the reinforced sections of the different pavements are shown in Figure 4. These sections are divided into 60-ft panels by alternate expansion and contraction joints, the steel being interrupted at the joints. The procedure followed in the construction of these graphs was the same as that described in the discussion of the previous figures for the plain concrete sections.

A progressive closing of the expansion joints and a progressive opening of the contraction joints has been observed in the reinforced pavements of the different States. These progressive changes as well as the annual changes in the widths of the joints vary among the different pavements and, except for magnitude, the variations are similar to those found in the plain concrete sections. The probable reasons for these variations have been mentioned in the discussion of the joint-width changes of the plain concrete sections.

It will be noted that the annual changes in the widths of the expansion joints in the reinforced sections are much greater than those of the contraction joints. Since the same types of load-transfer devices were used in both expansion and contraction joints in all cases except in the Missouri pavement, this behavior can hardly be attributed to differences in resistance offered by these devices. This same phenomenon was observed over a period of several years in two plain concrete sections of the Arlington experimental pavement.[3] Each of these sections was divided into two panels of equal length by a contraction joint. One of the sections contained dowels in the contraction joint and no load-transfer devices in the expansion joint, while in the other there were no load-transfer devices in either the expansion or contraction joints. In both sections the observed annual changes in the widths of the expansion joints were much greater than those of the contraction joints. It appeared from the data obtained in the Arlington study that the greater annual changes in the widths of the expansion joints compared to those of the contraction joints were caused by a shifting or translation of the panels of the section over the subgrade due to some condition of restraint.

[3] "The Structural Design of Concrete Pavements," Part 4. *Public Roads*, Vol. 17, No. 7, September 1936.

Comparison of the maximum openings thus far observed at the contraction joints separating the 60-ft panels of the 120-ft reinforced sections with maximum openings observed at the contraction joints separating the shorter

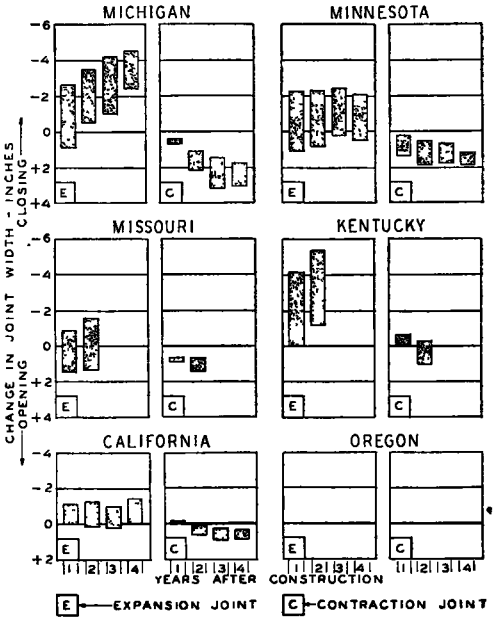

Figure 4. Annual and Progressive Changes in Width of the Expansion and Contraction Joints—Reinforced Sections

panels of the 120-ft plain concrete sections (Fig. 2) are, as follows:

State	Ratio of Slab Lengths	Ratio of Maximum Contraction Joint Opening
Michigan	6:1	3.5:1
	3:1	2.2:1
Minnesota	4:1	2.7:1
	2:1	1.3:1
Missouri	2.4:1	2.1:1
Kentucky	3:1	1.5:1
California	4:1	2.4:1

Thus it is evident from these data that the maximum openings of the contraction joints in the reinforced sections with the 60-ft panels are greater, but not proportionately so, than those in the plain concrete sections with the shorter panels.

DAILY CHANGES IN THE WIDTHS OF THE CONTRACTION JOINTS DISCUSSED

The observed daily changes in the widths of the contraction joints in the plain concrete sections of the pavements of four States are shown in Figure 5. Each value represents the width change of the average joint in a given section, and is based on data obtained on several selected days during the summer when large temperature changes occurred. The joint-width changes are not comparable

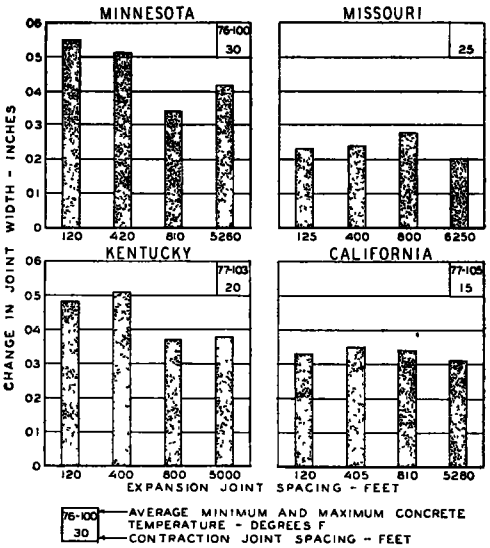

Figure 5. Average Daily Changes in Width of Contraction Joints—Selected Summer Days

between the different pavements because of the variation in spacing of the contraction joints and of daily ranges in temperature. Minimum and maximum concrete temperatures were not reported for the Missouri pavement, but the average daily range was 24 F.

It is evident from the data of Figure 5 that the daily changes in the widths of contraction joints in the pavements of Minnesota and Kentucky are greater in the sections with closely spaced expansion joints than in those with wider spacing of such joints. This may have been caused by restraint of expansion at the higher temperatures in the sections having greater expansion joint spacings. Similar behavior is not present in the pavements of Missouri or California, which indicates more nearly equal daily restraint or freedom at the contraction joints of these pavements regardless of expansion joint spacing.

In considering the measurements of daily change in widths of joints, it should be kept in mind that such measurements are necessarily made at the elevation of the upper surface of the pavement and are therefore influenced by changes in the condition of warping at the ends of the slabs. That is, an increase in downward warping, with respect to that which obtained at the time the measurements used as a base were taken, causes an apparent closing of the joints or lengthening of the slabs. Conversely, an increase in upward warping results in an apparent opening of the joints or shortening of the slabs. It was found in the Arlington investigation that, with respect to the flat condition of the slabs, extreme temperature warping alone caused an apparent closing of the joints of 0.007 to 0.009 in. during the day and an apparent opening of 0.003 to 0.004 in. during the night or early morning. Consequently, daily joint-width changes may be subject to a large error unless allowance is made for the effect of temperature warping. It was also found that seasonal changes in the widths of the joints are not affected nearly as much as the daily changes because of the compensating effect of seasonal warping produced by the moisture gradient within the slab. The reader is referred to the report on the Arlington investigation for a more detailed discussion of this subject.[4]

By assuming a thermal coefficient for the concrete of 0.000005 per deg F, it is revealed that the measured daily changes in the widths of the contraction joints in the 120-ft sections of the pavements of Minnesota, Kentucky and California are, respectively, 0.012, 0.017 and 0.008 in. greater than computed values; whereas in the Missouri pavement such changes are 0.013 in. less than computed values. The fact that the measured daily changes in the widths of the contraction joints in the Minnesota, Kentucky and California pavements are greater than those computed from the accompanying temperature changes raises the question of the effect of warping in the slabs in the interval between the initial and final measurements. The relatively small

[4] Ibid.

daily movements at contraction joints in the Missouri pavement may have been caused by a shifting of the slabs over the subgrade due to some condition of restraint. In this regard it is of interest to note that the average daily movement at expansion joints of the 125-ft sections of the Missouri pavement was three times greater than that observed at contraction joints.

STRUCTURAL EFFECTIVENESS OF WEAKENED-PLANE CONTRACTION JOINTS DETERMINED

As stated earlier, a study of the structural behavior of transverse joints of the weakened-plane type was conducted by the Public Roads Administration as part of the general investigation of joint spacing. This study was made on a test pavement divided into six sections, each being 30 ft long, 20 ft wide and of 8-in. uniform thickness. Each section was divided longitudinally by a deformed metal center joint and transversely by a weakened-plane contraction joint. Because of the nature of the tests the pavement was not subjected to traffic; thus the data obtained from the study furnishes information that is limited to the initial effectiveness of the joints.

In testing the joints an 8,000-lb load was applied through a 50-sq in. circular bearing area to points at the interiors of the panels of a section and to selected points along the edges of the joint. The critical strains for a load acting at a corner occur along the bisector of the corner angle and for a load acting at the edges and interior occur directly under the load. These strains were measured during the time that the load was acting. Strains at the edges of the joints were measured, first with the joints closed under a compressive force of approximately 90 psi, then at various controlled widths of opening and, finally with the joints separated to the extent that their edges were acting as free slab ends.

From the strain data, information was derived regarding the effect of interfacial pressure and various widths of opening on weakened-plane joints incorporated in pavements containing different types and sizes of coarse aggregate. Briefly, the study indicated that: (1) weakened-plane joints are more effective in controlling critical stresses produced by a load acting at their corners than by the same load acting at their edges but at some distance from their corners; (2) in the presence of interfacial pressure, much less than might develop in pavements under conditions of restrained expansion, weakened-plane joints are quite effective in reducing critical stresses; (3) without interfacial pressure aggregate interlock is an uncertain means of stress control regardless of type and size of coarse aggregate; (4) ¾-in. dowels spaced at 12-in. intervals

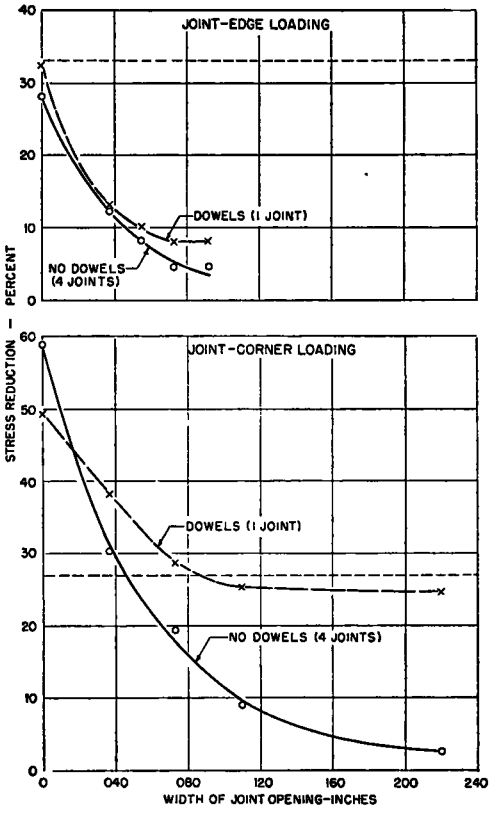

Figure 6. Effect of Width of Joint Opening on Stress Reduction

were considerably more effective in reducing critical corner stresses than they were in reducing critical edge stresses; and (5) the presence of such dowels greatly improved uniformity in stress reduction from point to point along the joint edge.

To summarize graphically Figure 6 was prepared from the data reported on the structural behavior of weakened-plane joints. This figure presents a composite graph of the effectiveness of four weakened-plane joints in controlling critical stresses caused by a load

acting, first at their edges but at some distance from their corners and, second at their corners. These joints contained no load-transfer devices and were incorporated in test sections having different types and sizes of coarse aggregate. Also shown are the data from a single weakened-plane joint with ¾-in. diameter dowels spaced at 12-in. intervals. In testing the edges of this joint the load was applied midway between dowels. Regarding the relations shown for the doweled joint, it is emphasized that the effectiveness of load-transfer devices in controlling critical stresses at the edges of a joint is related to several factors which include slab thickness and subgrade stiffness. Therefore, the relations shown apply only for the particular doweled joint and for the conditions of test stated in the report.

The horizontal dash lines of Figure 6 represent, respectively, the necessary reductions in the stresses determined at the free ends and free corners (joints separated) to make them equal to the stresses produced by an equivalent load acting at the interiors of the sections. These values were established from the average of all of the critical strains obtained at the free ends, free corners and the interiors of the panels of the six sections. Hence, if a reduction of 33 percent is attained at the joint edge, then the joint may be rated 100 percent efficient in stress control for the edge condition of loading. Likewise, the joint may be rated 100 percent efficient in stress control if a 27-percent reduction in stress is found at its corner. In reference to the preceding statements, it will be noted that a joint must be highly effective in load transfer to cause a stress reduction of 33 percent at its edges. However, a joint that is highly effective in that respect would cause a reduction in stress of approximately 50 percent at its corners. Conversely, a joint that effects a sufficient reduction in corner stresses to make them just equal to those of the interior of the slab would not be very effective in controlling load stresses at its edges. Thus, a joint that reduces corner stresses by only 27 percent may not be effective in controlling edge stresses and its ability in the performance of other functions such as reducing the tendency for faulting and pumping would naturally be less than that of a joint with higher load transfer properties.

STRUCTURAL EFFECTIVENESS OF CONTRACTION JOINTS IN THE EXPERIMENTAL PAVEMENTS DISCUSSED

Some measure of the structural effectiveness of the contraction joints in the experimental pavements is provided by comparing their observed openings (see Fig. 2) with the relations of Figure 6. Before making such a comparison, attention is directed to certain facts, namely: (1) traffic and other factors acting over a period of time would undoubtedly influence the relations of Figure 6; (2) the zero widths of opening shown in Figure 6 are the widths established when the joints were closed by a compressive force; and (3) a residual separation of about 0.005 in. existed at the established zero widths of opening.

Referring to the 120-ft sections of Figure 2, it is seen that the maximum widths of opening thus far observed in the contraction joints ranged from 0.05 in. for the pavement of California to about 0.15 in. for the pavements of Michigan and Minnesota. These openings are with respect to the condition at maximum observed closure. The spacing of the contraction joints in these pavements were respectively, 15, 20 and 30 ft. The widths of joint opening just given are those observed during the winter months and include residual separations. The joint openings of the Michigan pavement, as remarked before, are total openings since the basic measurements were obtained before the joints had fractured. In the pavements of both California and Minnesota the total openings are estimated to be between 0.005 and 0.010 in. larger than those given in Figure 2. Hence, the joint openings that occurred before the basic measurements were obtained are about equal to the residual separations at the zero widths of opening of the test joints of Figure 6.

Considering the 0.05-in. joint opening, such as was observed in the California pavement and assuming that the relations of Figure 6 are typical, it is evident that: (1) for the edge condition of loading, contraction joints, either with or without dowels, have a stress reduction of slightly more than ¼ of that necessary to make the free-end stresses equal to those of the interior of the slab; and (2) for the corner condition of loading, both the doweled and undoweled joints caused a suf-

ficient stress reduction in the critical corner stresses to make them approximately equal to those of the interior of the slab. In the case of the 0.15-in. joint openings of the Michigan and Minnesota pavements, the trends of the curves, if the data of Figure 6 can be extrapolated, indicate that, for the edge condition of loading, the stress-reducing ability of contraction joints without dowels would be nil; whereas the ability of those containing dowels would be about ¼ of that necessary for perfect stress control. For the corner condition of loading, the effectiveness of undoweled contraction joints at a 0.15-in. opening is about ⅓ of that desired; whereas doweled joints are nearly adequate in stress control.

Since the magnitude of the permanent or residual separation of contraction joints decreases with an increase in the spacing of expansion joints, it naturally follows that the stress-reducing ability of such joints would increase with an increase in the distance between expansion joints. For example, in the extreme cases of the central portion of the longest sections of the California and Michigan pavements the maximum reported contraction joint openings are, respectively, about 0.04 and 0.08 in. For the edge condition of loading and joints without dowels, these openings give respective stress reductions of about ⅓ and ⅙ of the reduction value necessary for perfect stress control as compared to values of ¼ and 0 determined for the 120-ft sections.

Thus, it must be concluded from the foregoing comparisons that, regardless of expansion joint spacing, weakened-plane contraction joints in pavements in service are, on the whole, not effective in controlling critical stresses caused by loads acting at their edges.

SUMMARY

The most important development shown by the data collected on the pavements of the various cooperating States is the manner in which the expansion joints close progressively and the contraction joints open progressively with time. Except in the pavements of California and Missouri, the progressive opening of the contraction joints has been much greater in the sections with 120 ft expansion joint spacing than in the sections with greater expansion joint spacings. The redwood board expansion joint filler in the California pavement and the particular type of load-transfer device used in the expansion joints of the Missouri pavement apparently restrained the annual closure of the expansion joints in these pavements and thus prevented large progressive openings at the contraction joints.

Aggregate interlock is, in itself, helpful in reducing the tendency for faulting and pumping and has been found to increase the effectiveness, in this regard, of contraction joints including those with load-transfer devices. Thus it appears to be desirable that the progressive opening of contraction joints be restrained as much as possible. As in the case of the California pavement, this might be done in pavements with expansion joints by the use of an expansion joint filler which offers reasonable restraint to the closure of such joints.

The data indicate that in pavements with expansion joints spaced at 120-ft intervals, aggregate interlock is not effective in stress control either at the joint edges or corners. In pavements with closely spaced contraction joints and with no expansion joints, aggregate interlock can be expected to cause a reasonable reduction in the corner stresses, but is an uncertain means of stress control at the joint edges.

DISCUSSION

Mr. J. N. Heltzel, *The Heltzel Steel Form and Iron Company:* Do engineers agree that faulty doweling has been primarily responsible for joint failures in concrete highways—that expansion and contraction joints in modern highways and airports are necessary to control cracking in the many miles of concrete ribbon—that joints serve as safety links and must be adequately bridged or doweled?

Early railroad builders recognized that contraction and expansion of rails had to be reckoned with—that rail joint required adequate bridging, which was constantly improved to meet demands of heavier traffic

loads. Highway engineers, on the other hand, have not improved the doweling of joints to withstand increased heavy present-day traffic.

Without logical explanation by engineers and constructors—including manufacturers of dowel installations, joints in concrete highways and airports, as far as dowels are concerned, have been a source of disappointment and failure. Some engineers have recommended their discontinuance; however, progressive engineers will never surrender to the elements; they will build to overcome failures.

Modern highways and airports must be built with ample factors of joint safety. This can be accomplished by proper design and installation of contraction joints and application of adequate load transfer elements to support the joints to such an extent that a continuous homogeneous structure is assured. This cannot be achieved by use of the old, so-called common dowels, which have constantly and definitely failed to serve as intended; even when properly installed, they become a liability rather than an asset to the structure.

The common dowel with or without sockets bears on the raw concrete. It usually freezes in place so that initial contraction of the slabs is resisted, resulting in funneling around the dowel. This funneling insures a pumping joint. The dowel corrodes and fails to function so that the tensile pull through the dowels is greater than the strength of the inner slab, resulting in tearing of the slab. At this stage the dowel fails to respond to expansion of the slabs, which jams the dowel into its setting and 'dead-ends' the common dowel.

High costs of labor and materials demand that more dependable load transfer elements be installed in future structures. These units must not resist free contraction of the concrete during initial setting; they must avoid bearing directly on the concrete at face of joint. A multiplicity of anchorage must engage the inner body of the slabs above and below the center of slab thickness to absorb stresses caused by traffic loads and transmit these stresses direct to the load transfer bearing at face of joint.

Although, perhaps, 40 long years of guessing and experimenting resulted in failures and disappointments, the vexatious problem had already been solved; engineers needed only to refer to their text books—the formulas were all there. There is no guess work about those tables of dowel shear, plate bearing or modulus of rupture of concrete.

The following observations and suggestions may be constructive:

1. The thicker the slab—the heavier should be the dowels with spacing equal to approximately twice the slab thickness when properly designed load transfer elements are installed.

2. The spacing of load transfer elements must be consistent with dowel bearing anchorages, slab thickness and dowel shear.

3. Sockets on dowels should be positioned alternately—reversible over the dowel.

4. One end of a dowel may be fixed or anchored in one slab and cantilevered across the joint into the socket in the adjacent slab.

5. Dowels or sockets should not bear directly on concrete at face of joint without stress absorbing anchorages extending into body of slab.

6. Dowel sockets should extend to face of joint; they should be made moisture-proof by filling with suitable long-life lubricant.

7. Conventional dowels never have been free of voiding directly under the dowel adjacent to the expansion joint due to slumping of the concrete; on this account bearing or shear plates or load distributing anchorages are necessary.

8. Bearing or shear plates on faces of joints with stress distributing anchorages extending back into the body of the slab will absorb traffic loads and transmit these loads to the dowel bearings at faces of joints.

9. Dowels should be installed parallel to the top surface of the slab rather than at right angles to the expansion strip.

10. Properly installed dowels must be parallel to one another.

11. Free ends of dowels should be chamfered slightly to insure freedom of movement during initial contraction of the concrete sections.

12. Urging slabs apart during initial contraction of concrete is desirable to avoid disturbing the anchorages of load transfer bearing elements.

13. Corrosion and rusting of dowel bars is destructive and limits the functioning and life of dowel bars. They should be shielded with a non-corrosive member.

14. Contraction crack control has been

successfully achieved; load transfer elements, however, are recommended to the extent of approximately 60 percent of transference elements, installed in expansion joints in the same project because the space to be bridged in a contraction joint is minute at practically all times of the year.

15. A sub-dummy joint should be provided directly under the surface dummy joint in slabs over 8 in. thick to promote vertical fracture and prevent diagonal cracking. The depth of the surface dummy should be 20 percent of slab thickness and the height of the sub-dummy should be 15 percent of the slab thickness. A wood strip approximately ½ in. thick may serve for the sub-dummy. The strip will absorb moisture and urge the slabs apart, thereby relieving some of the tension during the drying or shrinking of the slabs. A water seal may be desirable also for the sub-dummy to prevent capillary action on the subgrade.

16. Without tension-relief by installation of transverse dummy or other suitable joints, the endless, non-jointed concrete ribbon will progressively exceed its elastic limit during the drying-out period, which will result in formation of minute fissures throughout substantially the entire length of the ribbon of concrete. The most pronounced of these concealed fissures will develop into irregular open cracks at points of least resistance transversely of the slab while the lesser fissures remain as a menace to the structure constantly developing into slab disintegration.

17. The maximum thickness of expansion strip should be ¾ in.; greater thickness of strip permits excessive shifting of the slab over the subgrade. Better results would be obtained by installing two joints with ½-in. strips.

18. A properly finished and edged expansion joint will obviate contact of top edges of slabs. Slabs should contact first at the bottom to relieve pressure at top surface edges and prevent spalling and upward heaving.

19. Outside corner breaking may be reduced by providing wider space between abutting slabs at ends of expansion joints.

20. Engineers should specify the ultimate bearing in tons for load transfer devices for given slab thicknesses. It would then be the manufacturers' responsibility to meet those requirements.