An Experimental Self-Stressing Concrete Pavement: II. Four-Year Pavement Evaluation

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During the 1963 paving season, an experimental reinforced concrete pavement was constructed in which an expansive cement was used to produce thin prestressed slabs. The experimental section contains three slabs 24 ft wide, 6 in. thick, and approximately 490 ft long. The conventional pavement, of 40-ft contraction joint design, is placed 9 in. thick in two 12-ft lanes.

This report is devoted to the 4-year, post-construction performance of this pavement. Slab movement data, crack observation, and other information are presented, as well as suggestions for the future application of this self-stressing technique.

Early construction difficulties andpavement distress appear to have significantly affected the pavement performance to date. Transverse cracking that has developed in the test section is probably related to insufficient prestress in the concrete slabs.

•IN 1963, the Connecticut State Highway Department constructed an experimental expansive cement concrete pavement. The feasibility of the test installation was explored in early meetings with Bureau of Public Roads personnel and other interested parties (1), and a pilot installation was made in June 1963. Final plans for the test section were completed after the pilot installation, and the test pavement was placed in September 1963.

The 1500-ft test area is in the southbound roadway of Route 2 in Glastonbury. The facility is a four-lane divided highway with fully controlled access. The test area starts in a gravel fill at the northern end and runs into a reddish-brown clay cut that continues throughout the test section. The southbound roadway at the test site is a tangent section with a +1.5 percent grade that enters a superelevated horizontal curve to the right about 150 ft from the southern end of the test area (Figs. 1 and 2).

DESIGN AND CONSTRUCTION

The following design features were included in the test section. To minimize subgrade friction, two layers of polyethylene sheeting were placed on a 1-in. depth of dense-graded bituminous concrete for the subbase. To avoid difficulties with keyway formation, 24-ft paving with no longitudinal joint was required. Reinforced concrete sleeper blocks were placed flush with the surface of the bituminous concrete at the end of each experimental slab (a) to provide anchorage for the longitudinal reinforcement, and (b) to provide anchorage for devices to restrain the slab ends from curling and restrain vertical displacement.

Paper sponsored by Committee on Rigid Pavement Design and presented at the 48th Annual Meeting.

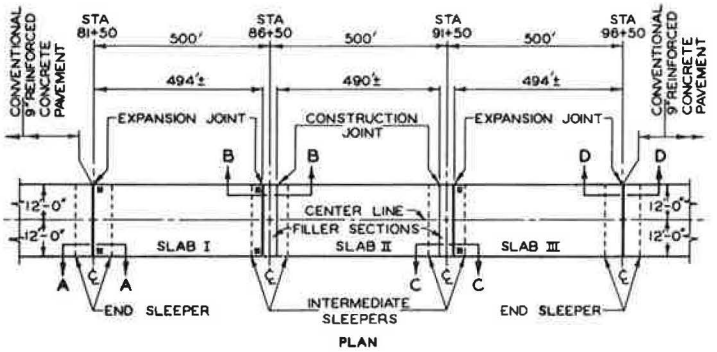

Figure 1. Self-stressing concrete pavement.

Longitudinal reinforcement for the test slabs was with a ½-in. diameter, 7-wire strand prestressing cable (ASTM 416), spaced 14 in. on centers with outside strands 4 in. from the edge of pavement. The cables were continuous throughout the slab length and were initially tensioned to a load of approximately 1,000 pounds. Transverse reinforcement was provided by No. 7 deformed steel bars (ASTM A 432 and ASTM A 305) spaced 2 ft on centers, and alternating above and below the longitudinal strands. The ends of the deformed bars were bent 90 deg to form hooks of 8-diameter length and laid flat to clear the forms by 1½ in. Transverse bars were tied to the longitudinal cables and all steel was supported by chairs.

To restrain the slab ends from curling, a device, which was anchored in the sleeper block and the experimental slab and allowed free end movement of the test pavement, was used at the ends of slab I and the northern end of slab III. The other end of slab III was restrained by using the downward component of the resultant pavement stress. Slab II was restrained by tying concrete filler blocks to the slab ends (1, p. 58).

Mix design for the project required an 8½-bag mix that included an expansive component conforming to State Highway Department specifications. Concrete was to be placed with a 1½-in. slump and fog-spray cured for a minimum period of 24 hours after placement.

Early difficulties with consistency control, and consequently with the finishing operations, were lessened by field changes in the mix design. The difficulties encountered were caused by the cement's affinity to water, resulting in slab growth that was less than anticipated. The resultant longitudinal stress in the pavement was approximately 70 to 90 psi at the slab ends, 30 psi at the mid-length, and about 140 psi transversely. In addition, the slab surface was very rough and wavy because of the finishing of the

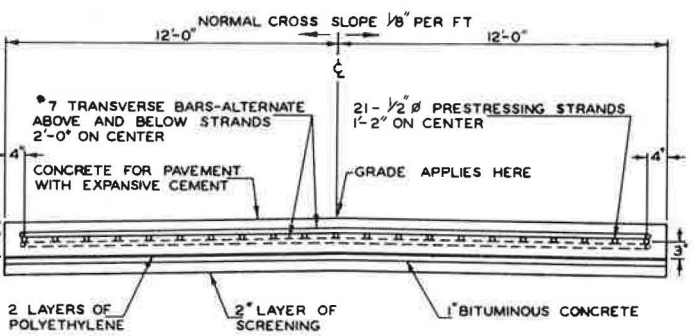

Figure 2. Typical cross section of pavement.

stiff concrete. Further details on the design and construction were given by Dougan (1).

SIGNIFICANT FINDINGS

Two types of transverse cracking have developed in the test area. They are full-width transverse cracks, greater than 12 ft in length, and partial transverse cracks less than 12 ft in length. Both types of cracking are attributed to inadequate tensile strength in the pavement slabs. Insufficient prestress is believed to be a major factor contributing to the cracking. The relative vertical position of the transverse reinforcement does not affect the full-width cracks except by reducing the cross-sectional area of the concrete slabs. The partial cracks start at the hooked ends of the transverse bars and progress toward the centerline of the roadway. The vertical position of the transverse steel does not affect the occurrence of partial transverse cracks.

The transverse joints in the test area are closing. The transverse cracks in the "self-stressed" slabs have opened, resulting in permanent end displacements. Resistance to movement at the filler blocks at the ends of slab II is also contributing to the closure at joints 2 and 3. The adjacent conventional design also shows a permanent end displacement, which is to be expected from a contraction-joint design pavement.

Slab end curling has not been detected. Slab hold-down devices (1) could probably be eliminated and cost savings achieved. If the filler blocks are used to prevent curling, further research on the amount of steel required to hold the filler blocks to the ends is needed. It appears feasible to use the downward component of the self-stress as a hold-down mechanism, but data are limited on this aspect of the project. To utilize the self-stress it is necessary to lower the longitudinal reinforcement below the neutral axis. As the pavement grows longitudinally, a downward component of the resultant prestress would be developed and, in conjunction with the weight of the slab, would act as a vertical restraint.

The daily and seasonal joint movements indicate a fairly uniform unit change in length per unit change in temperature. These data do, however, indicate a slight decrease in magnitude with time. The decrease is probably related to the end displacements at the transverse joints. Estimates of the coefficient of expansion indicate that there may be a seasonal variation in the expansion characteristics of the test slabs.

The compression joint seal is working well. Minor humps have been created by the slab end displacements, but to date the seal has adhered well to the joint faces and has undergone upward displacement in only joint 1.

The surface of the test area shows large areas of surface wear or abrasion and development of spalls at some full-width transverse cracks. The following abnormalities were detected by the Materials Division of the Bureau of Public Roads and they could well be a basis for the development of the previously mentioned defects: (a) calcium sulfoaluminate, a weak component in portland cement, was present; (b) large air voids were found that indicate insufficient compaction, i.e., consolidation of the concrete; and (c) the entrained air content of the test slabs was quite low.

TRANSVERSE JOINTS

In mid-July 1964, the transverse joints between the test slabs were sealed with an extruded neoprene joint seal. The four transverse joints in the test area are designated joints 1 through 4 from north to south respectively (joints 1 and 4 at the termini of the test area and joints 2 and 3 within the test area).

The joints were prepared for sealing by cleaning out the joint cavity and sawing the joint faces to a 2½-in. width for joints 1 and 4, and a 4½- to 5-in. width for joints 2 and 3 (Figs. 3, 4, 5, 6, 7). While sawing the end of slab III, it was noted that the steel collars used in the tensioning of the longitudinal reinforcement had not been removed after construction (Fig. 3).

The seals were placed in the joints and recessed ¼ to ½ in. below the surface of the pavement. All seals were continuous for the 24-ft pavement width, and the ends of the seals were butted against a steel plate to prevent infiltration of materials from the side.

Figure 3. Joint 3—sawed face of slab III showing the steel collars.

Figure 4. Placing seal in joint 1.

Joints 1 and 4 required a 4-in. wide seal. For joints 2 and 3, two 3-in. wide seals were bonded together in the field and placed in the joint. To compensate for the lack of depth in the 3-in. seals, redwood strips were placed in the bottom of the joint groove for support (Fig. 6).

Joint Seal Performance

To date, the joint seal appears to be effectively embedded in the transverse joints. Unfortunately, maintenance forces placed hot-poured liquid joint seal on the upper surface of the neoprene seals. It was felt that the liquid sealer would compensate for the recess of the neoprene seal and thus minimize the thump at the transverse joints.

The neoprene seals have maintained their compressed position throughout the winter. In the summer months the compression seals form a slight bump because of the pavement expansion. Figure 8 shows the only area in which the joint seal has been forced out of the joint. The seal is about 1¼ in. (maximum) above the pavement for a distance

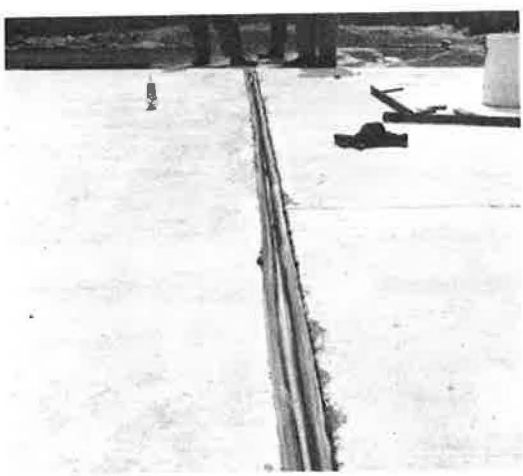

Figure 5. Joint 1—seal in place.

Figure 6. Joint 3—seal ready for placement; note the redwood strips used for support.

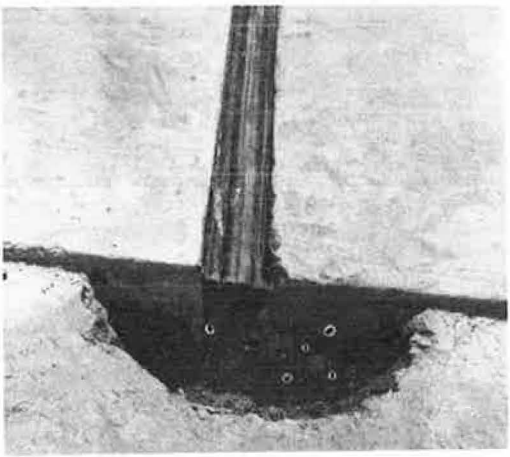

Figure 7. Joint 3—seal in place.

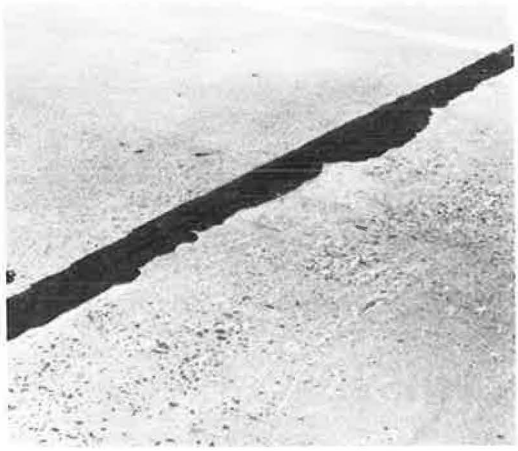

Figure 8. Transverse joint I—seal above pavement surface for a distance of 4 ft.

of 4 ft. Permanent end displacements are cited as primary factors in causing this extrusion. The rest of the seal appears to be performing well.

CONDITION OBSERVATIONS

Pavement Cracking

Pavement distress in the form of longitudinal edge cracks was noted immediately after the paving operation was completed (1). Since that time, the number of these cracks has doubled in slabs II and III and tripled in slab I. A few of these cracks have caused edge spalls that required maintenance (Figs. 9 through 11), but most of the cracks have been confined to the area above the hooked ends of the transverse reinforcement.

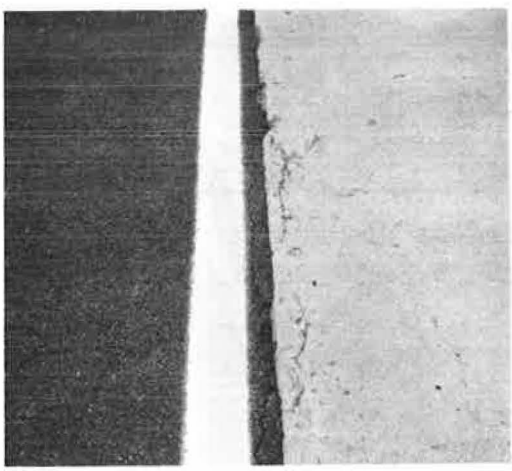

Figure 9. Slab II—longitudinal edge cracks near mid-length of slab; crack width approximately ⅛ in., August 1965.

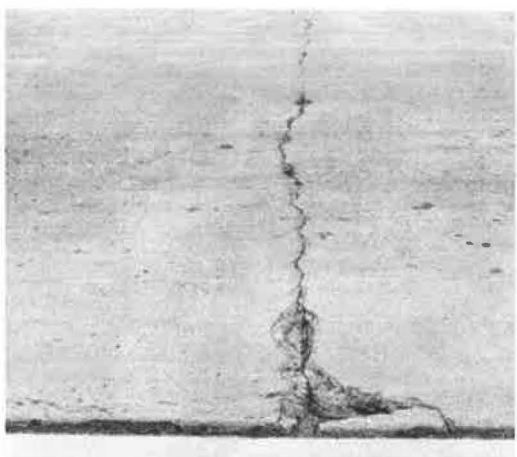

Figure 10. Slab II—distress at longitudinal edge crack, mid-length of slab; transverse crack open ⅛ in., August 1965.

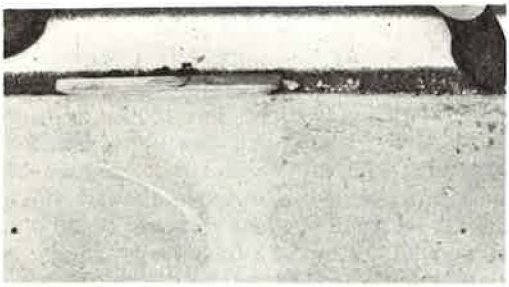

Figure 11. Slab II—completed repairs at area of distress shown in Figure 10; epoxy resin patching material used, August 1965.

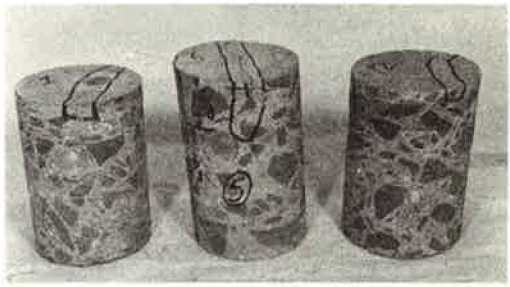

Figure 12. Cores taken at partial transverse cracks; core numbers 1, 5, and 12 taken in slabs I, II, and III respectively, January 1966.

About a year after paving, partial transverse cracks originating at the longitudinal cracks were noted. These cracks, generally 1 to 3 ft in length, are very narrow. Tables 1 and 2 summarize data obtained from pavement cores. The data show that these partial transverse cracks occur over the transverse reinforcement. The data indicate that these cracks can penetrate to the transverse bars but do not extend the full pavement depth, as can be seen in Figure 12. Crack widths measured at the surface of the core with a hand microscope range from 0.010 to 0.002 in. and they decrease to approximately 0.002 in. about ¼ in. below the surface.

Figure 13 summarizes the total amount of cracking with time. The figure indicates that the partial transverse cracking is still developing in the test area, but the rate of occurrence is decreasing with time.

These cracks are attributed to the plane of weakness created by the hooked ends of the transverse reinforcement. It is quite possible that this cracking will stop when most of the longitudinal edge cracks have developed a partial transverse crack. The greatest dangers from this failure are the development of full-width transverse cracks and the development of spalls along the crack faces and the edge of the pavement. Presently, the only areas that show adverse effects from the partial cracks are those in which construction difficulties were encountered, particularly the shoulder edge of slab II.

TABLE 1

SUMMARY OF CRACK DATA FROM PAVEMENT CORES

Slab No.	Core No.	Type of Crack	Depth of Core (in.)	Depth of Cover (in.)	Depth of Crack (in.)	Crack Width (in.)		Date Crack First Noted
						Top	Bottom	
I	1	Partial	5¾	1⅜	½ to ⅝	0.005	0.002 0.001	Dec. 1965
	2	Full	6 1/16	3 5/16	Full			Feb. 1964
	3	None	6 3/16					
	4	None	6¼					
II	5	Partial	6⅜	1⅝	1⅝	0.010	0.010 0.002	Sept. 1964
	6	None	6⅞					
	7	None	—					
	8	Full	6⅛	3 3/16	Full			Feb. 1964
III	9	Full	6 7/16	2	Full			Feb. 1964
	10	None	6 3/16					
	11	None	6⅛					
	12	Partial	5⅞	1½	3/16	0.003 0.001	0.001	Sept. 1964

TABLE 2

SUMMARY OF COMPRESSIVE STRENGTH OF CONCRETE CORES

Slab No.	Core No.	Strength[a] (psi)	Density (pcf)
I	3	5870	137.0
	4	5670	135.8
II	6	6190	139.7
	7	4980	136.2
III	10	5400	135.6
	11	4790	135.4

[a]All cores corrected to L/D = 2. Cores soaked in water for 48 hours prior to test and tested in moist condition.

Full-Width Transverse Cracks

In late November 1963, full-width transverse cracks appeared in the test slabs. The cracking was located at about the mid-length of the slabs, but has progressed toward the slab ends with time. Figure 14 is a plan view of the test site showing the present location of these cracks. There are now 38, 32, and 22 full-width cracks in slabs I, II, and III respectively. This cracking occurred before the pavement was 2 years old and there has been no appreciable increase in cracking of this nature since September 1965.

Cores taken over full-width cracks show that the cracks extend through the entire depth of slab, and that the location of the transverse reinforcement (above or below the longitudinal cables) has no effect on the cracks (Figs. 15, 16, 17). Inspection of the core holes gives no indication that the cracking has extended into the bituminous concrete

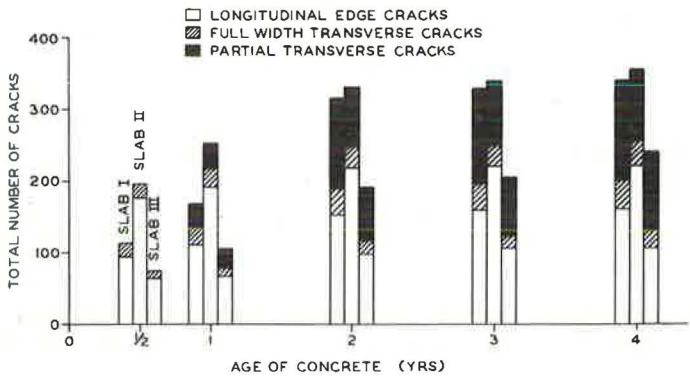

Figure 13. Summary of total cracking with time.

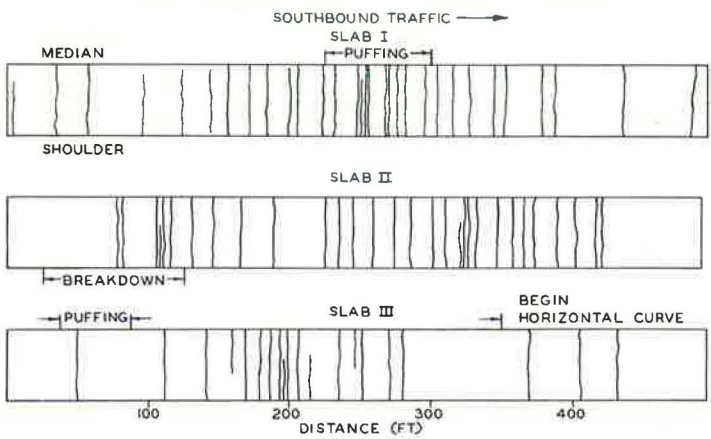

Figure 14. Plan view of test area locating full-width transverse cracks: age of concrete, 3 years.

Figure 15. Cores from slab I, January 1966.

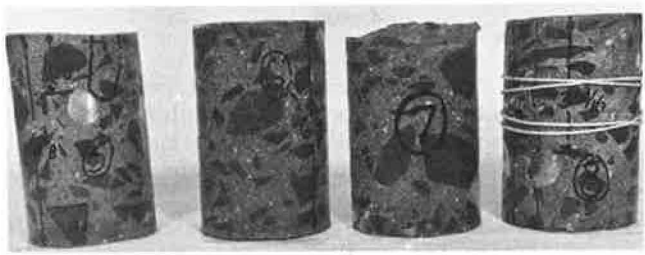

Figure 16. Cores from slab II, January 1966.

Figure 17. Cores from slab III, January 1966.

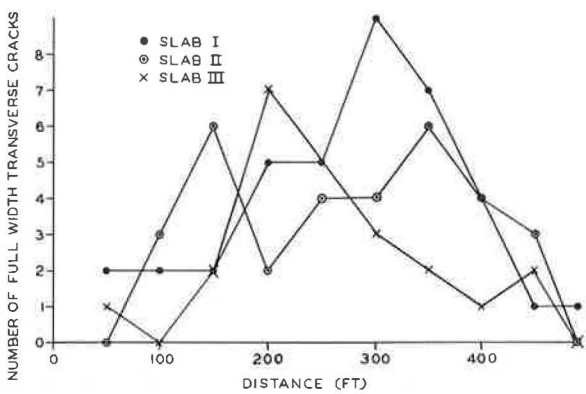

Figure 18. Longitudinal distribution of full-width transverse cracks: age of concrete, 4 years.

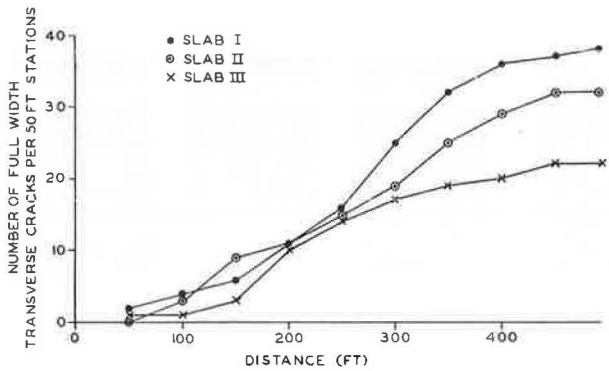

Figure 19. Cumulative distribution of full-width transverse cracks per 50-ft stations: age of concrete, 4 years.

subgrade. Crack widths from full-width cracks are not available; however, visual inspection of the cores shows that silt and fine sand particles have penetrated the crack to the depth of the transverse steel. Corrosion of the transverse steel has begun on the circumference of the bar. The corrosion is spotty and usually occurs on the surface of the bar closest to the surface of the pavement.

The full-width cracks are attributed to tensile stresses developed from subgrade friction. Figures 18 and 19 show the longitudinal and cumulative distribution of the cracks respectively. Note the high frequency of cracks in the central 200 ft of the slabs. Undoubtedly the low value of prestress (30 psi) at the mid-length of the slabs and construction difficulties have contributed greatly to this type of failure. Had the slabs attained sufficient growth to induce a uniform prestress of 300 psi, and had we assumed the tensile strength of the concrete to be 500 psi, then the concrete would have had 60 percent more resistance to the tensile stresses.

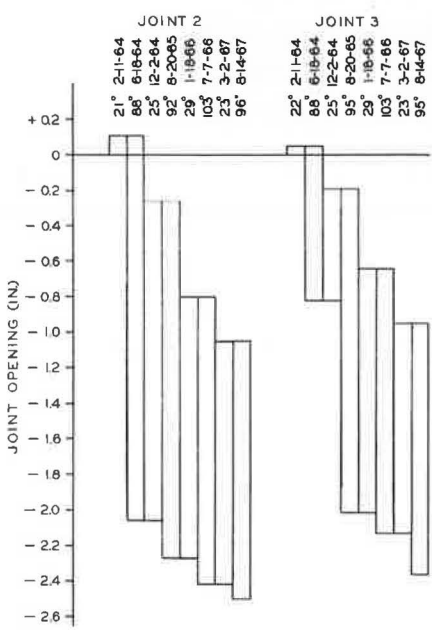

Figure 20. Seasonal joint movement for joints 2 and 3.

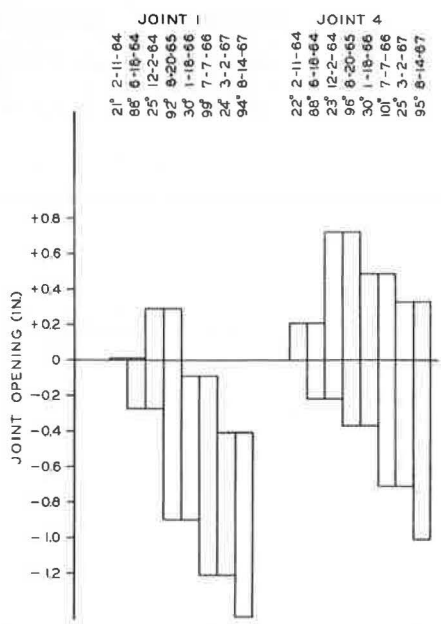

Figure 21. Seasonal joint movement for joints 1 and 4.

Figure 14 shows the approximate locations of the major difficulties encountered during construction. In slabs I and II, the high frequency of cracking in the areas indicated has been caused or been affected by discontinuities in paving operations experienced on the project. The highest frequency of cracking in slab III is about 200 ft south of joint 3. Although no breakdowns were encountered in this area, this is the approximate location of the start of the transition section that results in full superelevation at the south end of the test area. It is possible that the additional sand used in the concrete on this day and the stops required to adjust the screeds for the superelevation have caused a reduction in concrete strength sufficient to cause the high crack frequency.

In all probability the full-width transverse cracks are contributing to the development of compressive stresses within the test slabs by creating strains on the longitudinal reinforcement. This is supported by the core data. Unfortunately, the areas of slab II in which the strain instrumentation was located were filled with concrete shortly after paving the test site. This oversight precludes estimates of current slab stresses. However, data presented hereafter on joint closure reinforce the foregoing theory on slab stresses.

Joint Movement

Joint movement at the transverse joints in the test area is measured by instrumentation placed in the pavement during construction. All readings are obtained with a vernier gage and each set of readings is checked against highly probable limits established for the joint under observation. Figures 20 and 21 show the seasonal joint movement for the transverse joints. The data in these figures show the average seasonal movement indicated by four individual gage readings per joint. The base temperature for all joints is 54 F. Linear regression equations, which show change in joint opening as a function of change in temperature, are summarized in Figure 22. The equations were obtained using data from the time of construction up to and including June 1967. In all of the figures it is evident that the transverse joints are now closing. Joints 2 and 3 presently show a closure of 1.33 and 0.93 in. respectively, which has been gradually increasing in magnitude since construction. Joints 1 and 4 began to close about two years after construction and are still closing. Figure 23 shows the increase in joint closure with time.

Undoubtedly, their closure is owing in large part to slab end displacements resulting from the full-width transverse cracks in the test area. This is reflected in the increase in slope of Figure 23 between 1½ and 2½ years of age, which is the time period when the greatest number of full-width cracks were recorded.

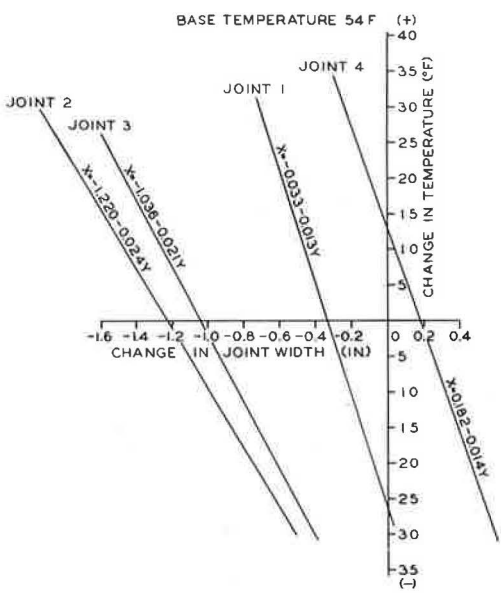

Figure 22. Regression equations, change in temperature vs change in joint width.

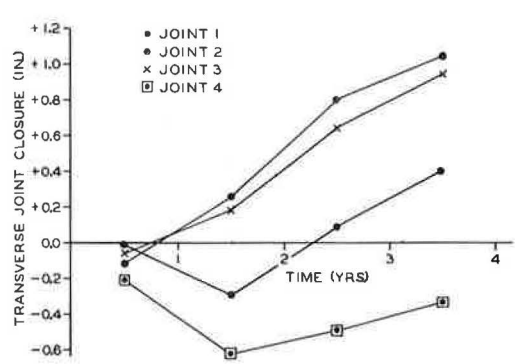

Figure 23. Decrease in joint opening with time.

The fact that the terminal joints and the central joints have, during the early stage, opposite slopes in Figure 23 is attributed to an obstacle to movement that developed at the filler blocks early in the pavement life. During the first winter, the interface between the filler blocks and the slab ends opened. Subsequently, longitudinal and then semicircular cracks developed over or extending from the tiebars, and a full-width transverse crack appeared at about the mid-length of the filler blocks. These defects are shown in Figures 24 through 27. The sequence in which the distress appeared suggests that the restricted movement is caused by frictional resistance. It was pointed out (1) that the steel I-beam used in tensioning the longitudinal reinforcement was to be burned off "reasonably flush" with the surface of the sleeper slab. Experience with this type of operation indicates that it is nearly impossible to remove the I-beam flush with the surface of a supporting concrete block. Furthermore, pieces of blown glass were set on top of the web sections that extended above the sleeper slab. Therefore, it is highly probable that early resistance to movement developed as a result of the filler block being forced to slide over pieces of the I-beam that protruded from the sleeper slab. At present no cores are available to support this theory, but in the future they could easily be obtained for confirmation.

Pavement Cross Sections

Elevations were obtained at the slab ends, 50 ft from the ends, and at the midpoint of each slab to measure slab curling or frost effects in the test area. Base readings were obtained after construction and the measurements continued each winter. To date, no curling of the slab ends or frost effects have been detected.

These data and the few full-width transverse cracks adjacent to the slab ends suggest that the means used to prevent curling show promise. The uniform movement at the transverse joints also supports this statement. At this time, it is not known what influence, if any, the full-width transverse cracks have on slab curling. Possibly the subdivision of the slabs by the cracks has offset any failure caused by curling stresses.

Data presented herein show that further research is required if the filler blocks are to be used for vertical restraint. The special hold-down devices (1) could be eliminated because of economic considerations. The resultant downward component of the self-stress is judged to be, presently, the most practical method of slab tie-down. The downward resistance to vertical displacement can easily be achieved in the field with no added construction cost.

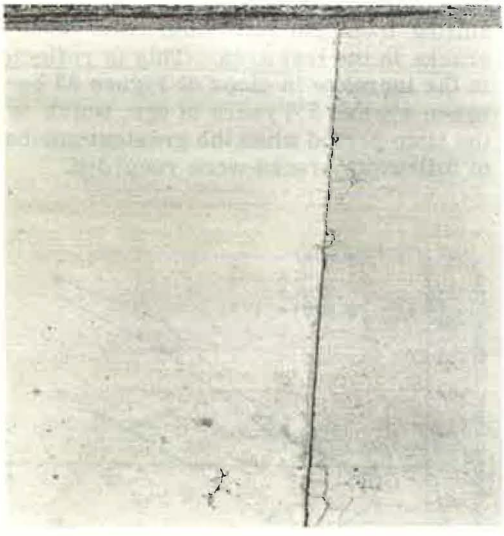

Figure 24. Interface at filler block between slabs II and III; crack width approximately ¼ in., April 1964.

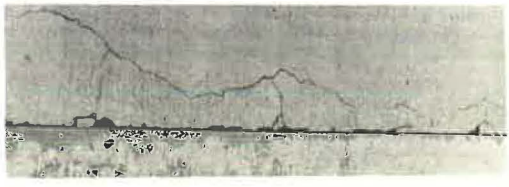

Figure 25. Longitudinal and semicircular cracks, same location as Figure 24, April 1964.

Figure 26. Close-up of cracking in Figure 25, April 1964.

Figure 27. Filler block between slabs I and II, faulted at median side $\frac{5}{8}$ to $\frac{3}{4}$ in., April 1964.

Figure 28. Slab I—gage plug instrumentation at a partial transverse crack, mid-length of slab; longitudinal edge crack in upper center of photo.

Surface Condition Observations

The present surface condition of the test area is shown in Figures 27 through 34. Large areas of surface wear as well as spalls have developed at transverse cracks. Cores removed from the test slabs indicate that the concrete is fairly homogeneous despite construction difficulties. A general examination of cores showed the following:

Figure 29. Distressed area of slab; width of spall, 3 to 6 in.; at median edge, 12-in. steel not visible.

Figure 30. Transverse crack 15 ft south of distress in Figure 29; minor spalling, crack tight.

Figure 31. Close-up of area in Figure 30 showing longitudinal edge crack.

Figure 32. Transverse crack in slab II; rust spots on surface of pavement at crack.

Figure 33. Cracking at interface between slab II and filler block 3.

Figure 34. Transverse crack in filler block 3 approximately at mid-length of the block.

1. Calcium sulfoaluminate was present, but this is to be expected because it is the normal product of an expansive reaction in portland cement.

2. Numerous large air voids were encountered. These indicate insufficient compaction of the plastic concrete, and are probably related to early construction difficulties.

3. The air content of the test slabs is quite low. The air content of slabs I, II, and III is 2.6, 2.6, and 3.2 percent respectively, as determined by the high-pressure method. Bureau of Public Roads comments on the marginal air content of the cores indicated a good probability of developing surface scale.

The author believes that the wear in the test area probably results from insufficient entrained air, coupled with the use of de-icing chemicals. Furthermore, the presence of calcium sulfoaluminate, which is a relatively weak crystalline structure in a cement paste, would tend to make the slab surface more susceptible to abrasion.

Present thinking is that the occurrence of spalling in the test slabs is affected by the presence of the large air voids and calcium sulfoaluminate. The wide, full-width transverse cracks that have developed have no load transfer mechanism; therefore, the reduced strength of the mortar could aggravate spalling in the test area.

SUPPLEMENTARY DATA AND INSTRUMENTATION

In March 1965, additional measurements on the end displacements of each test slab and the adjacent conventional concrete were started. The data are obtained by establishing a reference line at each transverse joint using monuments installed prior to paving. The slab end movements are then measured with a hand rule and recorded to the nearest $\frac{1}{32}$ in. Two sets of readings are taken at each joint at each edge of pavement. The average movement of each slab end is then reported as the average of four readings. All measurements are statistically analyzed.

Figures 35 and 36 show the slab displacements at each transverse joint. The total movement is the sum of the measured individual slab displacements at the joint. The

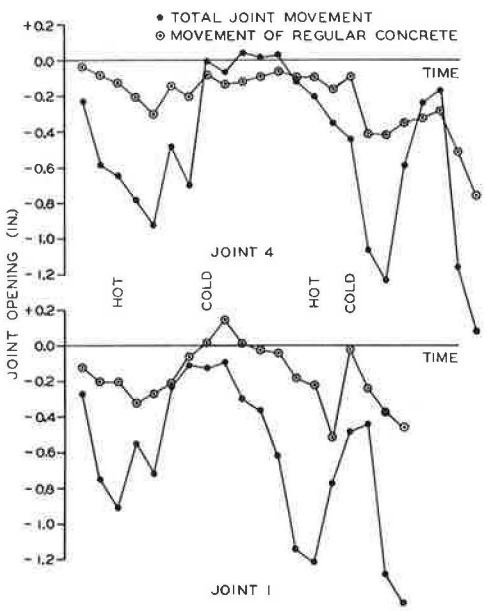

Figure 35. Thermal end movement of regular concrete.

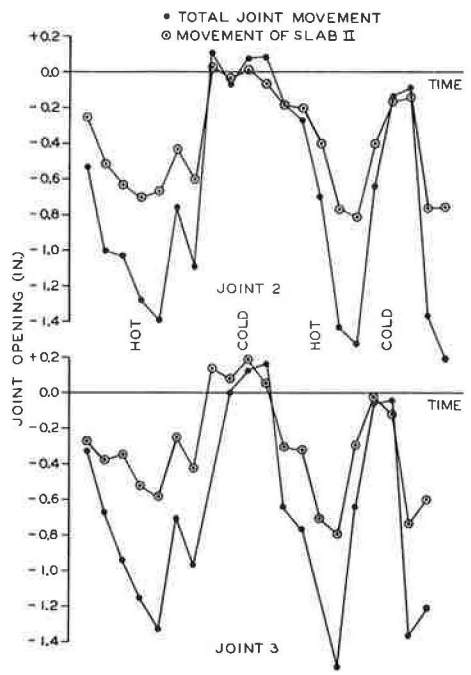

Figure 36. Thermal movement of slab II.

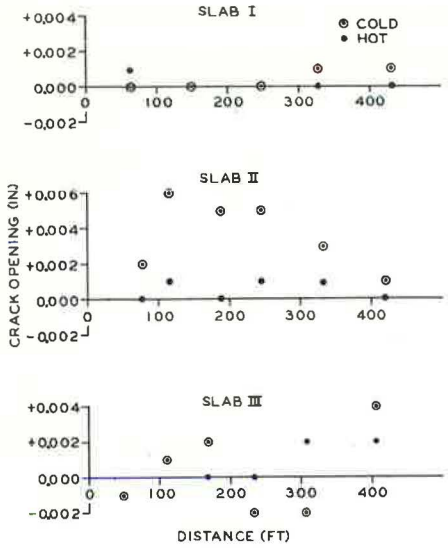

Figure 37. Width of full-width transverse cracks.

movement of slab II is shown and the movement of the opposite slab end is the algebraic difference between the total movement and the movement of slab II. The data are not corrected to the original base readings for the joint movements, but the total joint movement measured at the slab ends is comparable to that measured with the vernier gage.

In July 1966, additional gage plugs were installed in the test area at selected cracks, in the interface between the filler block and the ends of slab II, and at a full-width crack in the filler block. Figure 37 shows the increase in crack width from summer to winter for the full-width transverse cracks. Recognizing the fact that the additional gage plugs would have been more valuable if they had been installed at an earlier date, it is felt that the data support the observation that the full-width transverse cracking in the test area is contributing to the joint closure. This can be seen in the data for slab II in Figure 20.

The partial transverse cracks show a seasonal movement of 0.003 to 0.007 in. The cold-weather data indicate that these cracks are open more than the full-width cracks. It is quite probable that the smaller amount of movement at the full-width cracks results from the intrusion of incompressible material into the cracks that creates localized compressive stresses in the test slabs.

Figure 38 shows the change in crack width at the interfaces of slab II and the filler blocks. In Figure 36 there is about $^1/_{10}$ in. difference in the end displacement of slab II and the adjacent slabs. The difference in end displacements is probably owing to the excess movement of the filler blocks at the ends of slab II. This condition results from a combination of factors, the primary causes being insufficient steel tiebars and the resistance to movement cited previously. Initially, it was thought that the difference in end displacements might be caused by the difference in thermal characteristics resulting from changes in mix design. This theory was abandoned because of the error involved in measuring the end displacements and the fact that the unit change in joint width with respect to change in temperature is fairly constant for all three test slabs (Fig. 39).

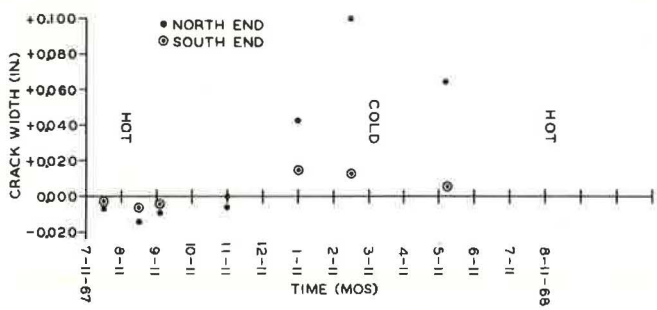

Figure 38. Crack width of interface between slab II and concrete filler blocks.

Estimated Coefficient of Expansion

Estimates of the coefficient of expansion were obtained from the daily joint movement data of joints 2 and 3. Computed values for the joints are 0.0000037 0.0000041 in. per in. per degree Fahrenheit respectively. These estimates are obtained by using an assumed length of 500 ft and the average joint movement measured over a 10 F or greater rise in temperature. No data are available on the amount of daily contraction in the test slabs. An estimate of the seasonal coefficient of expansion and contraction can be obtained from the data in Figure 20 by dividing the seasonal movement by 6,000 in.

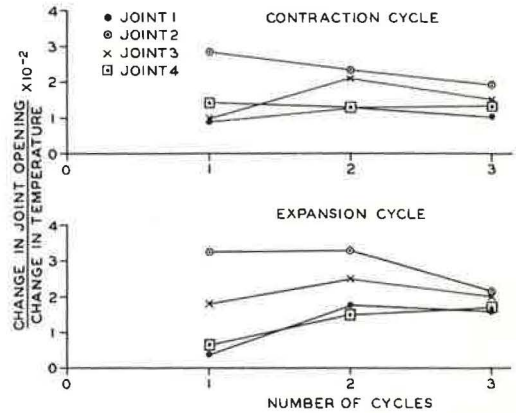

Figure 39. Decrease in joint movement per unit change in temperature.

Table 3 gives data on the daily change in length used to calculate the coefficient of expansion. Note the wide dispersion of data. There appears to be cause to suspect that the amount of expansion will vary with the season of the year—that is, the amount or rate of expansion is greater in the summer than it is in the winter. Figure 23 indicates a decrease in joint opening with time. The amount of influence this has on the expansion and contraction characteristics of the slabs is not known at this time. Presently, it is felt that more data are required to determine if significant differences can be detected in the daily and seasonal coefficients of expansion.

PAVEMENT SERVICEABILITY

Based on the minimum number of cracks and pavement surface wear, slab III has performed best. The cracks in slab III are tight and only minor spalling has occurred. This again reflects improved paving technology over that used in paving slabs I and II.

TABLE 3

DAILY CHANGE IN LENGTH MEASURED AT JOINTS 2 AND 3

Joint 2					Joint 3				
Temperature		T (F)	Length (in.)	L/T	Temperature		T (F)	Length (in.)	L/T
a.m.	p.m.				a.m.	p.m.			
77	92	15	0.3350	0.0223	77	95	18	0.4723	0.0262
77	92	15	0.4828	0.0322	71	85	14	0.3785	0.0270
60	70	10	0.1748	0.0175	39	52	13	0.2993	0.0230
69	84	15	0.4048	0.0270	45	56	11	0.2610	0.0238
36	52	16	0.3163	0.0198	81	93	12	0.2853	0.0238
43	57	14	0.3025	0.0216	75	87	12	0.3705	0.0309
80	94	14	0.2990	0.0214	75	88	13	0.2725	0.0210
89	103	14	0.2760	0.0197	77	95	18	0.3830	0.0213
70	87	17	0.4170	0.0245					x 0.1970
53	64	11	0.2708	0.0246					$\bar{x}$ 0.0246
72	87	15	0.2658	0.0177					e = 0.0000041
76	96	20	0.4070	0.0204					
				x 0.2687					
				$\bar{x}$ 0.0224					
				e = 0.0000037					

TABLE 4

SUMMARY OF TRAFFIC DATA
(Percent of ADT)

Year	ADT	Passenger Cars	Single-Unit Trucks			Combination Trucks			Buses
			4 Tire	6 Tire	10 Tire	3 Axle	4 Axle	5 Axle	
1965	4,900	88.7	5.7	2.4	0.1	0.5	2.0	0.5	0.1
1966	6,100	86.7	7.0	3.0	0.1	0.5	1.9	0.6	0.2

The present pavement condition of the test area is shown in Figures 27 through 34. The cracks in Figures 30 and 32 are typical of those in slab III.

Tables 4 and 5 summarize the traffic data and the estimated PSI and equivalent 18-kip axle loads respectively. In Table 4 it is evident that the test area has not been subjected to a heavy traffic load. This is supported by the estimated equivalent 18-kip axle loads shown in Table 5.

In Table 5 two values of D were used to compute the 18-kip equivalencies. This was done because it was not known to what extent the steel reinforcement has influenced the pavement stiffness. In other words, is D = 6 in. equal to D = 9 in.? Because the sums of the equivalent 18-kip loads, by both assumed values of D, differ by roughly 4 percent, the value of D = 6 in. will be used hereafter.

The PSI (2.98) of the test area is also shown in Table 5, and is based on pavement roughness determined by a BPR-type roughometer (7). The roughness readings, which are the average values from all wheelpaths in the 1,500-ft test section, indicate a decrease in roughness with time. This is inconsistent with condition observations of the test slabs. Supplementary calculations of the PSI, incorporating estimates of cracking and patching per 1,000 sq ft of pavement surface, show that the PSI of the test area is reduced by about 0.6 for the 1966 value and slightly less than 0.6 for the 1965 value. However, the decrease in pavement roughness from 1965 to 1966, when used in the PSI formula, indicates that the PSI has increased from 1965 to 1966. These data suggest that some other measure of pavement serviceability is needed for this project.

SUGGESTED FUTURE RESEARCH

Based on data from the test installation, the following suggestions are submitted for consideration in future research on expansive concretes:

1. To attain uniform longitudinal prestress in the pavement slabs, some type of positive anchorage should be provided along the reinforcing wires. Professor Alex Klein suggested attaching slotted washers at various intervals on the longitudinal reinforcement. Klein also thought that there should be a better distribution of steel in the cross-sectional area of the pavement, with more strands used to achieve a given area.

To fully utilize the potential of an expansive cement to develop prestress, additional data are needed on type and percentage of reinforcement, bond strength of the resultant

TABLE 5

ESTIMATED PAVEMENT SERVICEABILITY INDEX AND EQUIVALENT 18-KIP AXLE LOADS

Year	PSI*	Pavement Roughness (in./mi)	Estimated Equivalent 18-Kip Loads**		18-Kip Loads	
			D = 6 in.	D = 9 in.	D = 6 in.	D = 9 in.
1963	2.81	182	—	—	—	—
1965	2.84	177	97,400	92,600	—	—
1966	2.98	162	95,900	87,900	193,300	185,500

*PSI = 5.34 - 1.70 (2.16 log R - 3.39). R = pavement roughness (in./mi, BPR-type roughometer).
**p = 2.5.

concrete, optimum spacing of intermediate anchorages, and accurate information on the percentage of expansion and rate of gain of strength under field conditions.

2. Slab lengths of 500 ft are too long. Klein indicated a preference for slab lengths of 175 to 225 ft with filler sections placed between adjacent prestressed sections. The difficulties encountered in this test installation do not permit an evaluation of the optimum slab length, but the full-width transverse cracking that developed indicates that the most effective slab length is less than 500 ft. It could be anticipated that economic considerations would be of primary importance in determining an optimum slab length.

3. This project has provided field experience with expansive cement concrete highway construction. The data in this report suggest that the test slabs are now performing in a manner similar to continuously reinforced concrete pavements. However, the following questions have not been answered: (a) Will the transverse cracks in the test area, which have caused the joints to close prematurely, create sufficient strain in the longitudinal cables to stabilize the joint closure? or (b) Will the increase in crack width create localized compressive failures in the test slabs? It is the intent of future observations on this project to answer these questions.

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