CONDITION SURVEY OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN THE NORTH CENTRAL UNITED STATES

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This paper summarizes the observations made in a survey of continuously reinforced concrete pavements in the north central United States. This research was part of a larger effort to determine the condition of such pavements. Numerous forms of distress were observed. Data collected on the pavements included design, construction, and performance information. Their behavior is summarized in terms of crack spacing. This survey provided insight for further investigation of those problems found to be significant in nature and of those encountered more than once. Generally continuously reinforced concrete pavements are performing quite well. There are significant problems in some areas that still need to be addressed and solved, and many of the problems noted are those that are common to continuously reinforced pavements. Results show that most crack spacings observed are less than desired. The new problem of tensile failure in longitudinal reinforcement was noted along with significant shoulder distress.

•DURING 1968, several unique failures of continuously reinforced concrete pavements (CRCP) occurred in Minnesota. Although various distress manifestations such as spalling and pavement breakup due to unconsolidated concrete had been noted previously, fracture of the longitudinal steel was noted for the first time. Many of the other problems with CRCP had been associated with poor construction or inadequate thickness, but this represented the first indication of improper design for the longitudinal steel. Steel rusting compounded the problem, since it could not be identified as a secondary or primary effect.

Concern for this problem by the engineers in the Minnesota Department of Highways led to consideration of suspending CRCP construction in that state as well as in other states. This was eventually adopted by Minnesota and other highway departments in the north central United States.

OBJECTIVE

The objective of this study phase was to investigate the problem of wide variations in crack formations and general performance of CRCP. This would provide background for achieving the overall objective of performance, i.e., wide cracks and severe corrosion of some of the CRCP by investigating CRCP in several of the northern states.

SCOPE

This investigation was the first of a two-phased program or evaluation of distress on CRCP. This phase is a field survey and synthesis of the problems existing on CRCP

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and focuses on the problem of corrosion and the rupture of continuous reinforcement. Included were actual site visits and pavement surveys in nine northern states: South Dakota, North Dakota, Minnesota, Wisconsin, Illinois, Iowa, Indiana, Ohio, and Connecticut. The surveys included the gathering of design, construction, and performance information on the selected projects. The primary emphasis of this phase was to provide background for the diagnostic surveys; hence, a detailed analysis of the data was not performed as a part of this study.

COOPERATION OF STATE HIGHWAY DEPARTMENTS

As part of this investigation, nine state highway departments cooperated by participating in the field survey, gathering information from record files, and assisting in the selection of projects to be included in the survey. This project provided a rare opportunity for private industry, government agencies, and a private consultant to cooperate and work together.

FIELD SURVEYS

The pavement survey was objective. The problems of steel failure and corrosion were known to be present in Minnesota but not elsewhere. Therefore, Iowa, North and South Dakota, Wisconsin, and Illinois were added to the project. By the addition of these states, observations could be made on CRCP known to be giving good performance in environments similar to those in Minnesota.

Each of the states included was visited by at least two of the three survey team members. Before making any survey, the survey team met with design, research, construction, and maintenance engineers in each highway department to discuss the objective and scope of the survey. In each case this meeting provided the list of projects to be included in the survey as well as pertinent data relevant to these projects. All surveys were made by the team along with a group of engineers from each state highway department.

Data collection included crack pattern data, photographic coverage, construction data, design data, and performance data where available. The techniques used for this data collection process are described in the following section.

Data Collection

The data were collected from condition surveys made in the field and also from a survey of office records. Before the data were collected, inventory forms were designed to specifically meet the needs of this study. These forms were then used in the field and office surveys.

Data Forms

Data collection was based on the premises as expressed by the following conceptual equation of distress:

$$\text{Performance} = f(\text{material properties, design parameters, construction parameters, maintenance parameters}) \quad (1)$$

Based on equation 1, manifestations that may be used as indicators of performance and the parameters that may affect them were identified from experience and literature reviews. The left side of equation 1 may be developed from condition surveys that give

Figure 1. Data form for performance ratings for each project.

CRCP Performance Survey Form

ARE, INC.
3128 Manor Rd.
Austin, Tex. 78723

Date: ______________
State: ______________

Name: ______________
Pavement Ident: Highway: ______
County: ______________
Location: ______________

PERFORMANCE

Lane								
Sample Location	1	2	3	4	1	2	3	4
Crack Spalling								
Surface Spalling								
Longitudinal Cracking								
Radial Cracking								
Pumping (edge)								
Pumping (cracks)								
Repair Patches								
Distress in shoulder								
Crack width								
Average crack spacing								
Curbed Section (Yes, No)								

Has this pavement been included in prior field research? ______
Are reports and/or data available? ______
Have deflections been measured on this pavement? ______

Ride | very poor | poor | fair | good | very good |
0 1 2 3 4 5

Plan view of typical cracking

Edge view of cracks

General Comments:

Ratings for Condition or Performance
0 - None; 1 - Minor; 2 - Moderate; 3 - Severe

ARE Form 72.2

some quantitative nature to the various distress manifestations observed in the field. The right side of the equation was collected primarily in the office.

Figure 1 shows the data form developed for establishing a performance rating on each project. Ten parameters are listed; the first 8 are distress manifestations ranging from crack spalling to shoulder distress. The nature of these distress manifestations has been established previously by the authors and also by many others. Tensile failures of the longitudinal steel were placed in the general comments section of the survey form. At each location, a rating was made on a four-point scale ranging from 0 to 3, as shown in the bottom of Figure 1. Crack spacing and crack width were also included on the forms, since they may be used as indicators of performance and also as measures of performance.

Provisions were made to obtain an average value for each project by taking measurements at four different locations along the length and to separate the data on the basis of inside and outside lanes. In addition to these data, an estimate of riding quality was made by the survey team riding along the project in a vehicle. The rating scale is identical to that developed by the personnel at the AASHO Road Test.

Figure 2. Data form for detailed data from each project.

CRC Pavement Inventory

ARE, INC.
3128 Manor Rd.
Austin, Tex. 78723

Date: ________ State: ________

Name: ________
Pavement Ident: Highway: ________
County: ________
Location: ________

DESIGN	Concrete	Thickness: Flexural strength: Compressive strength:
		Modulus of elasticity: Cement factor: w/c ratio:
		Cement type: Coarse aggregate:(natural) (crushed) Maximum size:
		Other metals in coarse aggregate: Fine aggregate:(natural) (crushed)
		Concrete Admixtures: (air entraining agents), (Calcium chloride) (other:)
		Curing:(membrane)(plastic sheet)(mat)(other:) No. of days:
		Thermal coefficient: Cement source: Coarse aggregate source:
	Steel	Geologic identification of coarse aggregate:
		Amount (%): Longitudinal Transverse Yield strength:
		Type: (Smooth wire) (Deformed wire) (Rebars) Grade: Bar size:
		Spacing of steel: longitudinal transverse Depth of cover:
		Special treatment on longitudinal joint:
	Subbase	Special treatment on transverse joint:
		Thickness: Stabilized material: (yes) (no) Type:
		Section: (Crown width) (trench) Width: Gradation:
		Strength test: Strength: Modulus of elasticity:
	Subgr.	Grade: (natural) (cut) (fill) Strength test: Strength:
		Stabilization: (Mechanical) (Lime) (Other:)
	General	No. of lanes: Cross slope:(crown)(inclined) No. of 18 kip ESAWL:
		Shoulder: (Paved) (Gravel) (Stone) (Turf) Commercial vehicles/day: ADT:
		Date opened to traffic: Wheel load distribution: (Furnish later)
		Frequency of ice treatment: Rate of application:
		Material used for ice treatment: Location: (Rural) (Urban)
CONSTRUCTION		Date of concrete placement: High temperature:
		Low temperature: Concrete paver: (Slipform) (Conventional form)
		Concrete mix method: (central mix plant) (traveling drum mixer)
		Finish: (Burlap drag) (Belt) (Broom) Vibration: (Surface) (internal)
		Steel Placement: (set on chairs) (double strike off) (mechanical insertion) (other)

General Notes: ________

ARE Form 72.1

Figure 2 shows the data form developed for collecting information on the parameters that may influence the performance parameters. The parameters are divided into design and construction categories. Maintenance parameters are included in general under the design category. The data for these forms were collected in the office by using the plans, construction records, and weather reports.

Office Procedures

In each state, the data collection began with a meeting that presented to the state engineers the objectives and scope of the CRCP inventory study. In this exchange, the projects to be included in the state survey were selected, at least in most cases. For each project surveyed, the forms shown in Figure 1 were completed. Before the projects for the study were selected, many of the data shown in Figure 2 were completed during the office visit and were used to establish an experiment design for each state.

For example, the effect of thickness on performance was to be considered, then sites were selected that had as many other parameters as possible held constant, such as percentage of steel, subbase thickness, and subbase type.

Field Procedures

In the field, crack pattern data were collected at one or more locations on each project. Crack spacing was collected by using a roller tape and by moving down the edge of the pavement and recording the distance from the starting point to the crack. To expedite the operations, the records of crack spacings were dictated into a recorder, and this information was transcribed in the office at a later time. Crack width was measured in some cases by a microscope containing a graduated eyepiece. The average crack width in those sections considered is based on the average of 6 to 10 measurements. Extensive photographic records were also made of the project distress, corrosion, and steel failures on the particular project. A summary of the distress manifestations observed in the field is given in Table 1.

Data Processing

The data collected as part of the survey were transferred to IBM cards. A computer-plotter routine was used to make a crack spacing diagram (Figure 3). All projects were plotted with the same scaling factor so that they could be compared directly when various parameters that may influence crack spacing were considered. The crack spacing was correct at the outside edge of the pavement, and the crack was assumed to be straight across the pavement transversely. Y-cracks were not shown on the spacing diagram because they must be developed from the photographic survey of the pavement.

The crack spacing data were also used to develop a crack frequency distribution curve for the section as shown in Figure 4. In addition to the mean crack spacing, the distribution of the various crack spacings is also given. For example, 30 percent of the cracks are at a spacing of 1.8 ft (0.5 m) or less, and 70 percent are at a spacing of 1.8 ft (0.5 m) or greater. All plots are the same scale and may be compared directly.

Summary of Data

The survey data from each state are summarized in Table 2.

Fifty-four pavements were observed mostly in very cold weather in nine states; failures in longitudinal reinforcement were observed in seven projects and were indicated to be present in others; all actual steel failures were in projects containing deformed wire or plain wire reinforcement in lesser amounts; longitudinal cracking was noted frequently but was believed to be associated with subgrade conditions or performance of the centerline joint; significant spalling was noted only on 7-in. (17.8-cm) CRCP but was generally prevalent to a minor degree; shoulder pavement problems were noted to be quite common; and crack pattern, design, construction, and performance data were obtained from each project.

SYNTHESIS OF FIELD OBSERVATIONS

Crack pattern data obtained from surveys of the pavements were the largest single item of quantitative measure. For all the projects from which such data were obtained, the crack patterns were plotted and summarized as shown in Figure 5. These were plotted from the field measurements. The pertinent observations for each of the pavements included in the survey are given in Table 3. The data and observations from the many pavements were synthesized by considering crack spacing, steel failures, spalling, longitudinal cracking, concrete consolidation, and shoulder distress.

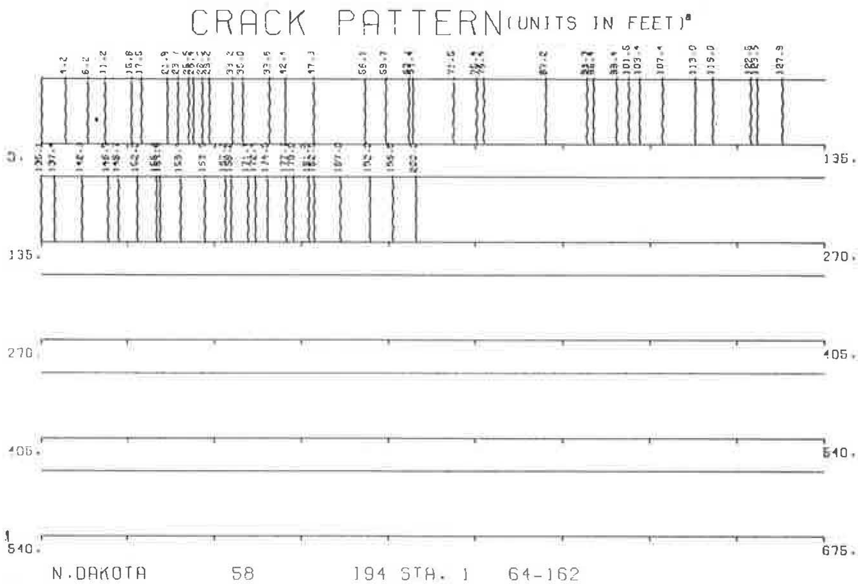
Table 1. Pavement observations.

State	Highway	Job Identification	Slab Thickness (in.)	Steel (percent)	Steel Type[*]	Mean Crack Spacing (ft)	Crack Spacing (standard deviation, ft)	Concrete Strength (psi)	Steel Failures	Spalling	Longitudinal Cracking	Honeycomb	Shoulder Problems
South Dakota	I-90W	192	8	0.6	RB	6.9	3.5						
South Dakota	I-90W	185	8	0.6	RB	11.1	4.15	665					
South Dakota	I-90E	395	8	0.646	RB	1.3	1.4	738					
South Dakota	I-90E	222	8	0.6	RB	5.6	3.2						
South Dakota	I-29N	110	8	0.6	RB	4.8	3.4	677		x			
South Dakota	I-29N	117	8	0.6	RB	2.2	1.5	647					
South Dakota	I-29N	134	8	0.6	RB	6.6	3.3	673					
North Dakota	I-94E	5(16)	7	0.7	RB	2.9	1.7				x		x
North Dakota	I-94W	4(24)	8	0.6	RB	3.4	2.0				x		x
Minnesota	I-94	2781-76	9	0.6	DWF	3.0	2.4	698	x	x			
Minnesota	I-494	2785-12	9	0.65	RB	2.8	1.2	625		x			x
Minnesota	I-35W	6284-28	9	0.6	DWF	1.8	1.2	821					
Minnesota	I-35W	0280-06	8	0.6	DWF	2.4	1.9	604	x	x			x
Minnesota	I-35E	6280-56	9	0.7	RB	2.9	2.2	527				x	
Minnesota	I-35E	6281-04	8	0.7	RB	3.4	2.0	688					
Minnesota	I-90E	5380-12	8	0.6	DWF	4.0	2.4	641	x				
Minnesota	I-90E	5380-10	8	0.7	RB	2.5	1.6	735					x
Minnesota	I-694	8286-17/ 6286-04	9	0.7	RB	1.9	1.0	636					x
Minnesota	I-694	8286-14	9	0.65	RB	2.3	1.2	604					x
Minnesota	I-94	6283-25	9	0.65	DWF	2.0	1.4	691					x
Wisconsin	I-90	Dane Co.	8	0.65	RB	4.3	2.4	673					x
Wisconsin	I-90	Juneau Co.	8	0.65	RB	3.3	1.9	548					x
Iowa	I-80	Dallas Co.	8	0.625	DWF	3.7	2.2	579	x	x		x	
Iowa	US-30	Marshal Co.	8	0.646	RB	2.0	1.2	628					
Iowa	I-35		8	0.646	RB	3.5	2.2						
Iowa	I-35	5(18)	8	0.646	RB	2.9	1.7			x			
Iowa	I-35	5(18)	8	0.646	RB	6.7	7.0	584					
Illinois	County Route 196	Section F	7	0.6	RB	5.5	2.8	813		x			x
Illinois	County Route 196	Section B	7	0.6	DWF	5.5	2.7	795		x			x
Illinois	County Route 196	Section G	8	0.6	DWF	5.2	3.9	760		x			x
Illinois	County Route 196	Section K	8	0.6	RB	4.6	3.1	800		x			x
Illinois	I-74	18	7	0.7	RB	2.6	1.4	975		x			
Illinois	I-74	17	7	0.7	DWF	3.4	1.9	836		x	x		
Illinois	I-74	16	7	0.7	RB	3.4	2.8	699		x			
Indiana	I-65	R-7633											
Indiana	I-65	R-7677											
Indiana	I-65	R-7913											
Indiana	I-65	R-7715											
Indiana	I-65	R-7782											
Indiana	I-65	R-7858											
Indiana	I-65	R-7935											
Indiana	I-65	R-8208											
Indiana	I-65	R-8232											
Indiana	I-65	R-7529											
Indiana	I-465	R-7596											
Indiana	I-465	R-7276											
Indiana	I-465	R-7391											
Indiana	I-465	R-7841											
Ohio	I-70	780(66)	8	0.61	DWF					x	x		
Ohio	I-70	748(66)	8	0.61	DWF					x			
Ohio	Ohio-16	178(64)	8	0.6	DWF	3.0							
Ohio	Ohio-16	West of Newark	8	0.6	RB	3.0							
Connecticut	I-84	Farmington-Plainville	8	0.55	DWF	6.9			x				
Connecticut	I-84	CRCP Test Road	8	0.61	RB	4.5							
Connecticut	I-84	CRCP Test Road	8	0.58	DWF	6.1			x				

Note: 1 in. = 2.5 cm, 1 ft = 0.3 m, 1 psi = 6.9 kPa.

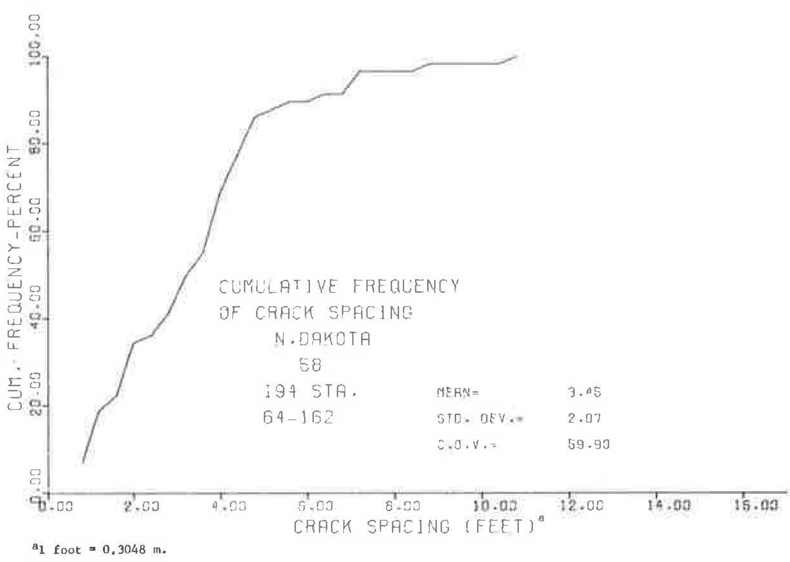
*RB = reinforcing bars; DWF = deformed wire fabric.

Figure 3. Crack spacing diagram.



[a]1 foot = 0.3048 m.

Figure 4. Crack spacing frequency distribution curve.



[a]1 foot = 0.3048 m.

Crack Spacing

Based on data summaries similar to those shown in Figure 5, the crack spacing was compared for the two types of steel over a range of concrete slab thickness and percentages of longitudinal reinforcement. Table 3 gives a summary of the average crack spacings based on steel percentage, slab thickness, curing temperature, and steel type. It is apparent from the data that there is no particular trend relative to reinforcement type, i.e., pavements with either type reinforcement having a consistently higher or lower average crack spacing than pavements with the other type of reinforcement. This

Table 2. Survey data.

State	Projects Observed	Design	Problems: Distress in Longitudinal Steel	Problems: Longitudinal Cracking	Problems: Other
South Dakota	7	Same, except some had limestone concrete aggregate and others had quartzite	None	Some in 7-in. pavement	
North Dakota	2	Same, except thickness	None	Significant in 7-in. pavement	
Wisconsin	2	Same, except subbase	None	None	
Minnesota	10		Tensile failures in pavements with lesser reinforcement		Deformed wire reinforcement in projects made by using double-strike-off method of concrete and steel placement
Iowa	5	Same, except reinforcement	Failures on project with deformed wire reinforcement	Some	
Illinois	7		None		Significant spalling on 7-in. pavement
Indiana	14	Differences in granular and treated subbase layers, steel variations of deformed bars and wire			Localized failures, punchouts, unclassified and wide cracks
Ohio	4	Same, except some had granular subbases and one had bituminous-treated subbase		Some	Localized failures, wide transverse cracks
Connecticut	3	Differences in deformed bars and wire and plain wire			Tensile failures noted in plain and deformed wire-reinforced pavement

Note: 1 in. = 2.5 cm.

Figure 5. Sample crack pattern summary.

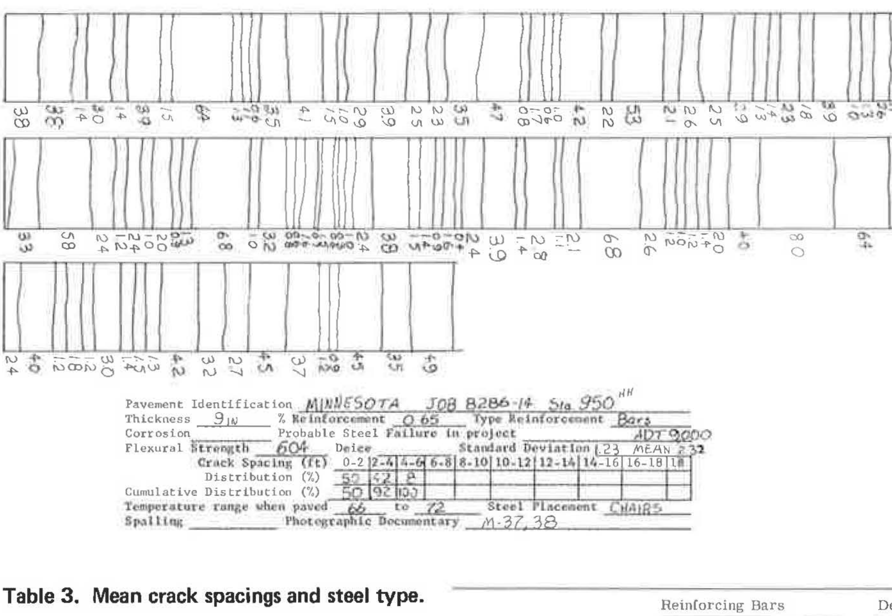


Table 3. Mean crack spacings and steel type.

Steel (percent)	Concrete Thickness (in.)	Reinforcing Bars: Crack Spacing (ft)	Reinforcing Bars: Temperature Difference[a] (deg F)	Deformed Wire: Crack Spacing (ft)	Deformed Wire: Temperature Difference[a] (deg F)
0.6	7	7.07	28	5.52	21
	8	4.64	29	5.23	15
0.65	9	2.78	13	1.97	14
	9	2.32	6	1.97	14
	9	2.78	13	2.90	5
	9	2.32	6	2.90	5
0.7	7	3.39	24	3.44	28

Note: 1 ft = 0.3 m. 1 in. = 2.5 cm. 1 F = 1.8 (C) + 32.

[a]Between high and low temperature at time of construction.

indicates that there is no particular difference in the average crack patterns that develop that may be attributable to the type of steel, i.e., deformed bars or deformed wire fabric.

The crack spacings have been summarized to compare various slab thicknesses. Table 3 gives the constant trend of reduced average crack spacing with increased pavement thickness. This trend is logical and has been noted on previous investigations of CRCP (1, 2). It was unfortunate that the factorial shown in Table 4 was not complete over the range of thickness for any one steel type and percentage.

The temperature at the time of concrete placement has been related to the development of the cracking pattern (6, 7). Similarly in this survey, it was noted where the average crack spacing was also related to the air or curing temperature. Figure 6 shows data from pavements with different types of aggregate in Iowa and South Dakota. Fundamentally, the three curves in Figure 6 show concrete with different strengths, thermal coefficients, and elastic properties.

Crack Pattern

An ideal crack pattern would be a uniform crack spacing with no deviation. Figure 7 shows an ideal crack pattern plotted on a frequency distribution curve. Thus, the curve is a vertical line that shows 100 percent of the cracks to be 6½ ft (2 m). In some of the early development work for continuous pavement, the designers were attempting to achieve a crack pattern between 5 and 8 ft (1.5 and 2.4 m). The average for this range is 6½ ft (2 m), as shown in Figure 7. It is recognized that an absolute uniform crack spacing was an impossibility, but the earlier designers had hoped for a crack spacing range of 5 to 8 ft (1.5 to 2.4 m) shown as the desired crack pattern on Figure 7. McCullough (8) in an investigation of deflections on in-service pavements found that the desirable load transfer characteristics were achieved with this range of crack patterns. In a later study using discrete element techniques, Hudson, Treybig, and Ayyash (9) found that minimum stress levels in a pavement were achieved with the crack pattern in this range as well. It was speculated that, when the crack spacing is greater than this range, the crack width will be excessive, thus decreasing the load transfer across the crack. On the other hand, crack spacings less than this range generally produce excessive deflections if soft spots are present in the subbase or the subgrade. This can lead to pumping, longitudinal cracking, and breakup of the slabs under traffic loadings.

A study of the crack spacing frequency diagrams using Figure 7 as a guide leads to several interesting observations. In a few cases, crack patterns are similar to the desired level achieved on the projects surveyed. In most cases, the crack patterns fall far to the left, given an average crack spacing smaller than the desired level. The designer has several options available to shift the crack pattern frequency to the right. Among the most evident are increasing the concrete strength, reducing the percentage of longitudinal steel, and reducing the bond area ratio of longitudinal bars. It is not possible to reduce the amount of steel to minimize crack width. Therefore, the designer's options in this area are to go to stronger concrete or use larger bars at a greater spacing to achieve the same steel percentage.

Steel Failures

Tensile failures were observed on a total of seven projects. Two of these seven projects were reinforced with plain welded wire fabric (4, 5). Fabric style was not part of this survey, but since failures were noted on such pavements, it is significant relative to other observations. The other five projects that were observed to have longitudinal steel failures were all reinforced with deformed welded wire fabric. All steel failures noted were in pavements with either plain or deformed welded wire fabric. The projects containing the failures were all 4 to 5 years old and contained approximately 0.6 percent longitudinal steel. They were all on 8-in. (20.3-cm) slabs. CRCP that were 9 in. (22.8 cm) thick with 0.6 percent steel the same age as that in the 8-in. (20.3-cm) slabs had

Table 4. Mean crack spacings and concrete thicknesses.

		Mean Crack Spacing (ft)		
Type	Steel (percent)	7-In. Concrete	8-In. Concrete	9-In. Concrete
Reinforcing bars	0.6	7.07	4.64	
	0.7		3.35	2.94
			3.35	1.93
Deformed wire	0.6	5.52	5.53	
			2.38	1.82

Note: 1 ft = 0.3 m, 1 in. = 2.5 cm.

Figure 6. Average crack spacing versus temperature when concrete was placed.

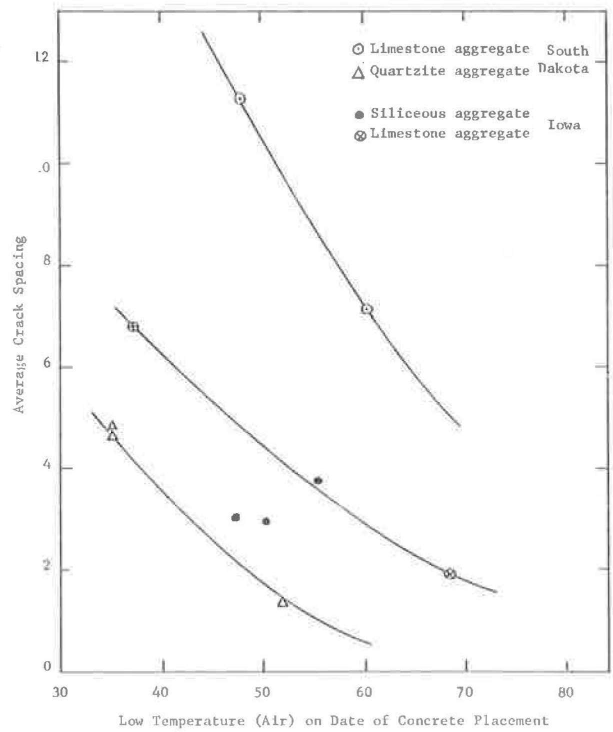
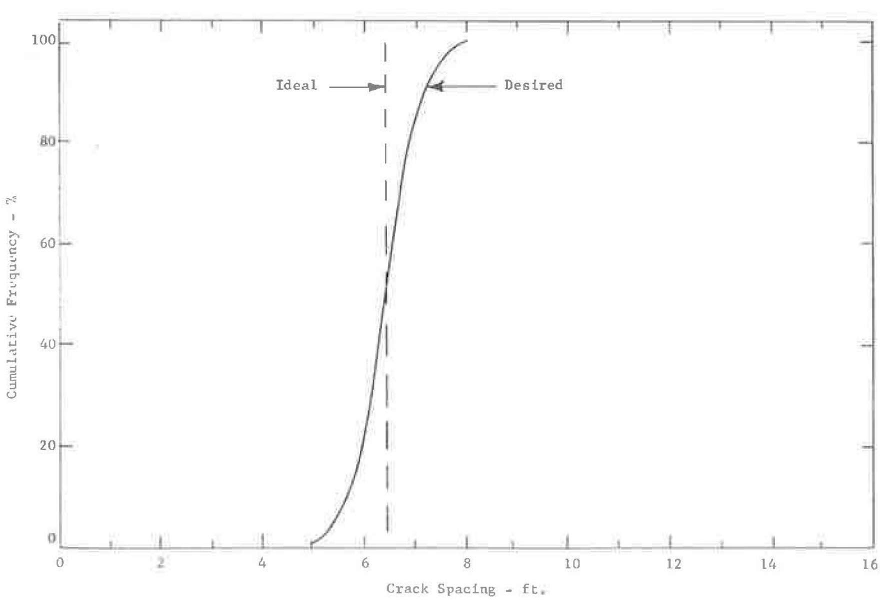


Figure 7. Ideal and desired crack spacings as proposed in early CRCP design developments.



not experienced any failures at the time of this survey. Thus, 8-in.-thick (20.3-cm) CRCP with greater than 0.6 percent deformed wire fabric had not shown failure. CRCP that were 9 in. (22.8 cm) thick with 0.6 percent steel [a larger cross-sectional area than the 8-in. (20.3-cm) slab] also did not show failure.

Corrosion seems to be the apparent problem, and time will tell the fate of the 9-in. (22.8-cm) slab with 0.6 percent steel and of slabs with larger percentages of steel.

Spalling

Severe spalling was noted on 7-in.-thick (17.8-cm) CRCP. Based on the observations and historical data on the pavement, it is surmised that the load deflections experienced by the pavement are greater than those allowable, thus causing surface spalling along the cracks. This same phenomenon has been experienced elsewhere on CRCP (3). Some minor spalling was noted on 8-in. (20.3-cm) pavements that were also apparently experiencing excessive deflections, since these pavements had only 4 in. (10.2 cm) of sand or gravel subbase on a weak subgrade.

Figure 8 shows the presence or absence of spalling in terms of average crack spacing by state. If spalling of any type was noted on a given project, it was plotted on the left scale. If spalling was completely absent, then it was plotted on the vertical scale to the right. Figure 8 indicates that spalling cannot be tied solely to average crack spacing, since spalling is present on sections with crack spacing as small as 1½ ft (0.5 m) and as large as 11½ ft (3.5 m). The most significant observation that may be made is that spalling was noted on all Illinois pavements. It should be pointed out that these pavements are generally older and have experienced a larger total of accumulative 18-kip (80-kN) single-axle loads. This emphasizes that spalling may be attributed to the number of wheel load repetitions, although possibly other parameters establish the condition that allows the spalling to occur.

Longitudinal Cracking

Significant longitudinal cracking was noted in three states. The longitudinal cracking observed probably has not and will not affect the pavement performance significantly since pavements observed did contain transverse reinforcement. The longitudinal cracking ing appears to be related to the subgrade conditions. It seems to occur on fill sections, particularly when the pavement cross section is partially cut and partially fill. The cracks have been located on the fill side of such sections. It is possible that some of the longitudinal cracking observed was due to late sawing of longitudinal centerline joint or the insufficient depth of the centerline joint. The presence of longitudinal cracking under some conditions brings the transverse steel into play. In most instances these cracks are tight and have not affected performance. However, if steel were not present, these slabs may have experienced severe distress if the longitudinal crack opened up.

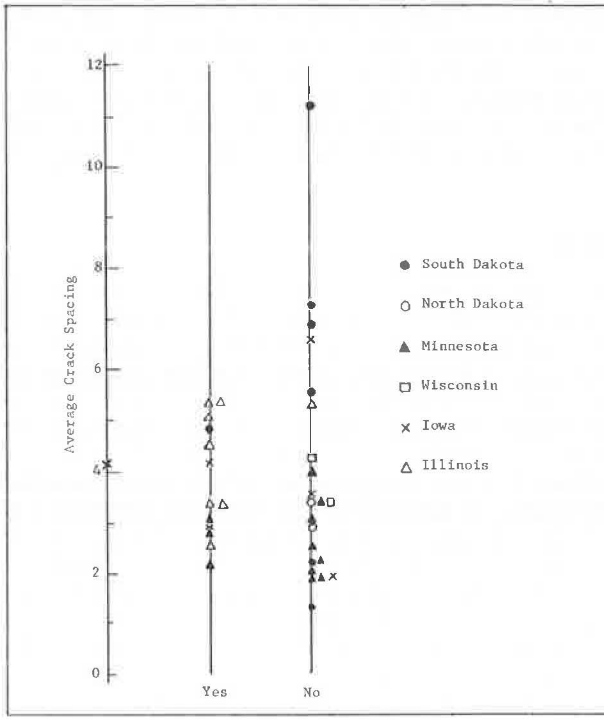
Concrete Consolidation

Concrete consolidation was found to be a problem in two states. The problem of localized failure has been minimized by most agencies through the use of updated construction specifications requiring adequate vibration equipment as well as desired levels of vibrator frequency and amplitude. The absence of distress manifestations due to poor concrete consolidation in Ohio and Indiana, where special precautions were taken based on experience in other states, emphasizes the preceding observation.

Shoulder Distress

The presence of distress or failures in the paved shoulders along the CRCP observed

Figure 8. Concrete spalling at transverse cracks as a function of crack spacing.



in this investigation was quite common. Excluding Indiana, Ohio, and Connecticut, which were not a part of the detailed survey, over 40 percent of the pavements had some type of shoulder problem. This indicates that shoulder pavement design and performance may seriously affect the general behavior and performance life of a continuous pavement without a crown-width blanket, impervious, stabilized subbase. There were no attempts to differentiate between the various shoulder distress modes and manifestations. This survey merely documents that there is a CRCP shoulder design problem that may warrant investigation.

Summary

Most of the observed distress manifestations were obviously tied to design or construction deficiencies. This emphasizes the need for improvement in techniques in these areas to prevent this problem from recurring. The existence of the tensile failures of longitudinal resistance in only deformed wire fabric and its absence in the deformed bars lead to the need for a diagnostic study to ascertain the causes of the failures.

CONCLUSIONS AND RECOMMENDATIONS

1. The pavement thickness, percentage of steel, coarse aggregate type, and concrete curing temperature have a greater influence on crack spacing distribution than steel type has; i.e., deformed wire fabric and deformed bars. The crack spacing distributions for deformed wire fabric and deformed bars were almost identical when all other variables were similar.
2. The crack spacing distributions observed on most of the projects considered in this study are substantially less than the desired 5 to 8-ft (1.5 to 2.4-m) range envisioned in the original design.

3. Tensile failures of the longitudinal steel were observed on seven projects containing wire fabric. Two of these seven projects were reinforced with plain welded wire fabric, and five were reinforced with deformed wire fabric. Tensile failures of the longitudinal steel were not observed on deformed bars.

4. Although minor spalling was noted at many locations in the study, the only case of extreme spalling was with pavement thicknesses of 7 in. (17.8 cm) or less. It appears that when a spalling condition is established, the amount of spalling is a direct function of the cumulative equivalent 18-kip (80-kN) single-axle loads.

5. Longitudinal cracking was observed on a number of projects. In all cases, transverse steel was present, and the longitudinal cracks were not detrimental to the performance of the pavement.

6. Over 40 percent of the projects considered had some type of shoulder distress. It is apparent that this may have an eventual effect on the performance of CRCP.

Based on these conclusions, the following recommendations are made:

1. A diagnostic study should be initiated immediately to ascertain the causes of the distress mechanism producing the tensile failures of the longitudinal steel with the deformed wire fabric. Since the distress mechanism is unknown at this time, it is difficult to take this factor into account in design until it is defined.

2. Designers should possibly consider larger bars or higher strength concrete to achieve a more desirable crack pattern. This consideration must be balanced against the other constraint of not achieving too large a crack pattern so that load transfer is lost across the cracks.

3. Use of concrete shoulders may be beneficial in reducing shoulder failures.

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DISCUSSION

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Since the objective of the study reported on in this paper included the general performance of CRCP in several of the northern states, I consider it worthwhile to comment briefly on experience with this type of pavement in a more northerly geographic region.

After a conventional concrete test section was constructed in 1955 to compare performance of CRCP in other parts of Canada and the United States with performance in Alberta, further test sections of various concrete pavement types involving different thicknesses, variable reinforcing, and joint spacing were constructed in 1958 on the Trans-Canada Highway near Calgary, Alberta (10). These test sections provide information on CRCP pavements under conditions beyond those reported in the literature.

The CRCP sections were 24 ft (7.3 m) wide and about 1.8 miles (2.8 km) long with two 0.5-mile-long (0.8-km) sections of 7-in. (17.8-cm) slabs with 0.71 and 0.78 percent longitudinal steel consisting of hard-grade deformed steel bars having a yield strength of 55,000 psi (379 MPa). Two slightly shorter sections were 6 in. (15.2 cm) thick with 0.72 and 0.82 percent longitudinal steel. The bridge type of steel tongue-and-groove expansion dams were placed 1.8 miles (2.8 km) apart at the extremities of the continuous slabs. Graded crushed-gravel subbase 4 in. (10.2 cm) in depth separated the slabs from a compacted subgrade consisting of inorganic clays and silty clays of low to medium frost susceptibility. The estimated subgrade modulus k value allowing for spring loss in strength was 170 lb/in.^{3} (4.7 Gg/m^3). Concrete flexural strength requirements called for a minimum of 550 psi (3792 kPa) at 10 days and 650 psi (4482 kPa) at 28 days. Actual strengths achieved were generally higher than 640 and 730 psi (4413 and 5033 kPa) respectively as average values for the CRCP sections. Average 28-day compressive strength was 4,750 psi (32 750 kPa). The area presents a large range of climatic conditions with the highest recorded July temperature of 97 F (36 C) and lowest January temperature of -46 F (-43.3 C).

The cracking pattern developed to about 10 to 22 cracks per 100 ft (30.5 m) after the first 3 years, resulting in an average spacing from 10 to 4.5 ft (3.1 to 13.7 m). About half the cracks developed after the first 2 weeks of construction (11). Construction was done from September 26 to October 21, 1958, for the CRCP sections, during which time the daily high and low temperatures ranged generally from 70 F to 30 F (21 to -1.1 C), although extremes of 78 and 18 F (25.6 and -7.8 C) were observed. After the third winter, there was still no evidence to indicate any differences in performance among the various types of CRCP sections. Periodic surveys of pavement condition and performance were conducted by the Highways Division of the Alberta Research Council for 10 years, and there were no unusual observations.

From June to July, 1968, three blowups occurred in the concrete pavement test sections, two in the CRCP, and one in an adjacent section of 8-in.-thick (20.3-cm) unreinforced concrete with 20-ft (6.1-m) transverse joint spacing. Both of the blowups in the CRCP were in the 6-in.-thick (15.2-cm) section with 0.72 percent longitudinal steel. The first occurred at a construction joint location where yielding of the steel had been observed during the first winter, and the second occurred at a location where the cracking frequency was extremely high, on the order of 3-ft (0.9-m) spacings. Between these two blowups, which occurred approximately at each end of a horizontal curve, the average crack spacing was 4.3 ft (1.3 m). The air temperatures before the first two blowups were not unusual; however, the third was preceded by a prolonged warm spell with a maximum of 91 F (32.8 C) and a minimum of 53 F (11.7 C) on the day of failure. Cores taken showed the full-design depth of concrete with the reinforcing bars at approximately middepth of the slab in the vicinity of the blowup.

Although there seems to be no single specific reason for the occurrence of these blowups about 10 years after construction, undoubtedly a combination of circumstances contributed to the buildup of sufficient compressive stresses in the slabs. Overzealous attempts by maintenance personnel to seal the fine transverse shrinkage cracks with asphalt may have had some influence, although the comparatively thin 6-in. (15.2-cm) slab and large ranges in maximum and minimum summer and winter temperature would contribute to the buildup of compressive stresses because of infiltration of debris into the cracks.

Despite the stress relief afforded by these two blowups in the CRCP in 1968, several others have occurred in following years, and this has resulted in CRCP being viewed with extreme caution for the particular conditions in Alberta, in view of the performance to date.

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