EXPERIENCE WITH CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN VIRGINIA

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This paper reports studies of the cracking characteristics of continuously reinforced concrete pavement (CRCP) built in Virginia since 1966 and observations concerning the performance of this type of pavement. Early studies involved crack frequency, in situ width determinations, and laboratory examinations of cores. Later studies determined the seasonal movement of cracks and the patterns of crack development with time. All studies were intended to determine the normalcy of characteristics of Virginia CRC pavement as compared to characteristics reported by other agencies. It is concluded from the studies that cracking patterns tend to be fully developed in 2 to 3 years and that an early progressive increase in surface crack widths seems to stabilize after a similar period of time. The core studies showed that cracks are discontinuous in the immediate vicinity of the reinforcing steel and grow progressively wider toward the top and bottom surfaces. This finding is taken as evidence that no danger of steel corrosion exists in pavements having normal crack patterns. The performance of the Virginia CRC pavement is considered good. However, at least 1 project shows severe damage related to inadequate consolidation of the paving concrete. The need for a strong effort toward the development of realistic consolidation specifications is pointed out.

•THE first continuously reinforced concrete pavement (CRCP), about 15 miles (24 km) in length, was built in Virginia in late 1966 and early 1967 on Interstate 64 around Richmond. Later projects brought the total to some 185 miles (298 km) by the end of 1972. All projects have been constructed in moderately heavy traffic corridors on the Interstate system.

The earlier pavements were constructed on unstabilized subgrades and subbases and contained both longitudinal and full transverse reinforcement. Later pavements were constructed on cement-stabilized subgrades and subbases and contained only longitudinal reinforcement with transverse tie bars between 12-ft (3.66-m) lanes. Similarly, there has been a transition from the use of side forms to slip forms in the placement operation.

A summary of the existing design features of CRCP in the state is given in Table 1. The pavements are 8-in. (20-cm) thick with 0.6 percent longitudinal reinforcement. Periodically, studies have been conducted in an effort to answer some of the questions that arose as changes were made in design features. Are the cracks in continuously reinforced concrete pavements in Virginia different in frequency from those found on such pavements in other states? Are the crack widths different from those found in other states? Are the cracks wide enough to allow corrosion of the reinforcement? Are cracking frequency and width related to some important variables in construction? Do the cracks tend to grow wider and more frequent with age? What are the effects of the elimination of full width transverse reinforcement? Has a cement-stabilized subbase led to a substantially better pavement structure?

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The first 4 questions were dealt with in reports to the Virginia Highway Department (1, 2), and the last 3 questions are still under consideration in a study on which 1 progress report has been issued (3). This paper summarizes the findings of these studies through early 1973. Several noteworthy performance problems also are discussed.

CRACKING FREQUENCY

Cracking frequency and its changes have been determined by periodic visual examination of a number of randomly selected, 200-ft (61-m) long test sites on projects given in Table 1 as groups A and D.

Group A Pavements

The results of 2 surveys of cracking frequency on several of the group A projects on I-64 near Richmond are given in Tables 2 and 3. The spacings shown are averages for two 12-ft (3.66-m) lanes unless otherwise indicated. Exact locations, paving dates, and climatic conditions at the time of paving are given in another report (1).

For the western sites (Table 2) there was a general reduction in crack spacing between 1967 and 1969. On the other hand, the section having the shortest spacing in 1967 (site 4, westbound) remained unchanged in 1969, which suggests that the crack pattern here was already fully developed at the time of the earlier survey. The average spacing for all western sites was 3.88 ft (1.18 m) in 1969 and to 4.70 ft (1.40 m) in 1967. These averages do not reflect the fact that some cracks are spaced 6 in. (0.15 m) to 1 ft (0.30 m). Such averages are consistent with experiences in other states (4), and the agreement of the averages with those from other states was considered important.

The 1969 survey shows that, unlike in 1967, the western and eastern test sections now are remarkably similar in crack spacings [average of 3.83 ft (1.17 m) and 3.97 ft (1.21 m) for west and east respectively]. This finding indicates that the crack pattern in the eastern sections was not so fully developed in 1967 as that in the western sections. This was no doubt due to the fact that most of the eastern sections were placed in cool weather (fall 1966 and spring 1967) and were surveyed before much exposure to extreme temperature changes. The western sections, on the other hand, were all placed in the summer of 1966 and had been subjected to extreme temperature changes before the 1967 survey. Time and cyclic temperature extremes by 1969 appear to have obscured any differences present in the earlier survey.

Group D Pavements

The results of similar, but later, cracking-frequency surveys on several group D (no transverse steel) pavements are given in Table 4. In this case, the average crack spacings have been determined monthly for each of the eleven 200-ft (61-m) long test sections during the past 3½ years. Precise site locations and other pavement details are given in another report (3).

The spacing for the spring and summer concrete is approaching that expected for the type of pavement under study. Similar pavements (0.6 percent longitudinal reinforcement) had crack spacings of 3 to 5 ft (0.91 to 1.52 m) at ages of 2 to 3 years.

The winter concrete, on the other hand, shows signs that the spacing may never be so close as expected. This is no doubt due to the extremely cold weather during which the concrete was placed. Overnight temperatures typically fell below 30 F (-6 to -2 C) during this December 1969 paving period. The nearly 13-ft (3.96-m) average crack spacing contrasts sharply with that for some of the July 1970 pavement, which had an average spacing of about 15 ft (4.57 m) within 48 hours after placement at temperatures ranging to 90 F (32 C). Clearly, hydrothermal stresses have had a more pronounced effect on the summer than on the winter concrete. It is apparent that the winter concrete, placed at very low temperatures, has seldom been exposed to significant tensile stresses and that, unless the pavement experiences an extremely cold and dry winter, the number of cracks may never increase significantly.

CRACK WIDTHS IN GROUP A PAVEMENTS

Crack Measurements In Situ

The width of each tenth crack within the 200-ft (61-m) sites was determined in each wheelpath by means of a measuring microscope at a magnification of about 15x. This method is subject to greater operator interpretations than is a mechanical strain gauges measuring between reference points but has been shown to be reliable (1).

The optics are such that the measurement is made at some depth below the surface [certainly less than $^1/_{16}$ in. (0.16 cm)]. At this point there are usually 2 definite "edges" of the crack. The microscopic method thus gives the width near the surface, which is probably somewhat greater than comparable widths measured with mechanical gauges from reference points.

The average crack widths for each of the sites west of Richmond are given in Table 5. The averages represent measurements at right and left wheelpaths on 3 cracks for each traffic lane, so that each average is a composite of 6 individual measurements. Crack widths were measured only in the eastbound lane.

There is little difference between the 1967 and 1969 crack widths for the western projects. Some sites indicate a slight closing of the cracks, others a slight opening. These minor differences may be due to the inherent difficulty in reproducing exact measurements on the very variable crack widths. The average widths of all cracks for 1967 and 1969 [0.012 in. (0.031 cm) and 0.014 in. (0.036 cm) respectively] are very close and probably indicate no true change. Average temperatures were 56 F (13 C) in 1967 and 40 F (4.4 C) in 1969.

Crack-width measurements on the eastern projects are given in Table 6. In these cases, crack measurements were randomly distributed between the eastbound and westbound lanes. The 1969 measurements were approximately double those for 1967. Although this increase in crack widths may be partially due to a more complete development of the crack pattern, as indicated by the crack spacings on these sections, it is also doubtlessly influenced by the marked difference in temperatures at the time of the 2 surveys, 80 F (27 C) in 1967 and 40 F (4.4 C) in 1969. The higher temperature in 1967 would have tended to hold the cracks tighter. In any event, the effect of the larger crack widths for these eastern sites is to make the eastern and western sites more similar, whereas in 1967 considerable differences existed between the two. In 1969, the average crack width for the 2 areas was the same, 0.014 in. (0.036 cm); the 1967 average for the eastern sites was 0.006 in. (0.015 cm).

Again, as with crack spacing, the crack-width data compared well with those from similar pavements reported by other states (4) so that the group A pavements were considered to be performing satisfactorily.

Crack Measurements on Cores

The possibility of corrosion of steel depends primarily on the width of crack at the steel, which in turn depends on the width of the surface crack and the depth of cover of concrete over the steel. Suggested maximum limits vary, but a recent survey of pertinent literature published at the University of Illinois (5) indicates that a limit on surface crack width of about 0.006 to 0.010 in. (0.015 to 0.025 cm) would be appropriate for members in aggressive environments but with a 1 to 2 in. (2.5 to 5.0 cm) cover. This report further states:

> If the crack is designated by its magnitude at the concrete surface, then depth of cover will control the limit on crack width. A limiting crack width so defined will increase with an increase in cover simply because the corresponding dimension of the crack at the reinforcement will be smaller.
>
> This would mean that for depths of cover of 3½ to 4 in. (9 to 10 cm) the allowable crack width at the surface would be about 0.0200 in. (0.051 cm).

To gain additional information concerning the possibility of reinforcement corrosion, we took a 4-in. (10-cm) diameter core from the pavement at each site in the western

Table 1. Continuously reinforced concrete pavement design features.

Group	Subgrade	Subbase	Transverse Reinforcement	Placement Method	Year Built	Mileage
A	Select material	Aggregate base	No. 4, 30 in., c. to c.	Side form	1966-67	26.98
B	Native	Cement-treated select material, min CBR 30	No. 4, 30 in., c. to c.	Side form	1968-69	13.92
C	Native	Cement-treated aggregate base	No. 4, 30 in., c. to c.	Slip form	1968-69	44.52
D	Soil-cement	Cement-treated aggregate base	Tie bars only	Slip form	1969-71	55.74
E	Soil-cement	Cement-treated aggregate base	Tie bars only	Slip form	1971-73	43.80

Note: 1 in. = 2.5 cm; 1 mile = 1.6 km.

Table 2. Average crack spacings for group A projects west of Richmond.

	Eastbound (ft)		Westbound (ft)	
Site	1967	1969	1967	1969
1	4.35	3.85	4.90	3.79
2	4.17	3.38	4.41	4.00
3	4.51	4.55	4.60	4.45
4	3.04	3.03	4.30	4.22
5	5.30	3.68	5.01	4.09
6[a]	5.60	3.37	6.10	4.17
Avg	4.50	3.64	4.89	4.12

Note: 1 ft = 0.3 m.
[a]Three 12-ft (3.66-m) lanes.

Table 3. Average crack spacings for group A projects east of Richmond.

	Eastbound (ft)		Westbound (ft)	
Site	1967	1969	1967	1969
1	6.17	3.11	5.60	3.42
2	10.66	4.43	7.44	3.92
3	10.64	4.20	7.18	3.83
4	68.75	3.76	12.03	4.51
5	32.67	4.13	7.59	4.51
Avg[a]	25.78	3.92	7.97	4.03

Note: 1 ft = 0.3 m.
[a]Avg is 16.87 ft for 1967 and 3.97 ft for 1969.

Table 4. Average crack spacings for group D pavements.

Date Paved	After 2 Months	After 1 Winter	After 2 Winters	After 3 Winters
May 1970	9.09	5.37	5.10	5.00
July 1970	8.57	5.97	4.92	4.96
December 1969	20.51	13.33	12.90	12.50

Note: Spacings are in feet, where 1 ft = 0.3 m.

Table 5. Average crack widths for projects west of Richmond in eastbound lane.

	Traffic Lane (in.)		Middle Lane (in.)		Passing Lane (in.)	
Site	1967	1969	1967	1969	1967	1969
1	0.012	0.019	–	–	0.012	0.011
2	0.013	0.015	–	–	0.015	0.013
3	0.012	0.010	–	–	0.009	0.008
4	0.014	0.019	–	–	0.013	0.011
5	–	0.022	–	–	–	0.018
6	0.009	0.016	0.012	0.013	0.016	0.011
Avg	0.012	0.017	0.012	0.013	0.013	0.012

Note: 1 in. = 2.5 cm.

Table 6. Average crack widths for projects east of Richmond.

		Traffic Lane (in.)		Middle Lane (in.)		Passing Lane (in.)	
Site	Direction	1967	1969	1967	1969	1967	1969
1	East	0.006	0.015	0.005	0.018	0.005	0.017
2	West	0.009	0.013	0.006	0.016	0.005	0.008
3	West	0.008	0.014	0.005	0.016	0.005	0.010
4	East	0.001	0.010	0.001	0.010	0.006	0.015
5	East	0.005	0.018	0.002	0.018	0.013	0.015

Note: 1 in. = 2.5 cm.

area for which cracking data had been collected. Of major concern in these cases was the nature of the cracks at points closer to the steel than could be studied in the field.

In the laboratory, photographs were taken of the top and each side of each core. The same measuring microscope that was used in the field work was used to determine the crack width at the surface of each core. Then the cores were sawed horizontally at distances of first 1½ in. (3.8 cm) and then ½ in. (1.3 cm) above and below the longitudinal steel, and crack measurements were made at these levels after the cut surfaces had been polished. Photographs illustrating the appearance of a typical crack at the various surfaces are shown in Figures 1, 2, 3, 4, and 5. Finally, the measurement of horizontal and vertical components permitted the calculation of the straight-line distance from the surface of the main reinforcing steel to the point of crack measurement.

The results of crack measurements for 5 of the cores are shown in Figure 6. For the measured cracks, the width tends to decrease by approximately a factor of 2 as the distance from the surface of the steel changes from 4½ to ¾ in. (11.4 to 1.9 cm). Although, as the scatter of points in Figure 6 shows, the correlation between crack width and distance from the steel is not high, definite trends can be seen in both the figure and in the visual examination of the cores. Projecting the crack measurements shown in Figure 6 to the surface of the steel suggests that at the steel a crack of approximately 0.003 in. (0.008 cm) still would exist. Visual examination (Fig. 5) showed that this was not the case and that no definite crack exists directly at the steel. Such a result has been reported in another paper (6) in which it was noted that in some cases cracks tend to diverge in the neighborhood of the steel and that it has been possible to find a zone around the steel where the concrete is not really cracked but is distended in a discontinuous manner. Such appears to have been the case with the cores in question. Figures 4 and 5 show that the cracks extend through the entire depth of the pavement but are discontinuous in the vicinity of the steel. Measurements indicated that whether cracks are located above or below the steel has little influence on the crack width and that the proximity of steel to the point of measurement is the determining factor.

The crack widths measured at the surface are of the same order of magnitude as those on pavements performing satisfactorily in other states, and the vast majority are within recommended limits for structural members if allowance is made for depth of concrete cover and age. Furthermore, the laboratory studies show that no definite crack exists at the surface of the steel for any of the cores examined. Thus, it is concluded that the cracks present no threat of steel corrosion and that the steel is functioning as designed, i.e., to restrain the concrete such that fine, harmless cracks develop at relatively close intervals.

Cores removed subsequent to the above studies have shown no evidence of steel corrosion except in cases were honeycombing has left the steel more vulnerable.

CRACK WIDTHS IN GROUP D PAVEMENTS

Because of the hazardous situation created by attempting to measure crack openings on Interstate highways with the measuring microscope used on the group A pavements, the changes in crack widths for the group D pavements are being measured by the use of gauge plugs spanning randomly located cracks within 11 test sections.

The gauge plugs are ½-in. (1.3-cm) diameter by 1-in. (2.5 cm) long brass cylinders tightly bound in predrilled holes with a paste made of expansive cement. Gauge points were drilled in the plugs after several days were allowed for the cement paste to harden. A nominal gauge length of 10 in. (25.4 cm) is used for monthly measurements with a dial gauge. Initial measurements were made with a Whittemore gauge, which was later replaced by a gauge with a larger dail (Fig. 7) built in the Virginia Research Council shop. The latter gauge was necessary because end anchorage movement (not reported here) exceeded the capacity of the Whittemore gauge. Measurements are recorded to the nearest 0.0001 in. (0.0003 cm) and are corrected for concrete volume changes within the gauge length. This correction is computed from measurements made on uncracked pavement immediately adjacent to the cracks. Average crack widths for the 3 study groups at critical times (maximum and minimum seasonal

Figure 1. Top view of core 6.

Figure 2. Plan view of core 6 at sawed surface 1½ in. (3.8 cm) above steel.

Figure 3. Plan view of core 6 at sawed surface 1½ in. (3.8 cm) below steel.

Figure 4. Left side view of core 6 showing continuous crack through core.

Figure 5. Right side view of core 6 showing discontinuous crack in vicinity of steel.

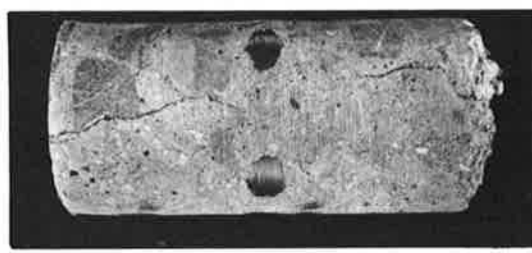
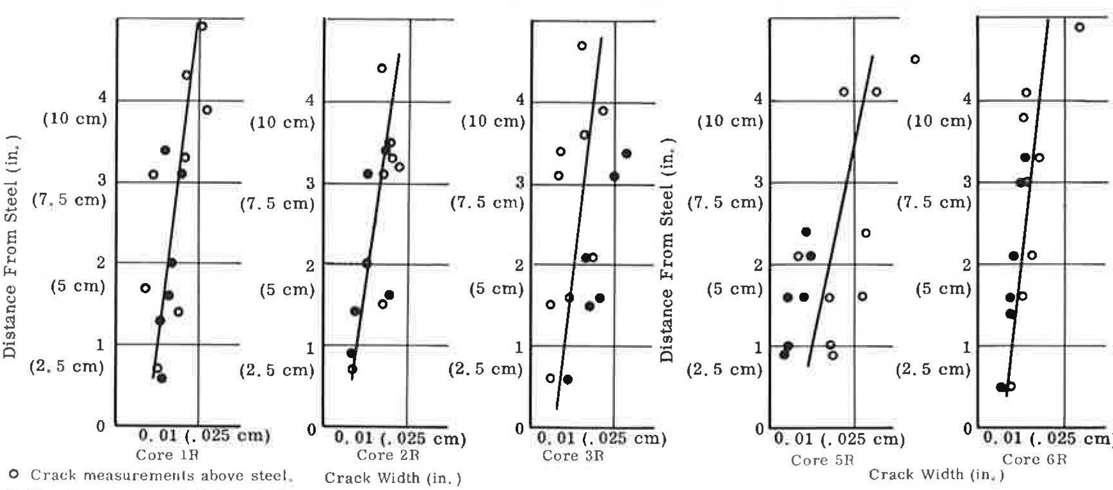


Figure 6. Crack width versus distance from steel for 5 cores.



measurements) are given in Table 7, where each entry represents an average of 12 measurements. Cracks were originally instrumented at an early date and at a width assumed to be nearly zero.

There is no apparent relation between paving date, crack spacing, and crack width. There is a pronounced seasonal effect as evidenced by the differences between the summer (narrow) and the winter (wide) openings. The summer 1972 measurements are wider than those for the summer of 1971. This may indicate a gradual increase in width. On the other hand, it may partially result from different seasonal conditions between the 2 summers. Further studies during the next several years should determine whether there is a progressive increase in crack width. The 1971-73 winter openings of 0.01 to 0.03 in. (0.03 to 0.09 cm) are in line with the results of earlier studies both in Virginia and in other states for similar pavements (1, 2).

EFFECTS OF THE ELIMINATION OF TRANSVERSE STEEL

Periodic examinations of the group D and E projects along with a study of the relevant literature have revealed no detrimental effects of the elimination of transverse steel from continuously reinforced concrete pavements. Although there was at one time some concern that occasional random longitudinal cracking was related to the elimination of the steel, studies showed that the cracking seldom went beyond the limits of the transverse tie bars provided between the 12-ft (3.66-m) lanes. It was also determined that several older projects having full-width transverse steel had even more of the longitudinal cracking than the projects from which the steel was eliminated.

The conclusion that elimination of the steel has not influenced the behavior of "no transverse steel" pavements is further substantiated by the similarity of their cracking characteristics, discussed earlier, to those on the group A projects studied in 1967 and 1969.

STRUCTURAL COMPARISON OF PAVEMENTS WITH STABILIZED AND UNSTABILIZED LOWER LAYERS

Although no extensive structural studies have been conducted, a series of Benkelman beam deflection tests was run recently on two 1,500-ft (457-m) long segments of pavement from group A (no stabilization) and group E (stabilized subgrade and subbase).

Edge deflections were measured at 50-ft (15.2-m) intervals through the test sections. Each measurement was made just as a dump truck loaded to 18,000 lb (8165 kg) on the rear axle traveled the normal wheelpaths past the point of testing.

The CRCP edge deflections (in inches, where 1 in = 2.5 cm) were as follows:

Group	Min	Max	Avg
A	0.008	0.016	0.010
E	0.004	0.010	0.006

The addition of the stabilized layers has been accompanied by a 40 percent average reduction in deflections. Although this reduction may be altogether due to the stabilization, slightly better subgrade soils in the stabilized area may have been a contributing factor. It is hoped that these preliminary deflection tests can be supplemented by tests on each pavement group.

PERFORMANCE PROBLEMS

Although the CRC pavement in Virginia has generally performed as expected, there have been several noteworthy problems that are indicated below in the order of their severity:

1. Pavement failures due to poor consolidation of concrete,
2. Pavement failures due to poor quality concrete,
3. Loss of shoulder material due to edge pumping related to inadequate drainage, and

4. Random longitudinal cracking.

A brief discussion of each of these performance problems follows.

Failures Due to Poor Consolidation

The most severe performance problem to occur in the CRCP to date is a result of honeycombing caused by inadequate consolidation of the paving concrete. An example of the most serious failure is shown in Figures 8 and 9. Figure 8 shows a segment of 8-in. (20-cm) thick by 24-ft (7.31-m) wide slip-formed CRCP from the group C pavements. A temporary bituminous concrete patch is evident in the picture. This type of failure is characterized by wide, arrow-straight cracks located directly above the transverse steel and by edge pumping, also shown in Figure 8.

The cause of this type of failure was immediately revealed by full-depth cores taken through the transverse cracks and at intermediate points between cracks. Figure 9 shows a side view of one of the cores taken through a crack at the transverse steel. There is a noticeable lack of consolidation of the concrete directly below the steel. The result is a void directly under the steel with a very wide crack extending from the void downward to the bottom of the CRCP. No doubt the reduced cross section caused by the void under the steel has resulted in the wider than normal crack at the pavement surface and in the crack lying in the same vertical plane as the steel. Strangely, cores taken from between the transverse bars showed no visual evidence of poor consolidation. Edge pumping in the section appears to have resulted from surface water entering the wide transverse cracks and becoming trapped between the pavement and the cement-stabilized subbase. The stabilization has had the advantage of preventing the loss of significant amounts of subbase material.

This type of distress has progressed to a serious degree in several segments on one contract in the state. All paving was with a slip-form paver employing "spud" vibrators at an unknown spacing and frequency. Also, there are no records of paver speed through the failed sections.

A second type of consolidation problem has occurred at construction joints and has been found to at least a small degree on almost all CRC pavements built in the state. These failures are much less severe than the type described above and are characterized by wide cracks at construction joints and by nearby potholes that are underlaid by honeycombed areas. These failures have emphasized the need for greater attention to vibration at the construction joints, where consolidation is more difficult because the longitudinal steel is doubled. The repair of problems caused by poor consolidation has been time-consuming and costly. In most cases the longitudinal steel is left in place, and new concrete is placed around it. However, the time consumed in chipping the old concrete from the steel has led to long lane closures and high traffic control costs.

The consolidation problems encountered in Virginia, although severe on only one contract, serve to point up the already recognized need for realistic vibration specifications for paving concrete.

Failures Due to Poor Quality Concrete

Several failures on one of the earlier (group A) CRCP projects built in the state resulted from poor quality concrete caused by excessive water-cement ratios. These failures are few in number and usually involve only a few square yards of pavement. The failures are characterized at early ages by closely spaced transverse cracks that are joined by short longitudinal cracks. Deep surface erosion has resulted from relatively few vehicle passes even without the application of de-icing chemicals. To date, none of these areas has been replaced, but several have small temporary bituminous patches.

Since the failed areas tend to represent about 1 batch of paving concrete, it was concluded that poor control of the concrete placed by a drum road mixer was to blame. All subsequent CRC pavement has been from stationary central mix plants, and the problem has not been detected on these projects.

Edge Pumping Due to Poor Drainage

An example of edge pumping due to poor drainage is shown in Figure 10, where staining of the bituminous shoulder is evident. The problem has occurred on the group E projects where the subbase is cement stabilized and the shoulder base material is unstabilized sand and gravel with a 2 to 3 plasticity index. The condition exists primarily at the low point of sag verticals.

Trenching and edge deflection tests showed that no measurable void existed under the CRCP. It was concluded that the pumping resulted from surface water trapped at the interface between the CRCP and the impervious shoulder material. The stains were attributed to fines lost from shoulder material, as was further evidenced by a slight depression of the shoulder at its contact with the CRCP.

Corrective action is to take the form of trenching the shoulder at the pavement edge and backfilling with a highly pervious 1-sized aggregate. Water will be carried to the ditch lines with occasional side drains filled with the same pervious crushed stone.

Design standards are being revised to prevent recurrence of the problem by the provision of nonplastic, pervious shoulder material.

Random Longitudinal Cracking

Since the introduction of slip-form paving using the polyethelene strip to create a weakened plane longitudinal joint, some of the CRC pavement has suffered a modest amount of random longitudinal cracking (group C and D pavements). This experience has been reported by many other agencies using similar construction procedures.

Fortunately, in Virginia the polyethelene tape and cement stabilization of subbases occurred simultaneously so that there is little danger, and no evidence, of faulting of the longitudinal cracks. Another factor making both faulting and the development of excessive crack widths unlikely is the tendency of the random cracks to occur within the limits of the 30-in. (76-cm) long tie bars spanning the center joint.

For these reasons, random longitudinal cracking is not considered to be a serious performance problem in Virginia at this time.

Fill Settlements

Although fill settlements are not strictly a pavement problem, several of the CRCP projects built in the state have been subject to distortions due to such settlements, particularly those directly over minor drainage structures. The structural integrity of the pavements does not seem to be affected by these deformations unless unsound concrete is present. Thus, in almost all cases the pavements conform to the fill settlements, and there is no evidence of the development of unusual cracks.

Such conditions are routinely corrected by mudjacking, and the pavements return to grade with no evident defects. As far as can be determined, the only way to combat these settlements is through careful attention to backfill and embankment construction.

CONCLUSIONS

The following conclusions appear to be warranted from the findings presented in this paper. Some of the pavements are relatively young, and therefore some of the behavior patterns may not have fully developed.

1. CRCP has performed well in Virginia except in cases where design or construction technology has been deficient. These deficiencies have been most noticeable in the areas of pavement consolidation, quality control of concrete, and provision of adequate drainage systems. The need for a strong research effort in the development of consolidation specifications is evident.
2. Cracks show a definite trend to become more closely spaced and to develop greater surface widths during the first 2 to 3 years of the pavement's life. There is some inconclusive evidence that the widening process stabilizes after this time period.
3. Although crack spacing can be influenced by weather conditions during pavement placement, there is no apparent relation between crack width and either crack spacing

Table 7. Changes in crack width from 1970 to 1973.

Paving Date	Summer 1970	Winter 1970-71	Summer 1971	Winter 1971-72	Summer 1972	Winter 1972-73	Summer 1973
May 1970	0	0.0138	0.0034	0.0139	0.0073	0.0147	0.0105
July 1970	0	0.0170	0.0139	0.0305	0.0248	0.0332	0.0186
December 1969	0	0.0185	0.0068	0.0239	0.0160	0.0398	0.0183

Note: Amounts are in inches. 1 in. = 2.5 cm.

Figure 7. Extensometer and standard used for crack measurements.

Figure 8. CRCP failure due to poor consolidation.

Figure 9. Core taken from a poorly consolidated pavement.

Figure 10. CRCP edge pumping due to poor drainage of shoulder material.

or placement weather. Crack spacing seems to be markedly influenced by weather conditions only in the case of extremely low placement temperatures.

4. Crack widths measured at the pavement surface are of the same order of magnitude as those on pavements performing satisfactorily in other states, and the vast majority are within recommended limits for structural members if allowance is made for depth of concrete cover and age. Furthermore, laboratory studies show that no definite crack exists at the surface of the steel for any of several cores examined. Thus, it is concluded that the cracks present no threat of steel corrosion and that the steel is functioning as designed, i.e., to restrain the concrete such that fine, harmless cracks develop at relatively close intervals.

5. The elimination of full-width transverse steel has had no effect on pavement performance for the first 2 or 3 years.

6. Although there is no confirmed effect on performance at this time, deflection tests indicate that cement stabilization of subbases has markedly strengthened pavements built in Virginia. Stabilization may have prevented CRCP damage due to edge pumping and random longitudinal cracking.

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