Nationwide Field Investigation of Continuously Reinforced Concrete Pavements

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There are over 48,300 km of continuously reinforced concrete (CRC) pavements in the United States. Many of these pavements are more than 20 years old and have provided excellent performance over the years. Much of the CRC pavement technology has developed through experience. This and the recent use of new design features (i.e., tied concrete shoulder, permeable cement treated base, and epoxy-coated steel) identified a need to conduct a study to evaluate performance of existing CRC pavement sections. A national pooled-fund study administered by FHWA aimed at updating the state of the art of the design, construction, maintenance, and rehabilitation of CRC pavements was recently completed. As part of the study, a comprehensive field investigation of 23 in-service CRC pavements was conducted to study the effects of various design and construction features on performance of CRC pavements. The investigation included crack mapping and distress survey, profile and roughness measurement, falling weight deflectometer testing, and materials sampling and testing for 305 m long in-service test sections. The key findings of the field investigation program as they relate to CRC pavement design and construction are presented.

Continuously reinforced concrete (CRC) pavement is portland cement concrete (PCC) pavement with continuous longitudinal steel reinforcement with no intermediate transverse expansion or contraction joints. The continuous joint-free length of CRC pavements can extend to several thousand meters with breaks provided only at structures. Terminal anchorage is provided at the ends of the CRC pavement to restrain length changes due to temperature variations and drying shrinkage of concrete. The CRC pavements develop a random cracking pattern with cracks generally spaced at about 0.9 to 2.4 m. The cracking pattern is governed by the environment conditions at the time of construction, the amount of steel, and concrete strength. The steel reinforcement restrains the opening of the cracks. Also, the higher the amount of steel reinforcement, the more closely spaced the cracks will be. Most of the cracks form shortly after construction but additional cracking may develop over the next few years as a result of continued drying shrinkage of concrete, temperature variations, and traffic loading.

A major concern with CRC pavement is punchout distress. Other distresses associated with punchouts include spalling along transverse cracks and faulting. Other leading causes of CRC pavement failure are wide (and spalled) transverse cracks due to steel rupture and spalling of concrete due to steel corrosion in the presence of heavy deicing salt applications in the northern states. The punchout distress is related to crack spacing, pavement thickness, poor foundation support, and heavy truck loadings. The repair of punchout distress typically consists of full-depth patches. With time, as the number of full-depth patches increases, the pavement may be resurfaced with asphalt concrete or PCC or it may be reconstructed.

This paper presents the results of a field investigation conducted as part of a recent study administered by FHWA aimed at updating the state of the art of the design, construction, maintenance, and rehabilitation of CRC pavements (*1,2*). Because CRC pavement performance is influenced significantly by crack spacing, the data analysis and evaluation were focused on a more detailed review of the crack-spacing–related data.

FIELD INVESTIGATION DETAILS

The specific objective of the field investigation was to conduct necessary field investigations and laboratory testing of existing CRC pavement sections and to evaluate the effect of standard and new design features on CRC pavement performance. After a detailed evaluation of available project sites in conjunction with participating state highway agencies, 23 project sites were selected. At each site, performance of a representative 305-m-long section was evaluated using visual condition surveys, profile measurements, falling weight deflectometer testing, and corrosion-related testing. In addition, concrete cores were obtained for strength, stiffness (modulus of elasticity), and coefficient of thermal expansion testing. Samples of base, subbase, and subgrade were also obtained for material characterization. For each project site, available inventory-type data related to design, construction, maintenance, performance, and traffic were collected from state agencies.

Test Section Details

The list of the test sections selected for field evaluation is given in Table 1. As shown in Table 1, the selected test sections incorporate a broad range of attributes of interest:

- Design thickness—ranging from 203 to 330 mm,
- Epoxy-coated reinforcement—three sections,
- Permeable base—two sections,
- Age—ranging from 0.3 to 22 years,
- Subgrade—both coarse and fine-grained soils,
- Base—CTB, LCB, ATB, and granular,
- Steel amount—0.45 to 0.7 percent,
- Steel placement—tube fed and chairs,

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TABLE 1 Final List of Test Sections

Test Section ID	Route	Age as of Fall 1991 Testing, years	Climatic Region	Terminal Joint Type	Design Thickness, mm	Subgrade Type (AASHTO)	Base Type	Outside Shoulder Type	Long. Steel Amount, %	Steel Placement Method	Epoxy Coated Steel	1,991 2-Way AADT	Design Lane Cumul. ESALs upto 9/91
IL-1	US51	0	wet-freeze	wide flange	254	A-7-6	perm. ctb	pcc	0.70	chair	no	na	180,000
IL-2	I72	15	wet-freeze	lug	203	A-6	ctb	ac	0.59	tube	no	7,500	4,800,000
IL-3	US36	20	wet-freeze	lug	203	A-7-5	atb	ac	0.60	chair	no	17,700	4,800,000
IL-4	I55	20	wet-freeze	lug	203	A-7-5	atb	ac	0.60	tube	no	17,700	13,700,000
IL-5	US50	5	wet-freeze	wide flange	203	A-7-5	lcb	pcc	0.70	chair	no	na	300,000
IA-1	I29	20	wet-freeze	lug	203	A-2-6	ctb	ac	0.65	tube	no	7,500	3,700,000
IA-2	I80	22	wet-freeze	lug	203	A-6	atb	ac	0.65	tube	no	12,700	8,850,000
IA-3	I380	15	wet-freeze	lug	203	A-6	atb	pcc	0.65	tube	no	27,700	5,300,000
OK-1	I40	4	wet-no freeze	wide flange	229	A-6	atb	pcc	0.50	chair	no	13,000	na
OK-2	US69	5	wet-no freeze	wide flange	229	A-6	atb	pcc	0.50	chair	no	8,000	na
OK-3	I35	3	wet-no freeze	wide flange	254	A-4	atb	pcc	0.50	chair	yes	21,000	na
OK-4	US69	7	wet-no freeze	wide flange	229	A-6	soil-asphalt	pcc	0.50	chair	no	9,000	na
OK-5	I40	2	wet-no freeze	wide flange	254	A-2-6	perm. ctb	pcc	0.61	chair	no	12,000	na
OR-1	I5	7	wet-no freeze	wide flange	330	A-4	granular	ac	0.60	tube	no	29,700	11,300,000
OR-2	I5	4	wet-no freeze	wide flange	254	A-4	ctb	ac	0.60	tube	no	30,300	3,000,000
OR-3	I205	20	wet-no freeze	lug	203	A-6	ctb	ac	0.54	tube	no	59,000	30,000,000
PA-1	I180	15	wet-freeze	wide flange	229	A-2-4	granular	ac	0.45	tube	no	9,000	5,500,000
PA-2	I81	22	wet-freeze	lug	229	A-2-4	granular	pcc	0.55	chair	no	13,700	32,000,000
WI-1	I43	18	wet-freeze	lug	203	A-2-4	granular	ac	0.65	chair	no	10,900	2,290,000
WI-2	I90	6	wet-freeze	wide flange	254	A-4	granular	pcc	0.67	tube	yes	31,400	2,530,000
WI-3	I90/94	7	wet-freeze	lug	254	A-4	granular	pcc	0.67	tube	yes	35,100	3,960,000
WI-4	I90/94	7	wet-freeze	na	254	A-2-4	granular	pcc	0.67	tube	no	42,600	4,180,000
WI-5	I90/94	16	wet-freeze	lug	203	A-1-a	granular	ac	0.61	chair	no	26,900	na

Note: ESALs = 80 kN equivalent single-axle loads.

- Shoulder type—11 asphalt concrete and 12 PCC, and
- Climatic region—wet-freeze (15 sections) and wet-no freeze (8 sections).

All field testing was performed during fall 1991.

Field Data Collection and Analysis Plan

The field data collection program was aimed at collecting data on the current condition of each 305-m-long representative test section. The following activities were completed at most of the test sections:

1. Visual condition survey
   - –Crack and distress mapping along the 305-m section,
   - –Joint width measurements (for cracks located within a 30.5-m subsection), and
   - –Windshield survey of adjacent 8.1 km of pavement;
2. Nondestructive deflection testing using a falling weight deflectometer (FWD)
   - –Basin testing (slab interior—midslab between cracks) at a spacing of 87.6 m along the 305-m length of the section, and
   - –Testing at crack locations (midslab and edge) (for cracks located within a 30.5-m subsection);
3. Profile testing using a South Dakota–type profiler
4. Corrosion-related testing
   - –Corrosion potential measurement, and
   - –Examination of steel bars (cores);
5. Coring and shallow borings
   - –Concrete testing (laboratory testing): splitting tensile strength, modulus of elasticity, coefficient of thermal expansion, and chloride content determination.
   - –Material characterization: Atterberg limits and particle size distribution.
6. Reinforcing steel location survey
7. Photographic and video imaging

All field testing was accomplished during 1 day of testing. The details of field data collection procedures used are given elsewhere (*1*).

Deflection Testing Data Analysis

Deflection data from the basin testing were used to backcalculate the radius of relative stiffness l, modulus of subgrade reaction k, and slab rigidity D, for the pavement at each test location. l and D are defined as follows:

$$l = [(Eh^3/12(1 - \mu^2)k]^{0.25}$$

where

E = concrete modulus of elasticity,
h = slab thickness,
μ = concrete Poisson's ratio, and
k = modulus of subgrade reaction.

and

$$D = Eh^3/12(1 - \mu^2)$$

Program ILLI-BACK (*3*) was used for this purpose. The backcalculation was performed for all three load levels used. Backcalculated data indicate that the radius of relative stiffness values computed for each of the three load levels at each test location were almost identical. Therefore, in subsequent data analysis, only the data for the nominal 9,000-lb load were used. For edge testing, no corrections were made for the boundary conditions (free edge). Thus, the backcalculated values computed using ILLI-BACK actually represent the effective or equivalent radius of relative stiffness, modulus of subgrade reaction, or slab rigidity, as appropriate.

Data Analysis for Deflection Testing at Cracks and Along Edge

A review of the data obtained from deflection testing at cracks indicated that edge deflections were almost twice as large as midslab deflections for early morning and midafternoon testing. However, the backcalculated radius of relative stiffness along the edge was not always proportionately less than along the midslab. Thus, care must be exercised in interpreting the backcalculated relative stiffness values without considering maximum deflections for edge testing. The radius of relative stiffness values was backcalculated without accounting for the edge boundary condition (free edge or tied shoulder). Thus, these values represent effective values and are used primarily to allow comparison of overall pavement stiffness along the edge to the overall pavement stiffness along midslab (interior) locations and to identify whether tied shoulder has any effect on the overall pavement stiffness along the edge. Also, as expected, afternoon testing produced lower deflections at the midslab and edge locations. In early morning, the slab edge is curled upward because of a cooler slab surface resulting in a slight loss of support along the free edges. In midafternoon, the reverse is true and the slab edge is either in contact with the base and subbase or is close to contact because of the downward curl along the slab edge.

Summary of Test Data

One of the major concerns at the beginning of the field study was the availability and reliability of data related to traffic along the test sections. Reliable traffic data were unavailable for many of the test sections—in most cases because the reliable traffic data did not exist or the required traffic data e.g., ESALs were not maintained by the agency. This is not unusual; the same problem has been encountered on many similar pavement data collection programs, including the Long-Term Pavement Performance (LTPP) program. For the LTPP program, the state agencies have initially provided the best estimates of the ESALs for the test sections, and efforts are under way to perform more in-depth traffic data collection using site-specific WIM and AVC equipment. Thus for this project, traffic effects are indirectly incorporated by considering age (time) effects. A summary of the key data elements for each of the 23 test sections is presented in Table 2.

Ride Quality and Serviceability

The ride quality of the CRC pavement test sections as denoted by the international roughness index (IRI) ranged from a low of

TABLE 2 Key Data Elements

Test Section ID	Average Crack Spacing, meter	Average IRI, m/km	Average Max. Deflection, mm			Edge Deflection as % of Basin Defl. (afternoon)	Measured E, MPa	Average Split Ten. Strength, MPa	Basin Test l Average, mm	Afternoon Edge Crack l Average, mm	Edge Crack l as % of Basin l (afternoon)	Basin Test k Average, kPa/mm	Basin Test D Average, kN-m	Afternoon Edge Crack k Average, kPa/mm	Afternoon Edge Crack D Average, kN-m	Edge Crack k as % of Basin k (afternoon)	Edge Crack D as % of Basin D (afternoon)
			Basin Testing	Morning Edge Crack Testing	Afternoon Edge Crack Testing												
IL-1	1.6	1.47	0.056	0.132	0.097	53	37,206	3.38	1,016	813	24	78	832,389	74	323,189	95	39
IL-2	1.3	2.01	0.109	0.305	0.277	77	39,273	3.98	940	635	21	54	423,186	45	73,209	83	17
IL-3	1.1	2.40	0.124	0.254	0.236	58	33,761	4.15	965	991	31	44	383,721	21	198,502	47	52
IL-4	0.6	2.48	0.099	0.201	0.175	54	29,627	3.25	1,067	991	28	45	579,662	28	269,023	62	46
IL-5	0.9	2.23	0.112	0.236	0.168	46	33,761	3.33	965	991	31	49	421,387	29	282,082	60	67
IA-1	1.8	1.14	0.104	0.175	0.135	39	30,316	3.33	1,016	838	25	49	526,024	49	243,681	100	46
IA-2	0.9	1.30	0.127	0.340	0.244	59	28,249	3.51	1,041	1,016	30	37	440,259	18	196,536	49	45
IA-3	0.9	1.86	0.107	0.196	0.231	66	35,828	3.86	940	889	29	53	414,722	26	162,644	49	39
OK-1	2.6	0.84	0.069	0.127	0.135	60	39,962	3.29	889	610	21	98	614,997	91	125,483	92	20
OK-2	1.4	na	0.069	0.104	0.094	42	45,474	3.95	1,016	762	23	77	820,828	92	310,012	119	38
OK-3	1.4	1.17	0.074	0.132	0.117	48	35,828	3.42	1,041	889	26	67	791,190	56	347,312	83	44
OK-4	1.9	na	0.076	0.236	0.196	78	44,096	3.27	838	635	23	115	569,035	66	108,049	58	19
OK-5	1.9	0.79	0.076	0.127	0.109	44	22,737	3.32	864	686	24	98	546,158	86	190,199	88	35
OR-1	1.2	na	0.069	0.104	na	na	24,804	3.64	965	na	na	80	692,111	na	na	na	na
OR-2	1.7	na	0.048	0.124	0.089	56	29,627	3.33	965	787	25	113	981,667	83	319,052	73	33
OR-3	1.4	na	0.119	0.183	na	na	32,383	3.09	889	na	na	62	389,667	na	na	na	na
PA-1	1.5	1.19	0.056	0.112	na	na	28,938	3.33	635	na	na	219	356,341	na	na	na	na
PA-2	1.3	1.19	0.135	0.163	0.147	33	33,761	3.76	1,067	660	19	44	565,610	78	148,071	178	26
WI-1	0.9	1.77	0.071	0.155	0.112	48	38,584	4.57	660	660	31	164	311,103	106	200,695	65	65
WI-2	0.9	1.53	0.071	0.178	0.137	59	31,005	3.36	864	762	27	99	550,684	65	220,392	66	40
WI-3	1.1	1.28	0.069	0.188	0.112	50	26,871	3.08	762	711	28	126	423,409	89	228,308	71	54
WI-4	1.4	1.99	0.081	0.406	0.130	49	35,139	4.36	965	686	22	72	623,841	88	193,799	122	31
WI-5	1.0	1.47	0.137	0.216	0.163	36	36,517	3.56	889	762	26	49	303,263	54	181,069	111	60
Average	1.3	1.56	0.089	0.191	0.155	53	33,641	3.57	924	789	26	82	546,141	62	216,065	84	41
Std Dev	0.4	0.51	0.027	0.078	0.055	12	5,737	0.40	117	134	4	44	180,531	27	75,149	32	14
Maximum	2.6	2.48	0.137	0.406	0.277	78	45,474	4.57	1,067	1,016	31	219	981,667	106	347,312	178	67
Minimum	0.6	0.79	0.048	0.104	0.089	33	22,737	3.08	635	610	19	37	303,263	18	73,209	47	17

Notes:

1. l = radius of relative stiffness (RRS); k = modulus of subgrade reaction; D = concrete slab rigidity
2. Values of concrete modulus of elasticity and average splitting tensile strength were measured using cores obtained during field testing.
3. Deflection data are for 40 kN FWD load

0.84 m/km to a high of 2.48 m/km. This represents good to very good ride quality considering that the test section ages ranged from 0.3 to 22 years at the time of testing. Thus, CRC pavements tend to provide a good riding surface even when a high amount of medium to high severity cracking is present. Also, there was a slight increase in IRI (rougher ride) with age.

Deflections Under Load

Average Sensor 1 deflections (maximum deflection under the load plate) ranged from a low of 0.048 mm to a high of 0.137 mm under the 40-kN FWD load for the basin (interior) testing. The deflection values are, of course, affected by slab thickness and base and subgrade support. The deflections and the subsequent backcalculated pavement stiffness characteristics therefore represent the conditions at the time of testing only.

The deflections measured at the transverse crack along the midslab location were generally comparable to the basin deflections, generally measured between crack locations. However, edge deflections measured at transverse crack locations tended to be almost twice as much as the basin (or midslab crack location) deflections for the morning testing (upward slab curl along the edges). The edge deflections tended to be less for the afternoon testing but still considerably more than basin test deflections. The afternoon edge deflections were reduced by about 10 to 30 percent from the morning edge deflections. Figure 1 shows a comparison of edge and midslab deflections at crack locations with basin test deflections. The tied-concrete shoulders appear not to have contributed much to reducing edge deflections.

Also, the relative change in edge deflection between morning and afternoon testing appears not to have been affected much by slab support condition—firm support such as LCB, ATB, or CTB versus softer support provided by granular or permeable bases.

Loss of Support Analysis

Loss of support analysis was performed using the data from deflection testing along the edge locations. At each test location, FWD loads of about 40, 53, and 67 kN were used. The maximum deflections at each of the three load levels were used to extrapolate loss of support conditions along the edge. Most of the sections exhibited some loss of support during the morning and the afternoon testing. The loss of support for the afternoon testing tended to be slightly lower. There appeared to be no significant influence of shoulder type or base type on the magnitudes of the loss of support. However, it should be noted that the data are confounded by actual temperature conditions and pavement thicknesses at each site.

Overall Pavement Stiffness

For concrete pavements, the overall pavement stiffness can be described very effectively using the radius of relative stiffness (RRS), *l*, value. The *l*-value is an important structural parameter of concrete pavements and has a direct influence on pavement behavior (structural response). The RRS was estimated for each section using the theoretical formula and using the actual slab thickness (average core thickness), laboratory measured modulus of elasticity value, and best estimate of the modulus of subgrade reaction. The RRS values were also backcalculated from the deflection testing using Program ILLI-BACK. These RRS values are presented in Table 2. The following is a summary of the comparison of the RRS values.

- For basin testing, the backcalculated RRS values were independent of load levels, which ranged from 40 to 70 kN. Thus, a single load level of 40 kN is considered adequate for CRC pavement basin testing. However, multiple load levels should be used for testing along the pavement edge if loss of support determination is desired.

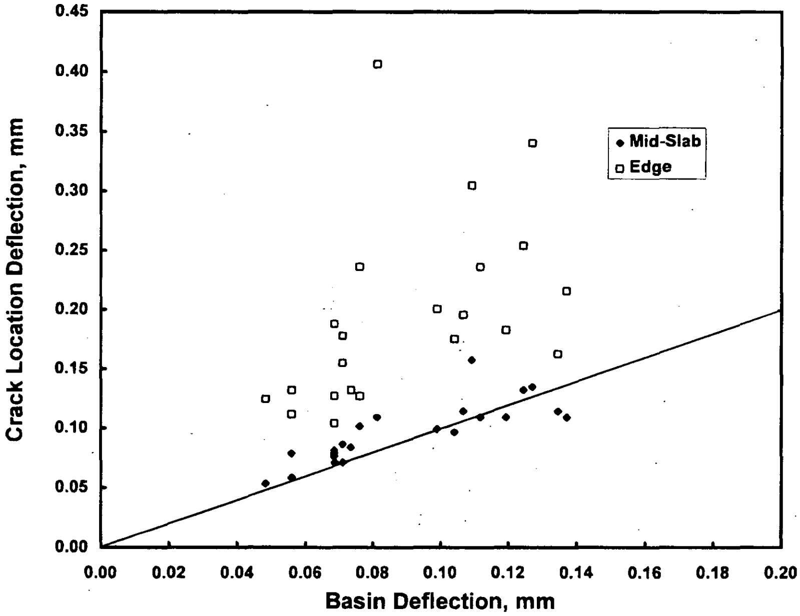

FIGURE 1 **Comparison of basin and crack location deflections.**

TABLE 3 Average Values for Various Parameters for 23 Test Sections

	Average Values		
	ℓ, mm	k, kPa/mm	D, kN-m
Basin Testing	914	81	546,000
Mid-Slab Crack Testing			
Morning Testing	762	103	341,000
Afternoon Testing	838	92	391,000
Edge Crack Testing			
Morning Testing	762	54	171,000
Afternoon Testing	787	62	216,000

- RRS values for testing at crack locations were generally lower than those along basin (noncrack) locations.
- RRS values for midslab crack location increased some from morning to afternoon testing. However, there was very little increase in the edge crack location RRS values from morning to afternoon testing even though actual deflection values were lower.
- The crack location RRS values for edge testing were only slightly lower than for the midslab testing.
- Based on the review of the RRS and *D*-values, it appears that concrete shoulders may not be very effective in all cases for structural strengthening of the mainline CRC pavements. The use of concrete shoulder may still be strongly desired for other reasons, such as maintenance-free shoulder, effective joint sealing, and so forth.

The deflection test data were further analyzed to backcalculate the modulus of subgrade reaction *k* and the slab rigidity *D*. Although the RRS term describes the overall stiffness of the total pavement system, the *D*-term describes the rigidity of the concrete slab only. Table 3 illustrates the average values for the various parameters for all 23 test sections.

Thus, the effective *k*-values along the edges are about 60 to 70 percent of the values for the midslab locations, and the *D*-values along the edge are about 30 to 60 percent of the values for the midslab locations. The basin testing (uncracked locations) resulted in the highest *D*-values. These trends in *k*- and *D*-values are what one would expect and appear to be more descriptive of the actual physical condition (edge) of the pavement system. It is also likely that the backcalculated *D*-value would be much lower at those edge locations that exhibit the beginning of a punchout or exhibit wide cracks. Thus, it is recommended that the *D*- and *k*-values be used in interpreting the results of edge testing, in addition to using RRS.

Crack Spacing Analysis

The average crack spacings for each site are shown in Figure 2. During the study, it was realized that a good method for characterizing the cracking pattern for CRC pavements did not exist. In the past, use has been made of the cumulative frequency distributions for representing the total number of cracks that have spacings equal to or less than the designated crack spacing. This is certainly a good method, providing a clear visual description of the cracking pattern. The cumulative frequency plots for each of the test sections are given in Figure 3. These plots can be used to identify the number of

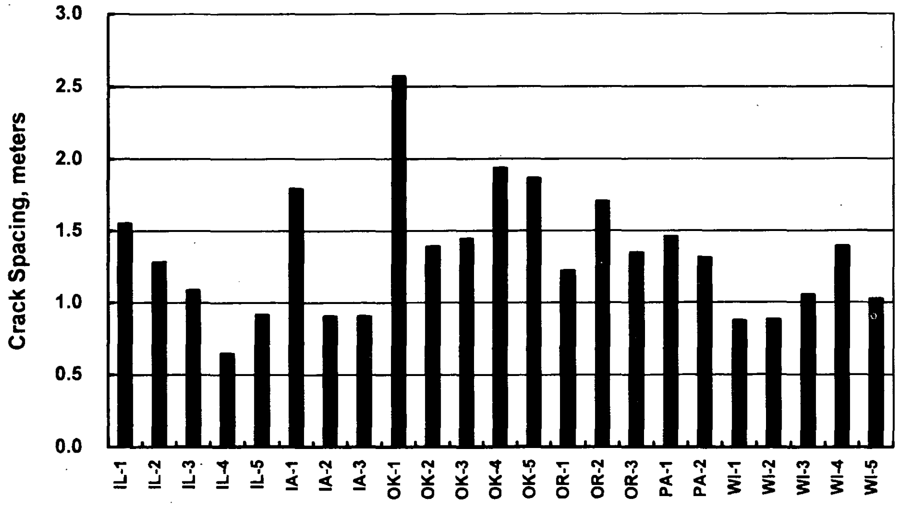

FIGURE 2 Crack spacing summary.

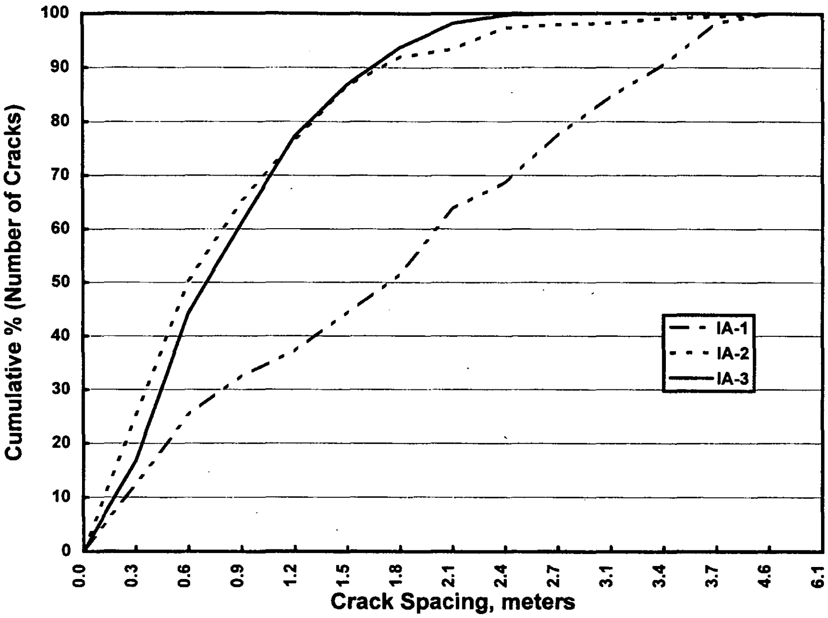

FIGURE 3 **Crack spacing distribution (by number of cracks).**

cracks (by percentage) that are greater or less than the designated crack spacing.

The cumulative frequency plots of the type presented in Figure 3 do not, however, represent the true picture of the cracking pattern as the focus of these plots is the number of cracks. A more representative characterization is the cumulative frequency based on the length of paving exhibiting a designated crack spacing. Thus, as an example, if 40 percent of the cracks (by number) have crack spacing equal to or less than 0.9 m, the length of paving exhibiting crack spacing equal to or less than 0.9 m may be only 20 percent or less. Similarly, if 10 percent of the cracks (by number) have crack spacing greater than 3.1 m, the length of paving exhibiting crack spacing greater than 3.1 m may exceed 20 percent. It is the length of paving that exhibits a certain cracking pattern that is as equally important as the number of cracks that exhibit a certain cracking pattern. The cumulative frequency plots based on length of paving are given in Figure 4.

Table 4 shows a comparison of the crack spacing characterization using the frequency distributions based on the number of cracks and the length of paving involved. The length of paving definition appears to be more descriptive. For cluster cracking, it indicates the potential for cluster cracking–related problems, in the presence of poor support conditions, based on the amount (by length) of cracking that is less than 0.9 m. It also clearly indicates the length of

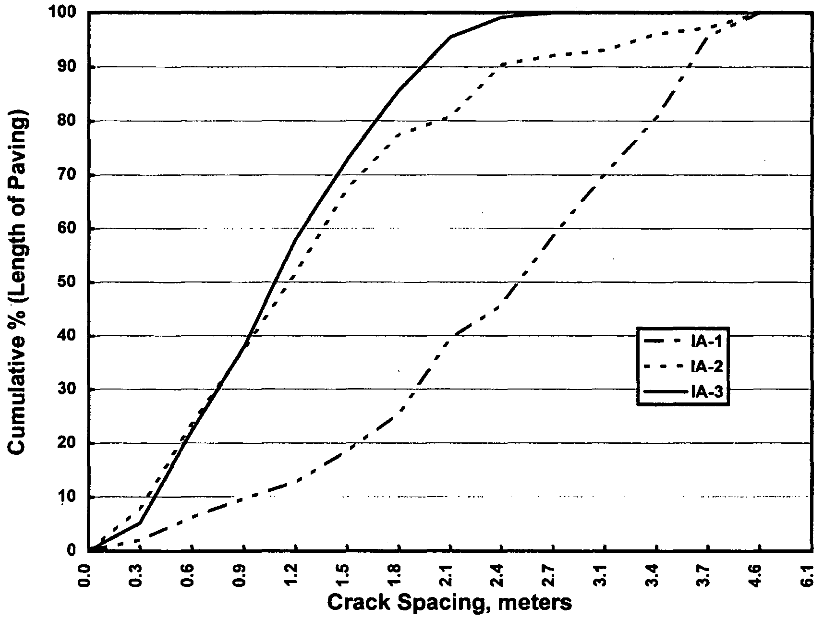

FIGURE 4 **Crack spacing distribution (by length of paving).**

TABLE 4 Crack Spacing Distributions

Test Section ID	Age as of Fall 1991 Testing, years	Long. Steel Amount, %	Total No. of Cracks (305 m)	% of Cracks = or < 0.92 Spacing (by no.)	% Length with Cracks or < 0.92 Spacing	% of Cracks > 1.84 m Spacing (by no.)	% Length with Cracks > 1.84 m Spacing	% of Cracks > 3.05 m Spacing (by no.)	% Length with Cracks > 3.05 m Spacing
IL-1	0	0.70	195	37	16	33	60	9	25
IL-2	15	0.59	237	45	22	23	43	3	7
IL-3	20	0.60	279	51	30	10	21	1	1
IL-4	20	0.60	470	86	79	1	2	0	0
IL-5	5	0.70	329	67	44	10	25	0	0
IA-1	20	0.65	169	33	10	49	73	15	27
IA-2	22	0.65	336	65	40	8	23	2	8
IA-3	15	0.65	334	61	41	6	10	0	0
OK-1	4	0.50	118	28	5	58	88	43	73
OK-2	5	0.50	217	40	18	25	50	8	21
OK-3	3	0.50	210	36	16	29	52	7	17
OK-4	7	0.50	156	15	5	51	72	13	25
OK-5	2	0.61	164	24	9	45	67	13	26
OR-1	7	0.60	248	36	18	12	23	1	1
OR-2	4	0.60	179	25	10	34	53	8	20
OR-3	20	0.54	227	41	21	23	43	3	9
PA-1	15	0.45	208	33	15	28	47	3	7
PA-2	22	0.55	231	39	20	20	38	2	6
WI-1	18	0.65	347	69	38	9	25	1	2
WI-2	6	0.67	345	68	52	2	6	0	0
WI-3	7	0.67	288	48	32	5	10	0	0
WI-4	7	0.67	218	34	17	25	41	0	0
WI-5	16	0.61	295	54	34	7	14	0	0
Average	11	0.60	252	45	26	22	39	6	12
Std Dev	7	0.07	82	18	18	17	24	9	17
Maximum	22	0.70	470	86	79	58	88	43	73
Minimum	0	0.45	118	15	5	1	2	0	0

paving that incorporates undesirable longer crack spacing, in excess of 3.1 m. A concern with longer spaced cracks is the development of crack spalls, steel rupture, and punchout at companion closely spaced cracks. These problems are also better characterized by the number of crack locations where these problems may develop in the future. Thus, for problem cluster cracking, the length of paving involved is more significant than the number of cracks. For longer spaced cracks, it is the number of cracks that is more significant than the length of paving involved.

To relate the crack spacing to the structural response of the pavements, the concept of the average of several of the closest crack spacings was developed. Individual crack spacing is very difficult to relate to the structural response that is provided by the effective length (or area) of the CRC pavement. The effective length is generally considered to be about one and a half to two times the RRS value on each side of the applied load—about 1.2 to 1.8 m on each side of the load. Thus, it was necessary to develop a different approach to represent the crack spacing pattern that would better incorporate the effective length of the pavement. The concept that was developed was to use the average spacing of the closest five cracks (ASCFC).

The plot of the ASCFC with distance is also useful in identifying locations of cluster cracking (groups of cracks with average spacing of less than about 0.6 m). Similarly, cracking patterns with large crack spacings can also be easily identified. The ASCFC trends provide a more visual definition of crack spacing pattern than use of the standard deviation or the coefficient of variation parameters. The ASCFC plot (with distance) can also be used to identify the extent of a pavement section that exhibits "acceptable" cracking pattern. For example, if acceptable values of ASCFC are assumed to be between 0.9 and 1.8 m, then as shown in Figure 5 the length of the pavement section outside the acceptable limits can be easily identified. It is possible that this length can be used as a performance indicator and compared with the extent of other manifested distresses, such as punchout and patching, ride quality, and so forth.

The RRS values were compared with crack spacing for each test section. The plot of basin test RRS values along side ASCFC indicates that pavement stiffness is not dependent on crack spacing as long as there is high load transfer efficiency at the transverse cracks. The load transfer effectiveness was generally greater than 90 percent for most of the test sections. However, there appears to be some interaction between cluster cracking (average crack spacing of less than 0.6 m) and RRS.

Overall, the variability in the RRS values along the test section appears to be more influenced by the apparent variability in the support condition. The extent of variability does not appear to be influenced by the base type (stabilized versus granular) or by the subgrade type (fine grained versus coarse grained).

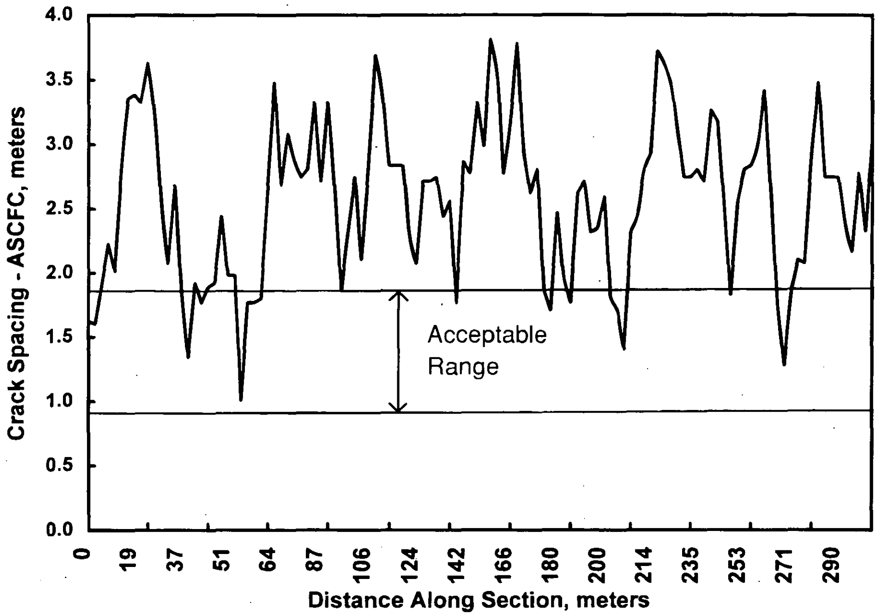

FIGURE 5 Illustration of procedure to identify extent of marginal cracking pattern.

Effect of Design Features on Crack Spacing Development

Thickness Effects No clear trends were apparent. The data were also confounded by age, percentage of steel, and climatic region.

Tied-Concrete Shoulder Effects No definitive trends were apparent between AC and PCC shoulder. The data were also confounded by age, percentage of steel, and climatic region.

Permeable Base Effects The two sections with permeable base exhibited slightly higher average crack spacings. However, both of these sections were young—IL-1 was only a few months old and OK-5 was only 2 years old. To further study the effect of permeable CTB, data from an additional CRC section constructed on a permeable CTB were obtained. This section was located along I-295 in Virginia (just south of the Exit 9B sign, near Milepost 8) and was constructed during the summer and fall of 1991. The section details are as follows:

- Slab thickness = 228 mm,
- Permeable CTB thickness = 101 mm,
- CTB thickness = 152 mm,
- Percentage of steel = 0.65,
- No tubes or chairs used—concrete placed in two lifts with the steel placed at the surface of the bottom lift,
- Permeable CTB cement content = 130 kg/m^3,
- Permeable CTB aggregate = ASTM No. 57, and
- Shoulder type = jointed plain concrete.

A 305-m length of the section was surveyed on May 17, 1994. The section exhibited the following cracking pattern:

- Total number of cracks per 305 m = 322,
- Average crack spacing = 0.95 m, and
- Standard deviation of crack spacing = 0.54 m.

The foregoing data indicate that 3 years after construction, the crack spacing along the permeable base section exhibits acceptable cracking pattern. Most of the cracks either exhibited no distress or were of low severity. Thus, the concern that CRC pavements constructed over permeable CTB may not exhibit acceptable cracking pattern (because of interlocking or bonding with the permeable base) may not be justifiable. However, it should be stressed that an adequate amount of steel (≥0.65 percent) should be used to minimize any potential problems related to use of the permeable CTB.

Epoxy-Coated Bar Effects The three sections with epoxy-coated bars exhibited slightly lower average crack spacing. Although the sample size is small, it appears that the use of epoxy-coated reinforcement may not result in undesirable cracking pattern. [The current FHWA Technical Advisory T 5080.14 (dated June 5, 1990) recommends that the bond area be increased 15 percent to increase the bond strength between the concrete and reinforcement if epoxy-coated steel reinforcement is used.] This implies that 15 percent more steel bars should be used if epoxy-coated bars are used. The sections with epoxy-coated bars had the following steel amounts: OK-3 = 0.5 percent; WI-2 and WI-3 = 0.67 percent.

Thus, on the basis of limited data, it appears that use of 15 percent more steel bars may not be warranted provided the steel content is properly estimated.

Effect of Age on Crack Spacing There appears to be a trend toward a decrease in crack spacing with age with crack spacing stabilizing after about 8 to 10 years. Crack spacing appears to have an effect on ride quality and estimated present serviceability index (PSI). Shorter crack spacing results in higher IRI (and lower PSI) values, indicating that cluster cracking may result in poorer riding surface.

Load Transfer Efficiencies at Cracks and Crack Width Analysis

Load transfer efficiencies were determined using the data from the morning and afternoon testing at crack locations. All sections, except the Oklahoma sections and WI-1 exhibited high load transfer efficiencies (greater than 90 percent) at crack locations. The Oklahoma sections have the widest crack spacings due to a smaller steel amount. This may be contributing to the development of the poor load transfer at crack locations.

Crack widths at the test section ranged from 0.2 to about 0.84 mm (ignoring the apparent high values noted at the two Pennsylvania sites) during the mornings. The morning slab middepth temperatures during crack width measurements ranged from 5°C to about 18°C. The cracks did close a little during the afternoon when midslab temperatures increased from about −15°C to −9°C. For each section, the crack widths were normalized to middepth slab temperatures of 5°C and −18°C to allow comparisons between sites. The normalization was performed by using the laboratory measured coefficient of thermal expansion for the 30.5-m subsection used for crack width measurements.

The normalized crack width (at 5°C) ranged from 0.24 mm at IL-1 to 1.01 mm at OK-1 and WI-1. The average normalized crack width at 5°C was 0.59 mm. Limiting crack width criteria for CRC pavements are presented in the AASHTO Guide based on studies performed in Texas. A maximum crack width of 1.07 mm is recommended to avoid spalling. For the wet-freeze region test sections, using the crack width data normalized to −18°C, the IA-1, WI-1, and WI-4 were marginal for the AASHTO crack width criteria. It should be noted that WI-1 also had lower load transfer efficiency at the crack locations.

Effect of Steel Amount

The longitudinal reinforcement has a significant influence on the performance of CRC pavements. Higher amounts of reinforcement result in smaller crack spacing for a given set of conditions (concrete quality, climatic conditions). Figure 6 shows the effect of steel amount on crack spacing. Considering that the data points incorporate a broad range of pavement age, concrete quality, and climatic conditions, there is a strong overriding linkage between the percentage of steel and crack spacing. With a steel amount of about 0.8 percent, average crack spacing may approach about 0.6 m, which borders on undesirable crack spacing in the presence of poor support conditions and results in a high incidence of cluster cracking and a high potential for future punchouts when the support condition is marginal. A steel amount in the range of 0.6 to 0.7 percent appears to provide average long-term crack spacing ranging from 0.9 to 1.5 m. It should also be pointed out that some of the European experience indicates that close crack spacing (e.g., average crack spacing of 0.6 m) from using a steel amount of 0.85 percent can still provide excellent performance under heavy truck traffic provided that a good support condition is constructed (*4*).

It should be noted that during the design process, the amount of steel determined to obtain acceptable crack spacing, crack width, and steel stresses is based on the assumption that the design concrete strength will be obtained. However, for a given (design) steel content, if a higher concrete strength is actually obtained, crack spacings may be larger than anticipated. Similarly, if lower concrete strength is actually obtained during construction, a much closer crack spacing may result. This is very important to establish, especially when using a marginal amount of steel—less than 0.6 percent. The larger crack spacing may result in higher steel stress and wider cracks resulting in premature failures. Therefore, if the possibility exists that higher-than-specified concrete strengths may be obtained on any given project, the prudent course would be to specify a slightly higher steel content to accommodate the expected higher concrete strength.

Based on the data obtained as part of this study, use of steel in an amount less than 0.6 percent is not recommended because the cracking pattern that develops is marginal. The larger crack spacings that develop as a result of a low steel amount create potential locations for steel ruptures and punchouts at closely spaced cracking adjacent to widely spaced (greater than 3.7 m) cracks.

FIGURE 6 Average crack spacing as function of percentage of steel reinforcement.

Summary of Distresses

Pavements 15 years old and older generally exhibited moderate severity of spalling at transverse cracks. The older pavements also exhibited various amounts of patching. Sections WI-1 and WI-5 had the most patches (partial and full depth) within the 305-m sections tested. Only two sections (OK-4 and PA-2) exhibited punchouts that had not been patched. There appeared to be no correlation between patching amount and ride quality, indicating that the patches, if constructed properly, are not detrimental to ride quality. Steel corrosion based on core examination was found at most of the sections in the wet-freeze regions with the exception of sections where epoxy-coated reinforcement was used.

SUMMARY AND RECOMMENDATIONS

The study has highlighted many new methods for evaluating CRC pavement performance incorporating the results of distress surveys (cracking pattern), crack width measurements, deflection testing along the edge and crack locations, and ride quality evaluation. An attempt was made to determine why certain CRC pavement sections behaved (in terms of cracking pattern) significantly differently from other sections with many factors being similar for these sections. The different cracking patterns at IA-1, OK-1, and OR-2 (compared with other in-state sections) could not be explained directly. It is quite possible that the ambient temperature conditions and curing conditions at these sections may have contributed to the development of the cracking pattern.

There is a strong interaction among percentage of steel, concrete strength, and crack spacing. For conventionally used concrete strengths (splitting tensile strength at 28 days of about 3,000 kPa), steel in the amount of 0.6 to 0.7 percent appears to provide desirable long-term average crack spacing in the range of 0.9 to 1.5 m. The reader is cautioned about the use of steel content less than 0.6 percent; it is clear from the Oklahoma test sections and a recent Maryland project along U.S. 50 (not reported here) that use of 0.5 percent steel will result in longer crack spacings and possible premature development of punchouts at closely spaced cracks adjacent to longer spaced cracks. The use of 0.65 percent as the minimum steel content is strongly recommended with the conventional concrete strengths typically used in the United States. If a higher steel content is to be used, appropriately higher strength concrete should be specified to maintain desirable average crack spacing in the range of 0.9 to 1.5 m or a stabilized base must be specified.

The effect of tied-concrete shoulder could not be classified as positive with respect to improving the structural response of the CRC pavements (as indicated by deflection testing along the edges). Use of tied-concrete shoulders may have other advantages and as such may be used in conjunction with CRC pavements. The use of widened lanes for CRC pavement appears to be promising and should be seriously considered as a design option. Based on the good performance of the three Oregon sections (each incorporating a 4-m-wide outside lane), using widened lanes should not cause concern because of the longer aspect ratio for each cracked portion of the pavement.

The effect of base type on CRC performance was not pronounced. The concern about using a hard support (e.g., LCB) could not be clearly addressed. The two sections with permeable bases exhibited higher crack spacing. However, both sections were young and one of the sections was constructed with a smaller amount of steel. Also, a separate evaluation of a 3-year-old Virginia CRC pavement constructed with a permeable CTB indicated that adequate crack spacing can develop in CRC pavements incorporating permeable CTB.

The use of epoxy-coated reinforcement resulted in no undesirable cracking pattern. The FHWA Technical Advisory T5080.14 (dated June 5, 1990) recommends that the bond area be increased 15 percent to increase the bond strength between the concrete and reinforcement if epoxy-coated steel reinforcement is used. This implies that 15 percent more steel should be used if epoxy-coated bars are used. Based on the limited field data, it appears that the use of 15 percent more steel may not be warranted provided the steel content is properly estimated. However, additional field data need to be compiled to verify this observation.

Based on the deflection testing, the following summary is presented.

- Load transfer efficiencies at transverse cracks of CRC pavements, even after many years of service, remain high—generally greater than 90 percent, provided that an adequate steel amount is used.
- The radius of relative stiffness, l, did not characterize well the effective pavement stiffness along the edge. The better parameter to describe the edge structural stiffness is the slab rigidity, D. The D-values were found to be considerably lower along the edge than at the interior. The backcalculated D-value may be a better indicator of potential punchout locations.
- Deflection data did not correlate well with the cracking pattern, indicating that the pavement stiffness is not dependent on crack spacing as long as there is high load transfer efficiency at the transverse cracks. Overall, the variability in the backcalculated pavement stiffness values appears to be influenced more by the apparent variability in the support condition.

Further research and development for CRC pavement should focus on improving the cracking pattern of the pavement through improved construction and design technology. For many years, the design of CRC pavement has focused on the percentage of steel reinforcement and the expected drop in pavement temperature over the course of a year. However, it is clear that the crack pattern of CRC pavement cannot be controlled by the steel design alone and that other considerations should be included in the design process, such as the type of aggregate, method of curing, concrete shrinkage potential, depth of steel cover, and rate of strength gain in the first 3 days, among others. Also, the design process for a CRC pavement should continue through construction and not end as soon as the plans and specifications are prepared. A more active interaction between the design process and actual ambient conditions during construction needs to be developed to achieve CRC pavements with acceptable cracking patterns. This may require imposing of guidelines on acceptable ambient conditions for placement of CRC pavements.

ACKNOWLEDGMENTS

The authors acknowledge the fine support provided by the many state highway agencies during the various phases of the data collection process, specifically, the many highway agency personnel from Illinois, Iowa, Oklahoma, Oregon, Pennsylvania, and Wisconsin that supplied site-specific pavement-related data and provided access and traffic control during the testing. Special thanks also go to the Oregon Department of Transportation for providing more extensive field testing support (FWD testing and coring). The support of the State Technical Advisory Committee and various FHWA staff (particularly Jim Sherwood and Roger Larson) is gratefully acknowledged.

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Publication of this paper sponsored by Committee on Rigid Pavement Design.