Test Project Constructed Utilizing the Contraction Joint Design

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● FROM 1937 to 1952, the standard reinforced concrete pavement design specified slab lengths varying through these years from 75 ft to 100 ft between expansion joints with grooved warping joints at intervals of about 25 ft. The wide expansion joints adversely affected riding quality and created a maintenance sealing problem. A slab design was developed to test the performance of contraction joints and to confine the joint opening to controllable limits within which known sealers would remain effective.

In 1952, a section of concrete pavement, approximately 1,800 ft long between expansion joints, was constructed on the Wilbur Cross Highway in Vernon, Connecticut. Intermediate contraction joints with load transfer provision were installed at 37 ft-4 in. intervals, six different types of load transfer assemblies, in groups of eight, being used.

All joints were plugged, in preparation for measurement of joint movement, temperature wells were installed and reference lines established at the ends of the test section to measure longitudinal end movements.

Joint movement measurements taken at frequent intervals, with particular emphasis on temperature extremes, indicate that movement in the contraction joints is not uniform even within the individual groups of load transfer assemblies.

In 1955, blocks were removed from the pavement, each containing one dowel unit. The varying amounts of force required to pull apart these blocks indicate differences which are, however, not fully in accord with the measured movement in the same joint in the opposite lane. Destruction of these blocks to permit removal and inspection of dowels revealed some early rust formation on the load transfer dowels, particularly in the center portion within the contraction joint.

The paper represents a progress report. Observations and tests, including roughometer studies, leading to an evaluation of the test section will be continued over an extended period.

In the earlier history of reinforced concrete pavement slab design in the State of Connecticut, transverse expansion joints were relied upon to compensate for the entire anticipated longitudinal movement in the slabs. Successive standard designs specified slab lengths varying from 60 ft to 100 ft, with corresponding expansion joint widths of ½ in. to ¾ in. In 1937, intermediate planes of weakness of the hand formed, surface groove type were introduced, at intervals of about 25 ft, to control random transverse cracking. In 1952, machine sawing of the grooves was substituted for hand forming to improve riding qualities.

Load transfer devices at the transverse expansion joints were first introduced in 1934. The early type, without a subgrade support frame, was superseded in 1939 by new types provided with support frames. On a limited number of projects of early design, mat reinforcement was interrupted at the planes of weakness, thus creating contraction joints, but in later designs the reinforcement was continuous throughout the entire slab, thus creating so-called "warping" or "hinge" joints.

In 1950, the then current slab design, 97 ft-4 in. between expansion joints with three intermediate grooved warping joints, was found unsatisfactory. The ¾-in. expansion joint width, which increased to 1 in. by contraction of slabs at temperatures 49 F below that at which the pavement was laid, resulted in poor riding qualities besides requiring frequent re-sealing.

Our present design which provides for a contraction joint spacing of 40 ft with expansion joints only at bridges, concrete paved intersections and at other critical locations, was adopted October, 1955.

The planned test section, a part of a paving contract for the eastbound roadway of the Wilbur Cross Highway to be constructed in 1952, necessitated development of load transfer assemblies suitable for contraction joints. Six manufacturers submitted designs and agreed to furnish contraction joint assemblies for the test installation. A contraction joint spacing of 37 ft-6 in. was used in the test section to accommodate the standard length reinforcement mat. With this spacing it was expected that the hand formed groove, ⅜ in. to ½ in. wide, would remain more effectively sealed and would result in better riding qualities.

The test section (Figure 1), which is 1,830 ft in length, consisted of two lanes of reinforced concrete 12 ft wide and 8 in. in depth, placed over 12 in. of gravel subbase throughout its full length. The gradation requirements of the subbase gravel were: 100 percent passing 5-in. sieve; 5-30 percent through the No. 40 and 0-10 percent passing the No. 100 sieve. The 100 mesh material had zero plasticity. The gravel was placed in 6-in. layers and rolled with 10-ton rollers. No compaction tests were performed on this material.

With the exception of the test section, the remainder of the pavement consisted of the design current at that time, which was an expansion joint spacing of 97 ft-4 in. with three intermediate sawed warping joints approximately equally spaced. While a bituminous cellular type of joint filler was normally used in the expansion joints, an exception was made in the six transverse expansion joints nearest to both ends of the test section where ¾-in. wood joint filler was specified and installed.

On August 11, 12 and 13, 1952, six different groups of contraction joint assemblies were installed in the outer lane of the eastbound roadway. Each group consisted of eight assemblies of the same type and they were installed in consecutive order. The same assemblies were used and identical order of installation was maintained in the paving of the inner or passing lane. The contraction joint spacing of 37 ft-6 in. was established by the available lengths of mat reinforcement. The concrete mix proportions were 1:2:4 with 4 to 5 percent entrained air. Concrete temperatures at the time of placing the south lane varied from 85 F to 90 F with air temperatures varying from 86 F during the day to 64 F through the night. Waterproof paper was used to cure the concrete pavement for a period of 14 days.

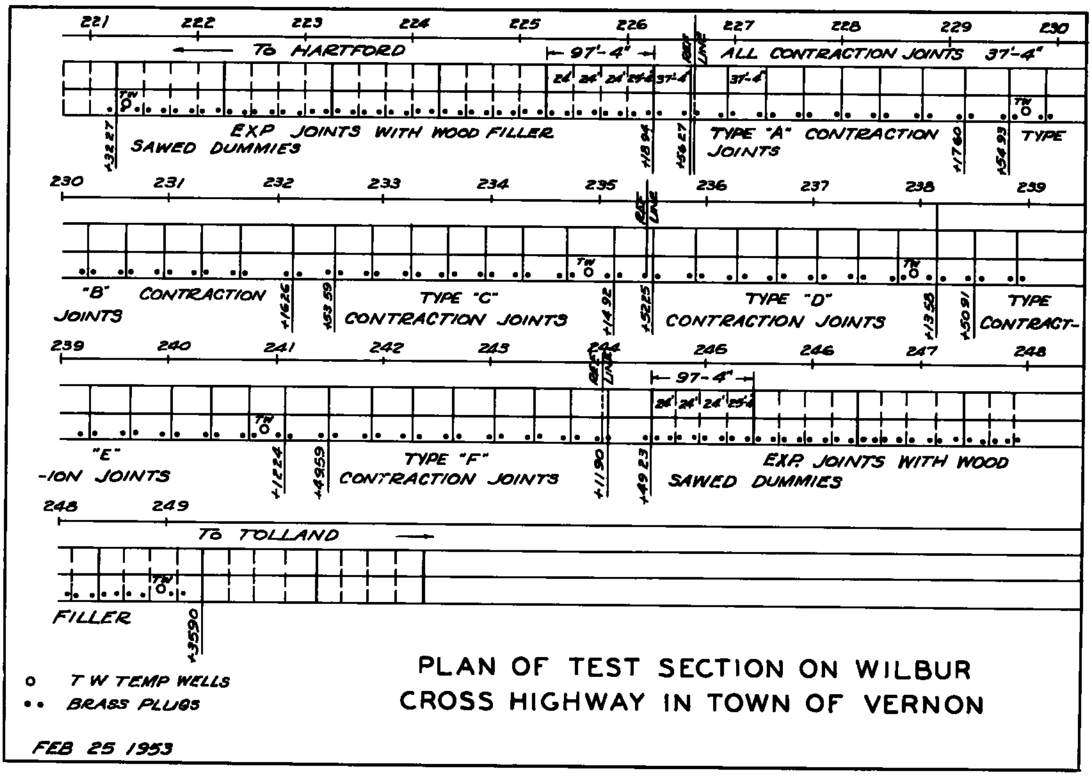

Figure 1. Schematic layout of test section.

The six different assemblies installed have been designated as Type A, B, C, D, E and F in the order in which they were installed.

The Type A contraction joint assembly (Figure 2) consisted of ten $1\frac{1}{2}$- by $\frac{7}{16}$ by 12-in. steel beam type dowels on 15-in. centers, sliding in a channel track. The sliding end was coated with a bituminous mastic, although when installed in the outer lane this mastic was omitted through a misunderstanding. However, the dowels installed in the inner or passing lane were coated.

The Type B contraction joint assembly (Figure 2) consisted of twelve 1- by 18-in. steel round dowels on 12-in. centers. One-half of this type assembly had $7\frac{1}{4}$- by 0.115-in. steel divider plates and the other half had $4\frac{7}{8}$- by 0.125-in. steel divider plates. The top of the steel plates was $\frac{1}{2}$ in. below the surface of the concrete. The only coating on these dowels was a rust inhibitor.

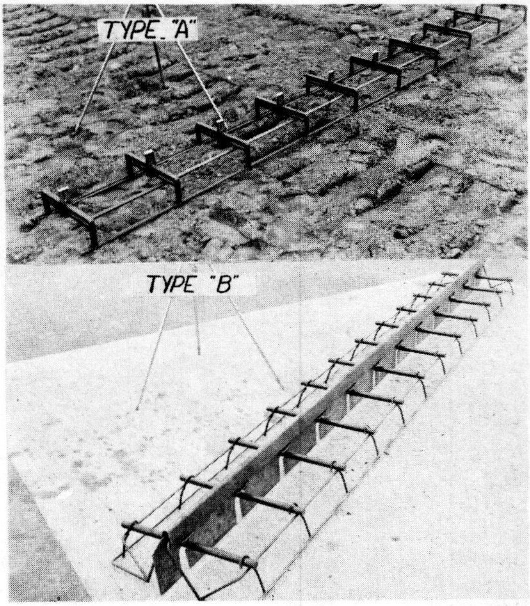

Figure 2. Type A and B assemblies.

The Type C assembly (Figure 3) consisted of twelve 1- by 18-in. steel round dowels on 12-in. centers with 5- by 0.078-in. steel divider plate set $2\frac{1}{2}$ in. below the surface of the concrete. The greasing of these dowels in the travel lane was omitted through a misunderstanding.

The Type D assembly (Figure 3) consisted of twelve malleable iron units on 12-in. centers. The female end contained a $\frac{3}{4}$-in. ID, 16-gauge, Type 410 or 430, stainless steel tube. The male end consisted of a tight fitting, Type 410, $\frac{3}{4}$- by $4\frac{3}{8}$-in. stainless steel dowel with sliding contact over a length of $2\frac{9}{16}$-in. This assembly did not contain a steel plate.

The Type E assembly (Figure 4) consisted of twelve malleable iron, hinge-type units on 12-in. centers. This assembly also contained a steel divider plate $3\frac{1}{2}$- by 0.101-in. set $2\frac{5}{8}$ in. below the concrete surface.

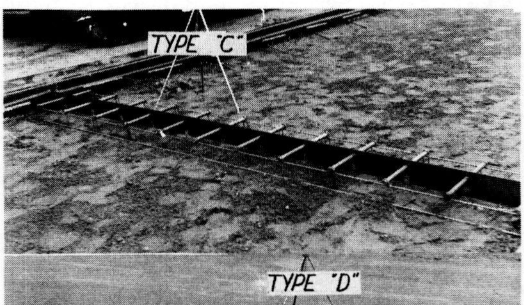

Figure 3. Type C and D assemblies.

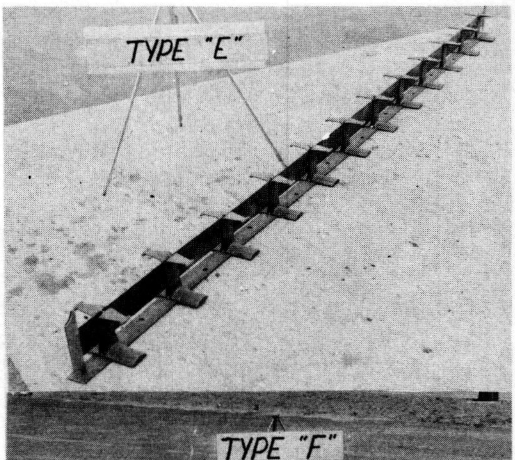

Figure 4. Type E and F assemblies.

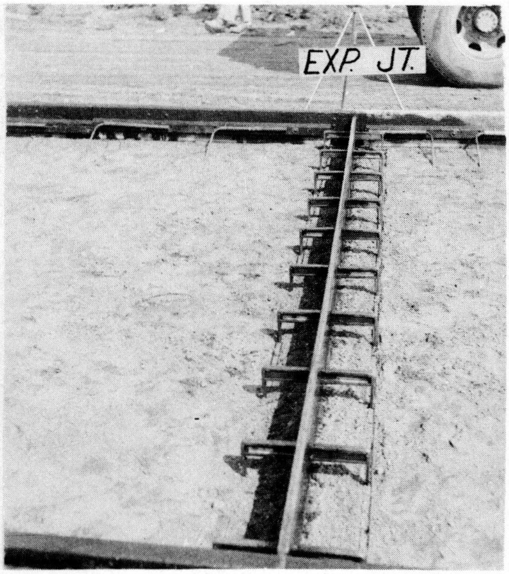

Figure 5. Expansion joint assembly.

The Type F assembly (Figure 4) consisted of twelve 1- by 6-in. cold rolled steel dowels on 12-in. centers. This assembly did not contain a steel plate.

The expansion joint assembly (Figure 5) consisted of ten 1½- by 7/16- by 12-in. beam type dowels on 15-in. centers sliding in a channel track. The sliding end was coated with a bituminous mastic and enclosed with 24-gauge sheet steel. In this case, the joint filler was ¾-in. redwood.

Immediately after the finishing of the concrete was completed, brass plugs were placed at each side of the contraction joints along the outer edge of the traveled lane. The first measurements across the joints were taken on the following day and pavement temperatures, obtained by the use of oil wells which were also installed in the pavement at the same time as the brass plugs, were recorded. Daily both edges of concrete were carefully examined for visible cracking (see Figure 6).

Twenty-five percent of the 48 contraction joints showed visible cracking at the edge of pavement one day after placing of concrete. Fifty percent of the joints showed visible cracks seven days after placing of concrete, and at 28 days all joints were cracked through. The eight contraction joints with the Type E or hinged assembly were all visibly cracked the day after placing of concrete. The eight contraction joints using the Type A assembly required 28 days for visible cracking of all joints. Oddly enough there appears to be very little difference in the effect on visible cracking of the joints between the full depth steel plate and the half depth steel plate of the Type B assembly. This would indicate that joint cracking depends more on the time required to break the

PAVEMENT TEMPERATURE RANGE 60°-90°

SOUTH LANE ONLY				DAYS AFTER CONCRETE WAS PLACED											
JOINT DESCRIPTION	DATE POURED	PLATE DEPTH	JOINT NO'S	1 da	3 da	6 da	7 da	12 da	13 da	14 da	15 da	22 da	23 da	24 da	28 da
TYPE "A" Ⓐ	8/11/52	NONE					3,6	4,5,8							1,2,7
TYPE "B"	8/12/52	* 4⅞", 7¼"	"			1,2,5,6,8			3,7	4					
TYPE "C" Ⓐ	8/12/52	5"	"		8	2,5			1,3,4	6			7		
TYPE "D"	8/12/52	NONE	"		3,7				1,5				2	4,6,8	
TYPE "E"	8/13/52	3½	"	1-8											
TYPE "F"	8/13/52	NONE	"	2,4,6,8				1,5			3	7			
	TOTAL			12	3	7	2	5	7	2	1	1	2	3	3

* JOINTS #1-#4 WITH 4⅞" PLATE JOINTS #5-#8 WITH 7¼" PLATE

Ⓐ DOWELS WERE NOT COATED WITH LUBRICANT

JOINT DESCRIPTION	DEPTH OF JOINT GROOVE	EDGE CRACKING TYPE (1) TYPE (2)			
TYPE "A"	2" to 2⅜"	6 ℄ JTS TYPE (1)	2 ℄ JTS TYPE (2)	7 EDGE JTS TYPE (1)	1 EDGE JTS TYPE 2
TYPE "B"	¼" to ⅞"	8 ℄ JTS TYPE (1)		8 EDGE JTS TYPE (1)	
TYPE "C"	1¾" to 2"	8 ℄ JTS TYPE (1)		8 EDGE JTS TYPE (1)	
TYPE "D"	1¼" to 2"	6 ℄ JTS TYPE (1)	2 ℄ JTS TYPE (2)	5 EDGE JTS TYPE (1)	3 EDGE JTS TYPE (2)
TYPE "E"	1½" to 2"	8 ℄ JTS TYPE (1)		6 EDGE JTS TYPE (1)	2 EDGE JTS TYPE (2)
TYPE "F"	2" to 2¼"	4 ℄ JTS TYPE (1)	4 ℄ JTS TYPE (2)	8 EDGE JTS TYPE (1)	

Figure 6. Approximate cracking schedule of contraction joints.

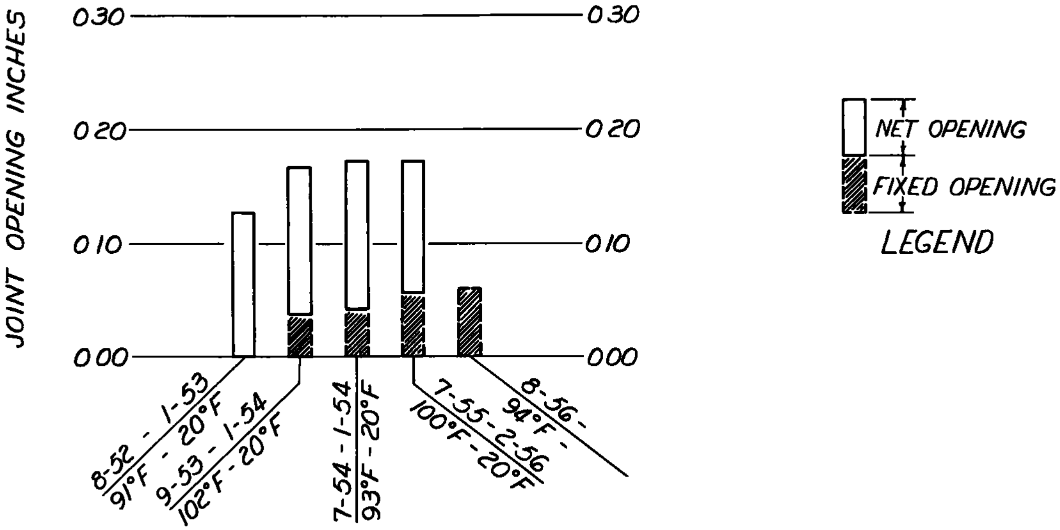

Figure 7. Permanent opening and seasonal opening of contraction joints.

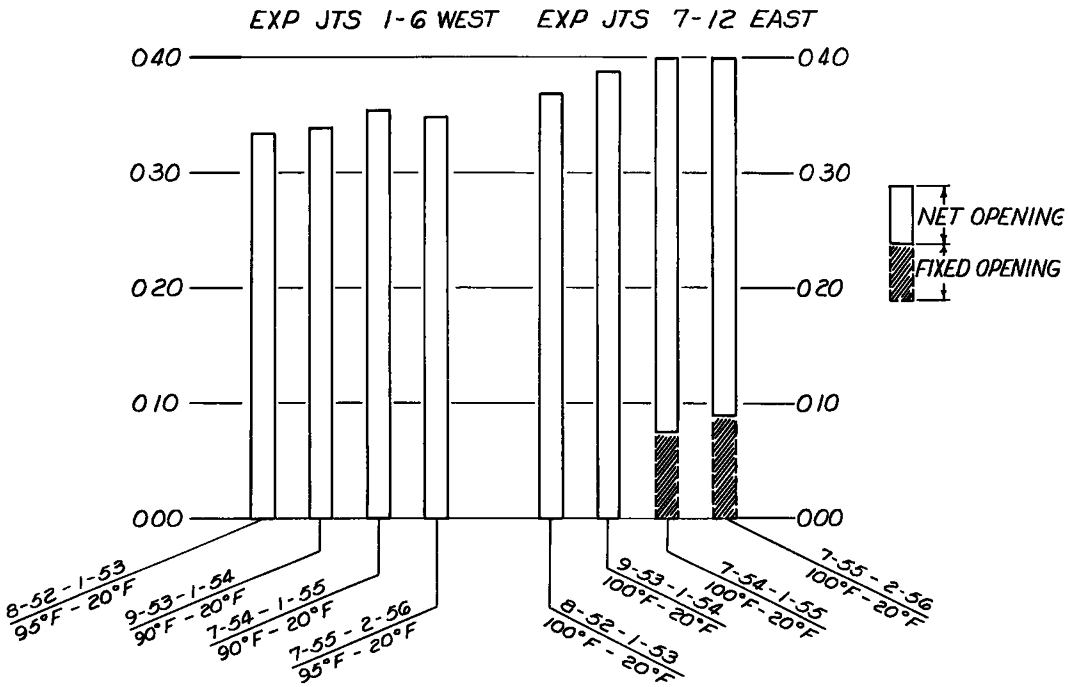

Figure 8. Seasonal expansion joint opening.

bond of the concrete around the dowel than the depth of plate used to weaken the joint. However, of the 24 joints visibly cracked, on the seventh day, 16 contained steel divider plates.

It is quite probable that the actual cracking schedule of the joints does vary from that of visible cracking due to dowel restraint maintaining a tightly closed crack. Since the instrument used to measure the joint openings is only accurate to ±0.002 in., it was felt that more reliable information would be obtained by observing the visible cracking.

Over a four-year period, the contraction joints have shown a gradual permanent opening at an average rate of 0.015 in. each year. In July, 1955, the average permanent opening for the 48 joints was 0.055 in. (Figure 7). Between July, 1955, and February, 1956, for a temperature range between 100 F and 20 F, the average total opening (permanent opening plus actual movement) was about 0.17 in., which indicates the net actual movement was about 0.11 in. (0.17 in. - 0.06 in.).

The average opening (Figure 8) for the six expansion joints at the westerly end of the contraction joint section between 95 F and 20 F has been about 0.35 in. Early in 1956, the average permanent opening of the warping joints was 0.017 in. The average opening for the six expansion joints on the westerly end has been somewhat erratic. For the first two years the average opening between 100 F and 20 F was about 0.38 in. During the two following years, there has been considerable permanent opening at some of the expansion joints such that the over-all average indicates at 100 F an opening of 0.08 in. per joint and at 20 F the average opening is now 0.40 in. In early 1956, the average permanent opening of the warping joints at this easterly end was 0.014 in.

It is quite apparent that either the load transfers or infiltrated material are preventing these expansion joints from closing. This may account for the lesser displacement measured at the easterly end of the contraction joint section compared to that measured at the westerly end, as shown elsewhere herein.

The over-all average movement of the concrete pavement at the contraction joints has been fairly consistent. Movement which has been restrained at some joints has been compensated for by greater movement at other less restrained joints. However, there has been considerable variation in joint movement within some groups using certain types of joint assemblies. The Type A assembly has shown considerable variation in joint opening, whereas the Type D assembly has shown the least variation. In all fairness, it is pointed out again that through a misunderstanding the dowels of the Type A assembly were not greased when installed in the south or travel lane.

In Figure 9 it will be noted that for the Type A assemblies joint 1 has consistently exhibited comparatively very little change in joint opening over a temperature range from 20 F to 100 F. The lack of movement at joint 1 is compensated for by an abnormally large amount of movement at joint 2. In January, 1955, joint 2 was open 0.37 in. as against the over all average of 0.149 in. for the 48 joints at a pavement temperature of 24 F. Figure 11 shows the condition of the rubber asphalt joint seal in joint 2 at the

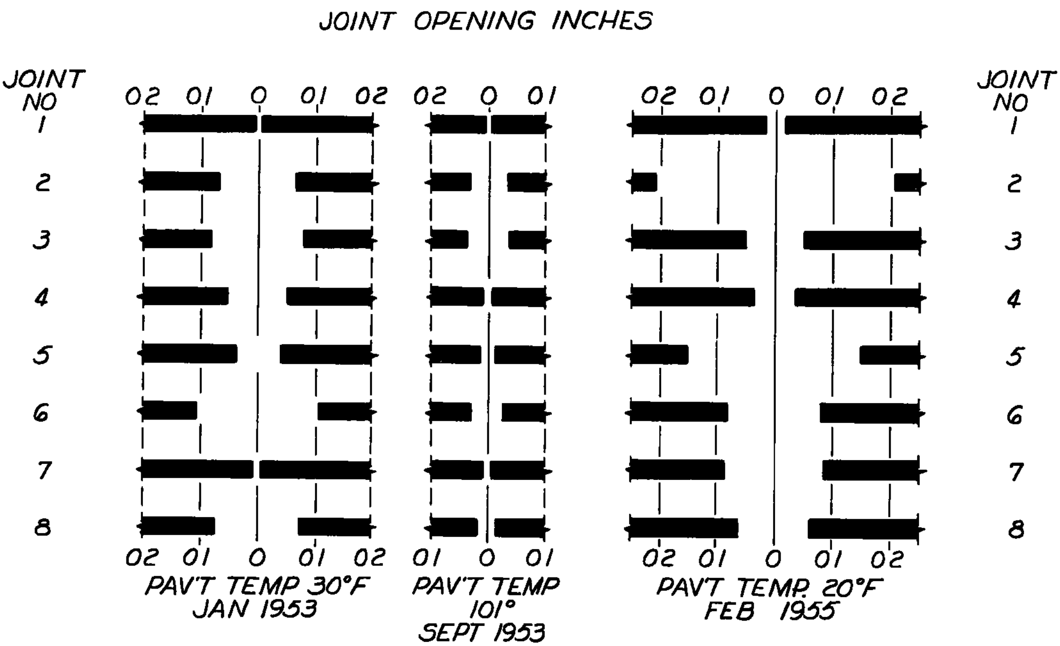

Figure 9. Relative openings at temperature extremes of joints containing Type A assembly.

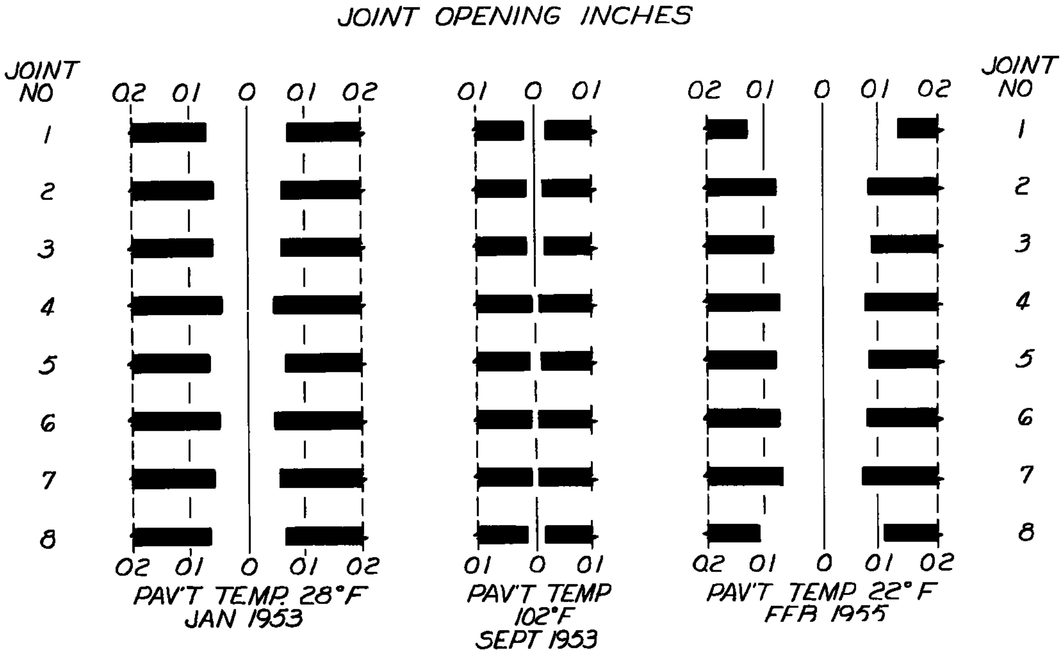

Figure 10. Relative openings at temperature extremes of joints containing Type D assembly.

same time. The joint seal in joint 1 appears to be in good condition. It is becoming increasingly evident that there is a very definite limit in the ability of the joint seal to withstand horizontal or vertical or a combination of both movements at a joint. From the examination of considerable mileage of concrete pavement in Connecticut, it has become evident that if the joint seal in an expansion joint is in good condition, it is only because there is little or no movement at the joint.

The joints containing the Type D assembly (Figure 10) have exhibited the most uniform movement. Movement at the end joints has been greater than at the intermediate joints due to greater restraint to movement in the Type C and E assemblies immediately preceding and following the Type D assemblies.

The joints containing the Type B assembly closely follow the Type D assembly for uniformity of opening. The dowels of the Type C assembly are restraining movement at some of the joints although here again the omission of a bond breaking agent may be affecting the joint opening. The dowels of the Type F assembly as well as the hinge type load transfer of the Type E assembly are performing satisfactorily to date, insofar as freedom of movement is concerned.

End movement of the 1,830 ft contraction joint section has been very slight. In July, 1955, with a pavement temperature of 100 F, the west end moved 0.328 in. west and the east end moved 0.175 in. to the east. At the same time the sum total of the permanent openings at the 48 joints was equal to 2.648 in. Assuming that the concrete is completely restrained at 100 F, the net restrained movement is 2.145 in. (2.648 - 0.328 - 0.175), which indicates a compressive stress of about 500 psi, neglecting the effect of plastic flow.

The end movement of 0.328 in. to the west and 0.175 in. to the east has been absorbed in varying amounts along the line of expansion joints preceding and following the contraction joint section.

Measurements of joint faulting were begun in February, 1954, or about 1½ years after the pavement was placed. In 1954, the average daily traffic count of heavy two-axle trucks was 150 and for three or more axles, 750. While no count is available for 1956, the above figures appear, from actual observation of the traffic on this highway, to be very conservative. The instrument or inclinometer arrangement used to deter-

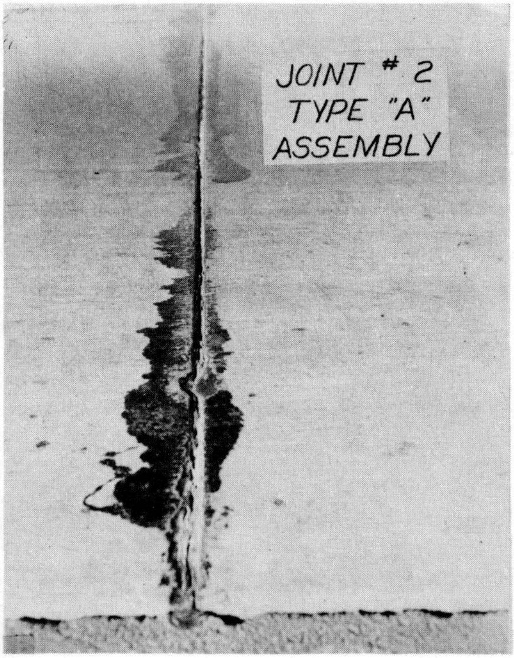

Figure 11. Condition of joint seal in Joint 2.

mine the faulting was simply a steel bar about 15 in. long with a spirit level and Ames dial attached to the bar. A movable pointer at one end of the bar accommodated variations in the width across the joints and a pointer fixed in the horizontal direction, but free to move in the vertical direction, permitted leveling of the bar. Brass plugs with drill holes were placed in the concrete at each joint and placing the pointers at each end of the bar in the drill holes, the amount of vertical movement at one end required to bring the bar to a level position was measured by the Ames dial through a connecting linkage with the leveling device. Readings taken at different times at the same joints were compared and any differences in vertical movement required to level the bar were recorded as the amount of joint faulting. Figure 12 shows joint faulting at the joints using the Type E and Type F assemblies.

Measurements of joint faulting taken in February, 1956 (Table 1), with pavement temperatures of 25 F and average joint openings for each group of contraction joints varying from 0.141 in. to 0.171 in., show faulting in the group of contraction joints using the Type F assembly to be the greatest. In this group the average joint faulting was 0.038 in., with a range of 0.008 in. to 0.088 in. The least amount of faulting occurred in the group using the Type D assembly. The average faulting for this group was 0.004 in., with a range of 0.000 in. to 0.010 in.

In July, 1955, it was decided to cut out from each group a block of concrete containing one of each of the various load transfers. This was done to determine the con-

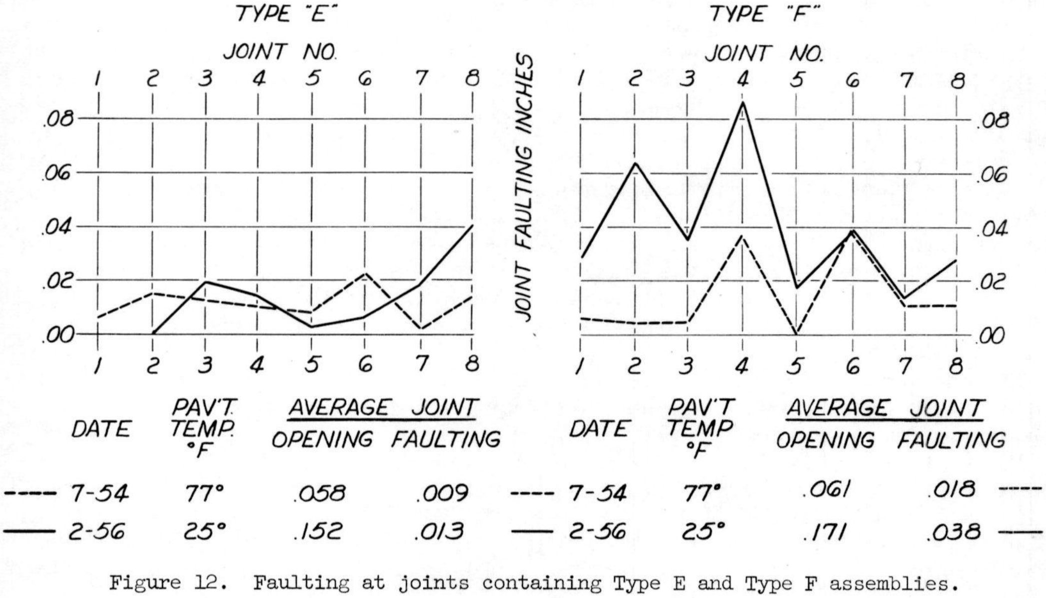

Figure 12. Faulting at joints containing Type E and Type F assemblies.

TABLE 1

Assembly Type	Average Faulting	Range - Faulting	Average Joint Opening
D	.004 in.	.000 in. - .010 in.	.16 in.
C	.006 in.	.002 in. - .015 in.	.14 in.
E	.013 in.	.000 in. - .041 in.	.15 in.
B	.016 in.	.003 in. - .033 in.	.17 in.
A	.016 in.	.002 in. - .036 in.	.17 in.
F	.038 in.	.008 in. - .088 in.	.17 in.
Exp. Jts. West	.027 in.	.018 in. - .066 in.	.31 in.
Exp. Jts. East	.035 in.	.012 in. - .054 in.	.37 in.

dition of the various types of dowels used and the degree of restraint offered by the various dowels. Unfortunately, these blocks had to be removed from the left edge of the inner or passing lane, since removal of these blocks from the right edge of the outer lane would have necessitated, also, the removal of the brass plugs used to measure joint movement and faulting. It should be noted that all dowels installed in the passing lane were greased or coated with a bond breaking agent of some sort. A cross section of the joint crack with and without the steel divider plate is shown in Figure 13. The cracking of the contraction joints without steel plates is fairly typical.

Cracking more or less follows a path around the aggregate indicating that, at the time of cracking, the mortar strength was less than that of the coarse aggregate. Where the steel plates were used to insure speedy cracking at the joint, no aggregate interlock is available. The pavement temperature at the time the concrete blocks were removed from the pavement was between 90 F and 95 F. While the joint seal appeared to be tightly bonded to the concrete, considerable fine silty material was found in some cases between the joint seal and the concrete. Samples of the joint seal were removed from the joints containing the Type C, D, and F assemblies to determine the amount of foreign material embedded in the seal. The percentage of ash, after dissolving the joint seal samples and igniting the insoluble portion, was found to vary from 10 to 40 percent. No values of the ash in the original seal are available but there appears to be an indication of considerable foreign material either embedded in or attached to the joint seal.

Each concrete block containing a different type of load transfer was subjected to sufficient loading to pull the dowel out of the concrete. The loads required to open each joint 0.1 in. were noted.

As a basis for comparison, the average net joint movement measured in the field at temperatures of 100 F and 25 F for the respective types of assemblies installed in the outer lane has been plotted alongside of the loads required to open the concrete blocks cut out of the inner or passing lane (Figure 14). The Type D load transfer required the least amount of load (500 lb) to open the joint 0.1 in. The average net movement (0.125 in.) for the group of joints in the outer lane containing this assembly was greater than that of any of the other assemblies. The Type C load transfer required the greatest load (8,000 lb) to open the joint 0.1 in. The group of joints in the outer lane containing this assembly also showed the least average net movement (0.09 in.). To this extent only was there any correlation between the amount of load required to open the joint 0.1 in. and actual movement as measured in the field. It is entirely possible that even such correlation as was obtained may have been due to chance since surely each of the twelve dowels in a joint will be subjected to varying amounts of moisture, salt brine and any other factors which will promote

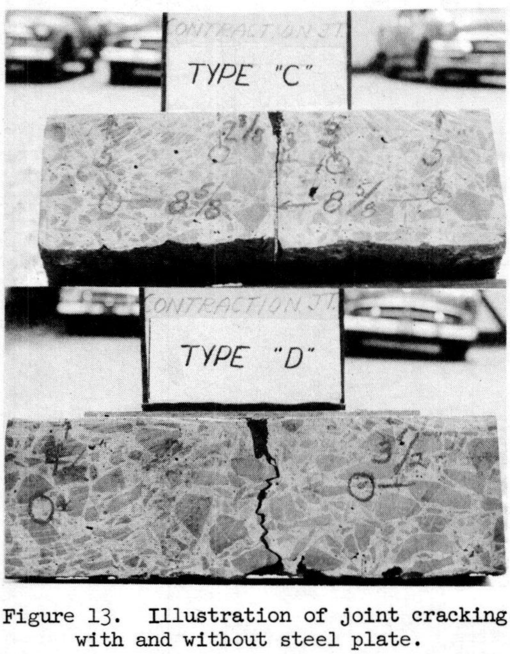

Figure 13. Illustration of joint cracking with and without steel plate.

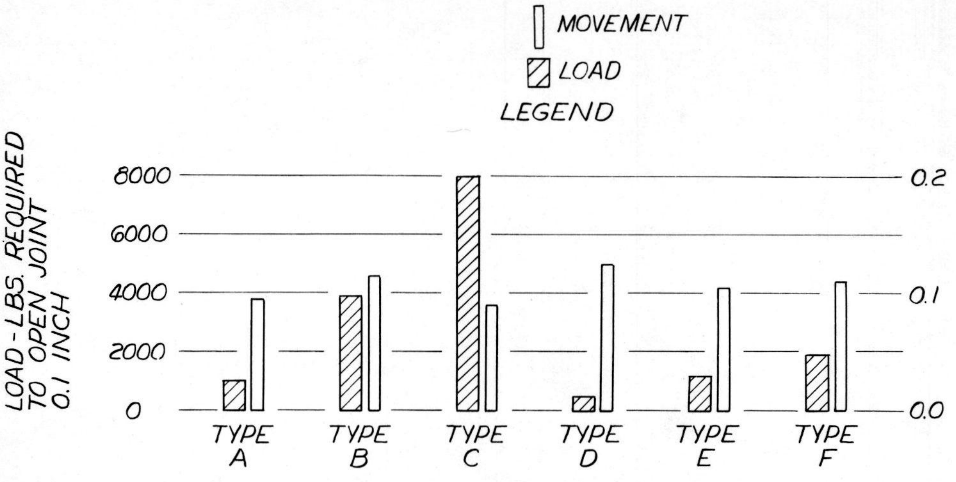

Figure 14. Loads required to open the joints 0.1 in.

rust. In which case, the load required to move any one dowel may be different from another. It is certain that if the 8,000 lb load required to open the joint with the Type C load transfer was representative of all twelve dowels in the joint, no such movement as 0.09 in. would have been obtained. The Type B assembly removed from the inner or passing lane required 4,000 lb to open the joint 0.1 in. Assuming this load to be representative for each of the twelve dowels in the joint, there should be little or no movement at the joint for slab lengths of 37.5 ft. This is based on the assumption that the joints immediately preceding and following are free to move and a coefficient of friction of 1.0. In other words, to develop a 4,000 lb pull at the joint, it would require a slab length of 40 ft each side of the joint. Field measurements of joint movement in the outer lane for the group of joints using the Type B assembly do not substantiate this condition.

The Type A assembly required a load of 1,000 lb to open the joint 0.1 in. To develop this pull requires at least 10 ft of concrete each side of the joint with a temperature drop of 1 deg. Measurements across the joint from which this assembly was cut, but in the outer lane, show a net joint movement of only 0.019 in. as against the average net movement for the joints of all assemblies of 0.131 in. through a temperature drop from 93 F to 20 F. The measurements indicate a condition of very high restraint which does not agree with the condition indicated by the pull-apart test. Therefore, the results of load tests on one dowel, insofar as measuring restraint is concerned, are questionable.

After the pull-apart tests were completed, the dowels or load transfer assemblies were removed from the concrete (Figure 15). All dowels showed some signs of rusting at the joint opening. There was very little or no evidence of a bituminous or grease coating remaining on any of the dowels. The dowel of the Type A assembly probably shows the greatest degree of rusting with a visible loss in metal particularly along the bottom edge of the dowel for a distance of $1\frac{1}{4}$ in. from the joint face along the sliding end of the dowel.

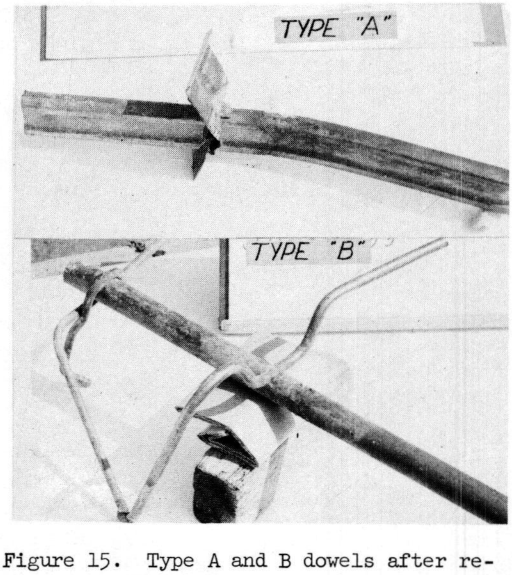

Figure 15. Type A and B dowels after removal from concrete.

The dowel of the Type B assembly also shows pronounced rusting on the underside of the dowel at the joint opening. The accumulation of rust on the dowel at the joint face has caused an increase of 0.027 in. in the diameter of the dowel. This may account for the continuing increase in the permanent opening of joints at high temperatures.

The dowel of the Type C assembly (Figure 16) shows severe rusting at the joint face and extending with decreasing severity for several inches on both sides of the joint. Over a distance of approximately $1\frac{1}{4}$ in. from the joint opening along the sliding end, there is, due to rust, an increase in the diameter of from 0.006 to 0.009 in. This probably accounts for the extremely high load (8,000 lb) required to move this dowel 0.1 in. from the concrete. The steel plate used in this assembly is very badly rusted.

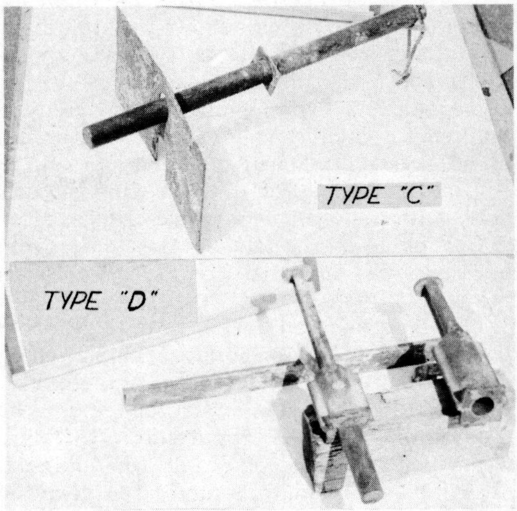

Figure 16. Type C and D dowels after removal from concrete.

The dowel of the Type D assembly has no rust on either the male or female ends due to stainless steel construction. However, there is rust forming at the shoulder of the malleable iron casting from which the male end protrudes. While this rust does not affect the sliding of the dowel, it does prevent the concrete joint from closing during hot weather.

The load transfer unit of the Type E assembly (Figure 17) is rusted at the joint opening. However, due to its hinge action, the joint movement is not affected. The steel plates incorporated in this assembly are severely rusted which may also account for the permanent opening of these joints.

The sliding end of the dowel of the Type F assembly (Figure 17) is free from rust at this time. However, rust is forming around the dowel where the steel wire hangers encircle it.

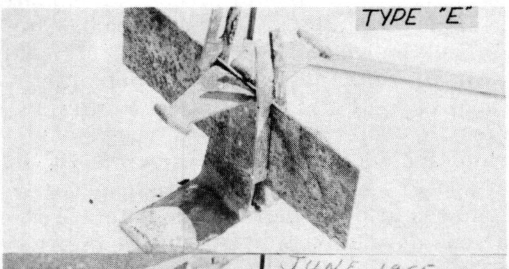

Figure 17. Type E and F dowels after removal from concrete.

SUMMARY

In general, the data obtained and observations made to date in the contraction joint test section indicate the following:

1. There is no certainty of a uniform schedule of visible cracking even when steel divider plates are used. It is true all eight joints with the Type E assembly which includes a $3\frac{1}{2}$-in. steel plate did crack on the first day after placing but it is felt that a large factor in this event was the hinge type of load transfer which for very slight movements did not offer any appreciable resistance to slab end movement. However, the joints containing the Type B assemblies, four of which contained $7\frac{1}{4}$-in. plates and four of which contained $4\frac{7}{8}$-in. plates, showed no visible cracks until the sixth day when five of the eight joints were noted as visibly cracked and in 14 days all eight

joints were cracked. Four of the eight joints with the Type F assembly, which contained no steel plates, showed visible cracking in one day, while the remaining four joints required up to 22 days before visible cracking was observed. Five of the eight joints containing the Type A assembly were cracked at 12 days and seven of the joints containing the Type C assembly were cracked at 14 days, despite the omission of dowel lubricant. Oddly enough four of the joints containing the Type D assembly did not show signs of cracking until the 23rd and 24th days. This assembly contains no steel divider plate and up to the present time the dowels in this assembly are functioning apparently with the least restraint. However, it does appear from the observations made of the cracking on the test section that, over a given period of time and for an equal number of joints, there will be more joints cracked containing the steel plate than without the steel plate.

2. That each summer, to date, there has been an increase in the permanent opening of the contraction joints. This opening has gradually increased to an average of 0.06 in. per joint at the end of four years. Rust, either on the dowel or certain portions of the dowel assembly, appears to be a factor to be considered in the refusal of these joints to close completely. Undoubtedly the permanent opening of these joints will eventually become stable when compressive pressures become great enough to overcome those factors tending to keep the joint open.

3. That the rubber asphalt joint seal has not performed in the shorter slab lengths as satisfactorily as expected. This might be attributed in part to the fact that the actual joint opening is greater than was originally estimated.

In considering the design of the test section, it was estimated the seasonal joint opening would be in the vicinity of 0.12 to 0.15 in. for a temperature differential of 80 F. Actual measurements show the average total opening per joint to be 0.17 in. However, this includes a permanent opening of 0.06 in., which really leaves 0.11 in. of actual movement per joint. There have been individual cases where the total joint opening was found to be as much as 0.40 in. and the seal condition was obviously unsatisfactory in these instances. Where the joint openings have been at or near the average of 0.17 in., the seal, in some instances, was in good condition whereas in others it has been in poor condition. Observations were made of the joint seal condition at temperatures between 10 F and 25 F. Photographs were taken of the joints at this time and again in July when temperatures were between 90 F and 100 F. At the latter time, the joint seal condition appeared quite good in all joints due to combination of warm weather and the kneading action of traffic.

4. In general, joint faulting has been slight. The average faulting of the group of joints containing the Type F assembly is comparatively high, but as yet no conclusions have been reached as to the effectiveness of these dowels in the transfer of loads. In November, 1956, the road roughness of the test section was measured with a duplicate model of the BPR Road Roughness Indicator. A roughness index of 114 in. per mile was obtained in the outer wheel track of the travel lane. Unfortunately, no earlier readings are available for this test section alone. However, in 1954, the roughness index obtained on four miles of this roadway which included the test section was 103 in. per mile. While the joint faulting as measured appears slight, the road roughness oscillograph recorder does show a definite roughness at the joints. More data are required to determine the effectiveness of the various dowels insofar as faulting is concerned.

5. That end movement of the 1,800 ft test section during periods of high temperature has been considerably restrained with the result that compressive stresses of 400 to 600 psi may be expected. Undoubtedly, the fact that wood filler was used in the six expansion joints immediately preceding and following the test section accounts for a good deal of the lack of movement at the ends of the test section.

6. That rusting of the dowels at the joint openings has become quite pronounced after three years and is undoubtedly a large factor in the non-uniformity of joint opening.

7. To date, there has been no sign of any physical distress in the pavement of the test section.