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Impact, Cause, and Remedies for Excessive Cracking in CRC Pavement

Study SD2004-07
Final Report

Prepared by
Office of Research
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16. Abstract <p>Newer continuously reinforced concrete pavements (CRCP) in South Dakota have exhibited undesirable levels and types of transverse cracking. This poor performance was not expected under the current recommended design practices. Research was undertaken to identify design, construction and material issues that may be contributing to the undesirable cracking. After preliminary surveys of existing projects and analysis of the available Long Term Pavement Performance CRCP data, a systematic construction program was initiated whereby changes in design and materials were incorporated and monitored for any beneficial effects. Beneficial changes were incorporated into projects scheduled for construction the following year and new parameters modified to understand how critical the effects on the cracking behavior any given parameter was with minimum ambiguity. The results are a series of recommended changes in design, construction and materials yielding much more normal and desirable cracking patterns. They include using an optimized 1½” maximum coarse aggregate concrete mix, reducing steel content to 0.6% regardless of thickness, requiring a nominal steel depth of 3¾” for all new CRCP pavements, using a modified chair configuration providing more support near centerline to reduce steel settlement, increasing the minimum cement content to 512 lbs/yd³ with 112 lbs/ yd³ modified Class F fly ash, wetting down aggregate stockpiles the night before paving if temperatures above 80°F are expected and increasing the curing compound application rate to 1.5 gal/125 ft².</p>			
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INTRODUCTION

In the 1990's, the South Dakota Department of Transportation began a concerted effort to rebuild its aging Interstate highway network. Through analysis of performance and life-cycle cost, continuously reinforced concrete pavement (CRCP) was identified as an alternative of choice for interstate reconstruction in South Dakota. From 1995 to the present, the SDDOT has constructed over 250 2-lane miles of CRCP interstate highway at an approximate investment approaching \$300 million.

CRCP pavement is designed with substantial reinforcing steel to eliminate the need for transverse joints, which are the primary point of failure on jointed concrete pavement. Instead of joints, CRCP exhibits hairline transverse cracks, which normally are spaced at intervals of two to four feet and which are tight enough to resist faulting, spalling, and intrusion of foreign material. However, some recently constructed CRCP pavements exhibit cracking that is irregular, with spacing varying from as little as one foot to as much as 10 feet. Some of the cracks appear to be significantly wider than normal, which could allow intrusion of water, chemicals, and other foreign material. Some cracks have begun to deteriorate and spall. Finally, isolated punch-outs—usually located within a single lane and less than 20 feet in length—have appeared on some projects.

Several factors could potentially contribute to these problems, including:

- concrete mix design and composition
- aggregate type and gradation
- aggregate thermal expansion coefficients
- ambient temperature during pour and cure
- material temperatures at pour
- reinforcing steel percentage and size
- reinforcing steel location and lapping
- slab thickness
- underlying cushion or pavement layers
- subgrade or drainage problems
- paving equipment or operation
- concrete vibration
- attack from deicing chemicals
- grade and paving direction

The effect of the cracking and distress on long-term pavement life and serviceability is unknown. It is not yet clear whether the problems represent a major structural problem or a relatively minor, cosmetic problem. Whether the cracking and distress will significantly accelerate long-term failure mechanisms, such as steel fatigue, corrosion, or freeze-thaw damage is presently unknown.

RESEARCH OBJECTIVES AND APPROACH

The objectives for this research project are to:

1. Determine the character, extent, and severity of cracking and other distress in CRCP constructed in South Dakota since 1995.
2. Identify factors and interactions among factors that contribute to observed distress.
3. Assess the impact of observed distress on long-term pavement life and performance.
4. Recommend changes to design, specifications, and construction practice to substantially reduce the incidence and severity of distress in CRCP pavements.
5. Recommend cost-effective maintenance and rehabilitation strategies for CRCP pavements exhibiting unexpected levels of distress.

To address these objectives and adequately formulate a work plan for such a complex system with so many potential contributing variables, an effort was undertaken to collect sufficient preliminary data on new CRCP pavement sections to obtain a better idea of the extent of the problem and the variations in performance in the CRCP pavements statewide. Initial surveys of projects for crack spacing and distress levels indicated that performance was not as desired on all sections. As expected, cracking was more closely spaced for quartzite (Average 2.4' on projects surveyed too date) than limestone (3.8') which is not surprising considering the almost threefold greater coefficient of thermal expansion for the quartzite. What was disconcerting was the incidence of Y-cracking, network cracking and cluster cracking. The fundamental concept behind CRCP is to generate the right cracking pattern, i.e. cracks 4-10' apart with no associated subparallel cracking, no spalling and minimal crack widths. Many of the cracks observed in the new CRCP are not tight, isolated transverse cracks but have associated fine cracking along their length (network cracks, Figure 1), separate into two cracks toward centerline or shoulder (Y-cracking, Figure 1) or have one or more additional transverse cracks within a foot or less (cluster cracks, Figure 2). All of these cracking patterns are undesirable and can lead to punchouts and other performance issues under traffic. In addition, spalling was apparent to some extent on all projects surveyed and ranged from a few very minor spalls (There were two minor spalls on a 600' section of 3 week old CRCP on I-90 west of Rapid City) to severe spalling at almost every crack (I-90 east of Sioux Falls). Many of these spalls were in the process of forming and exhibited a small secondary crack along the main crack which formed an oval lozenge of what appears to be crushed concrete fragments. In addition, a notable punchout failure on I-29 occurred at a lap and had steel depth 1" greater than nominal.

Numerous design changes have occurred over the course of the construction of the new CRCP pavements. This fact, coupled with the reduction in maximum aggregate size from 1½" to 1" and the routine use of modified Class F fly ash in all the new projects adds considerable difficulty in determining the contribution of a given variable to performance. To address this issue using the best approach possible, the original work plan called for selecting construction projects with different design parameters divided into three 500' subsections based on paving temperatures and different pavement zero energy values. This refers to the temperature that produces no expansion or contraction stresses in the pavement. These three sections were surveyed based on random selection and an average used to characterize the pavement cracking as no temperature data were available for most projects.



Figure 1: Network and Y-Cracks on CRCP

The surveys initially included:

- crack distribution
- crack width
- steel depth, slab depth and lapping locations
- distress (spalls, Y-cracking, etc.) with a weighted distress rating.



Figure 2: Typical Cluster Cracking on CRCP

Table 1: Factors Affecting CRCP Performance

Property	Old Projects	New Projects	Research Approach
Base/Subgrade	<ol style="list-style-type: none"> 1. Granular 6-8" (2³/₄") 2. Lime-Treated 3" 3. Asphalt-treated (AE³/₄") 2" 4. Lime-treated subgrade 5. Drainage-no provisions 	<ol style="list-style-type: none"> 1. Granular 5-6" (3¹/₄"/1") 2. Existing AC 6" 3. Existing PCC 9" 4. Rubblized 5. Drainage-variable 	<ol style="list-style-type: none"> 1. FWD—mid-slab & edge 2. Pumping 3. Anomalous cracking

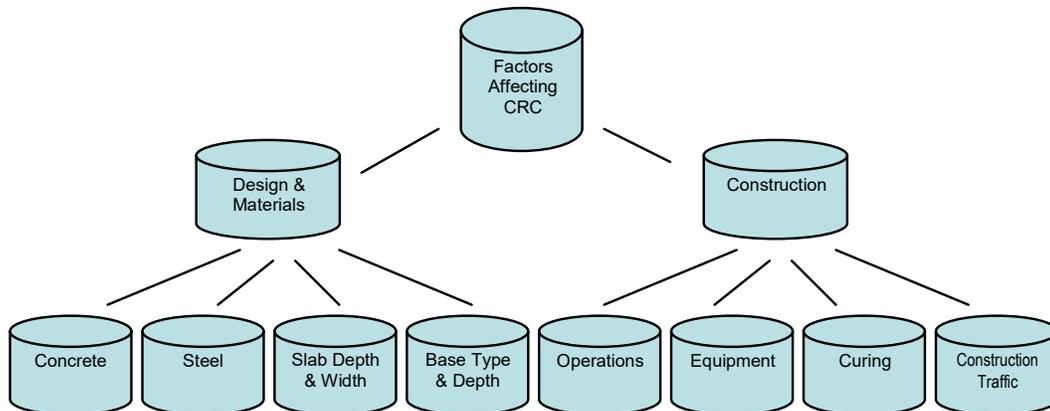


Figure 3: Factors Affecting CRCP Performance

Steel	<ol style="list-style-type: none"> 1. 3¼" depth 2. #5 bars Grade 60 3. 0.6% steel 4. 18° skewed lap 5. #5 transverse at 4' 6. Hand-tied 	<ol style="list-style-type: none"> 1. 3¼-4" depth 2. #6-#7 bars Grade 60 3. 0.6-0.8% steel 4. Staggered variable 5. #4 transverse at 3 & 4' 6. Machine-tied 	<ol style="list-style-type: none"> 1. GPR survey-steel depth 2. Cracking patterns 3. Crack widths 4. Distress levels 5. Strain gauge instrumentation
Pavement	<ol style="list-style-type: none"> 1. 8" thickness 2. 24' wide 	<ol style="list-style-type: none"> 1. 8-12" thickness 2. 24 & 26' wide 3. Variable width urban sections 	<ol style="list-style-type: none"> 1. Check adjacent sections of increasing thickness 2. Correlate performance vs. thickness & width 3. Strain gauge instrumentation
Concrete Strength	5000 psi	4500 psi	
Aggregate	<ol style="list-style-type: none"> 1. 1½" maximum size 2. Quartzite/limestone 	<ol style="list-style-type: none"> 1. 1" maximum size 2. Quartzite/limestone/granite 	<ol style="list-style-type: none"> 1. Construct new projects with optimized 1½" maximum size 2. Quartzite cracks closer spaced than limestone
Cement	523 lbs./ yd ³	480-512 lbs/yd ³	<ol style="list-style-type: none"> 1. Early strength and bond development 2. Shrinkage
Fly Ash	None	112-120 lbs/yd ³	
Grade Elevation	Bluetopping 25'	Bluetopping 50'	<ol style="list-style-type: none"> 1. Compare steel depth and pavement thickness variability 2. Performance at different temperatures
Paving	<ol style="list-style-type: none"> 1. Fixed form (1 slip form) 2. Ambient & concrete temperatures 3. Construction traffic 	<ol style="list-style-type: none"> 1. Slip form 2. Ambient & concrete temperatures 3. Construction traffic 	
Curing	Burlap	Curing compound	<ol style="list-style-type: none"> 1. Cure test sections using burlap in new CRC

RESEARCH TASKS

The research effort involved the following tasks.

TASK 1: PANEL MEETING

Meet with the project's technical panel to review project scope and work plan.

A meeting was held with the technical panel on Tuesday, November 23, 2004.

TASK 2: LITERATURE REVIEW

Review literature pertaining to CRCP design, construction, and performance, with particular emphasis on cracking and distress.

A thorough literature search was conducted as well as efforts to obtain various mathematical models of CRCP responses both to initial restraint shrinkage and to load. Initial results indicate that the use of a concrete containing fly ash and a smaller maximum size aggregate in the new CRCP may have a profound effect on crack spacing and initiation. South Dakota is the only state listed on the Concrete Reinforcing Steel Institute (CRSI) webpage that utilized a 1" maximum aggregate size exclusively. Ambient and concrete temperatures are also critical to both crack spacing and the occurrence of subsequent spalling, based on an analysis of CRCP performance in Texas.

Table 2: Comparison of Some Design Features among States Using CRCP

Design Feature	State					
	Illinois	Oklahoma	Oregon	South Dakota	Texas	Virginia
Slab Thickness (inches)	10	9-12	8-12	8-11	8-15	10-11
Outside Lane Width (ft)	12	12	14	12 or 14	12	12 or 14
Maximum Aggregate Size	1½"	1½"	1½"	1"	¾-1½"	100% passing 2"
PCC Strength (psi)	3,500 comp 650 flexural	3,000 comp	4,000 comp	4,000 comp	650 flexural	3,000 comp
Steel Depth (inches)	3.5	Mid-slab	4.0	3½-4	Mid-slab	Mid-slab

The first literature reviewed involved the construction of two experimental test sections of CRCP on Interstate 90 in South Dakota in May of 1963 (1). The sections, 3400 and 4400 feet long, were constructed 8" thick with 0.646% longitudinal steel (#5 bars of A60 steel on 6" centers) with a 9" granular subbase. A quartzite coarse aggregate with an extremely high coefficient of thermal expansion was used in both and the concrete strength requirement was for 5000 psi at 28 days. The primary experimental features were steel depth and end treatment. One section was built with a nominal depth to the center of steel of 2½" while the other was 3^{11/16}**Error! Bookmark not defined.**". The sections were monitored over a period of five years with both performing extremely well. Transverse cracking commenced immediately after construction and stabilized after the first winter. The shallow steel section exhibited an average crack spacing of 1.7' whereas the deeper longitudinal steel section had a spacing of 2.9'. After five years in service, the 2½" section had an average crack width of 0.004 inches while the 3^{11/16}**Error! Bookmark not defined.**" section averaged 0.005 inches. These pavements continued in service up to the present with no significant maintenance having been performed over the last 45 years and are scheduled to be replaced

as part of a reconstruction of the adjacent mesh-dowel concrete pavement, not due to any significant loss in serviceability.

The exemplary performance of these sections resulted in a decision by the South Dakota Department of Highways (now SDDOT) to construct a major portion of the remaining Interstate system under construction using CRCP. Over 700 lane miles of CRCP were constructed from 1968 to 1975 with an 8" nominal thickness and 0.6% longitudinal steel content.

The history of CRCP pavements in the United States goes back to the Columbia Pike experimental pavement constructed in 1921 near Washington, D.C (2). In 1938 another experimental pavement incorporating continuous reinforcement was built on US 40 west of Indianapolis. Based on this experience, the Illinois Division of Highways in cooperation with the Bureau of Public Roads, embarked on an experimental study to generate a better understanding of the relationships between slab dimensions and steel content with regard to performance (3). Eight test sections were built on U.S.40, four with 7" thickness and four at 8", with steel contents of 0.3, 0.5, 0.7 and 1.0% as subsections in each thickness section. The pavement was 22' wide and constructed directly on fine grained soil (A-4, A-7-4). Transverse bars (#3) were continuous and spaced at 12" for half of each section and 18" for the remainder. No centerline joint was cut in the pavement.

A twenty year review (4) of the performance of these test sections yielded the following conclusions:

- Transverse cracks begin to develop immediately at closely spaced intervals and continue to develop at a greatly reduced rate over the life of the pavement. The more steel in the pavement, the greater the number of transverse cracks.
- There were no strong performance differences between the 7" and 8" pavements.
- Crack width is a function of steel content with lesser width associated with higher steel content. Crack width also increases with age.
- Slight spalling begins to occur at transverse cracks soon after construction and increases with age. The degree of spalling is inversely related to the amount of longitudinal steel.
- Without a centerline joint, even with a pavement width of 22', longitudinal cracking occurred necessitating inclusion of a centerline joint.
- Construction joints require at least 0.7% steel to eliminate potential problems.
- There was an unexplained permanent increase in pavement length, possibly explained by the occurrence of non-deleterious ASR expansion.
- CRCP pavements can be designed and constructed to serve at least as effectively as conventional jointed PCC pavements.
- CRCP can be built to and retain a high standard of smoothness over 20 years.
- CRCP does not provide protection from pumping in fine grained soils.

A further review of CRCP field performance in Illinois involving the analysis of 476 pavement sections built between 1947 and 1994, including the original experimental sections above, provided some further insights into critical design elements for superior field performance (5). These include:

- CRCP sections of 7” thickness and steel content less than 0.6% have the most failures among the sections.
- CRCP sections with 10” thickness and steel contents of 0.7 to 0.8% have the lowest number of failures amongst all the sections.
- A higher longitudinal steel content has a greater effect at reducing failures in 7” thick sections than in thicker slabs.
- Field results clearly show that crack tightness is critical, whereas, crack spacing can be very short (<3’). Increased steel content keeps tight cracks and short crack spacing.
- There was no significant difference between sections where steel was placed on chairs and steel placed using tubes.
- Sections with bituminous-aggregate mixture base performed better than sections with cement-aggregate base or granular base. (Granular base sections performed better than cement-aggregate base sections). Sections with any of these base types performed better than sections without base.
- The closer the steel is to the surface of the slab, the tighter the transverse cracks remain and fewer punchouts develop over many years. At least 3” of cover is needed for construction and to keep chlorides out.

Texas began experimenting with CRCP IN 1951 on two contiguous projects in Fort Worth. The Concrete Reinforcing Steel Institute initiated a report summarizing Texas DOT’s experience with CRCP and presented the following major findings with respect to CRCP design and performance in Texas (6):

- A linear relationship exists between crack spacing and the longitudinal steel bond area. For smaller reinforcing bars, the bond area is larger and the resulting crack spacing is smaller. For larger reinforcing bars, the total bond area decreases and crack spacing increases, assuming the same steel percent. TxDOT limits bar size to $\frac{5}{8}$ ” diameter (# 5) to prevent the formation of wide crack spacing.
- Crack spacing is wider for cold-weather placement than it is for warm-weather placement, i.e. higher temperatures increase the crack frequency.
- Crack spacing decreases rapidly during the first 30 days after concrete placement while the concrete is gaining strength. Lower concrete strength translates into reduced crack spacing.
- There is an inverse relationship between crack spacing and percent longitudinal reinforcing steel. For pavement with 0.6% steel, the crack spacing was slightly less than that measured in pavement with 0.5% steel.
- Crack width is directly related to crack spacing. The closer the cracks are spaced, the smaller the crack widths.
- Aggregate type and properties such as modulus and coefficient of thermal expansion directly impact crack spacing, with siliceous aggregate generating a crack spacing of about half that of limestone. Typically, performance is affected by the greater number of cracks.

- High temperature placement and curing conditions result in erratic crack spacing, including y-cracking, narrow crack spacing and intersecting cracks.

Reports from other states using CRCP were also reviewed and yielded similar conclusions and design recommendations (7,8,9)

The Netherlands uses CRCP as a primary pavement type for high volume traffic roadways with a low noise porous asphalt friction wearing course. They have modified the design used for constructing new CRCP pavements based on a desire to optimize cracking behavior (tight, homogenous cracks with a spacing between 0.8 (31.5”) and 3.0 m (118”). Some of the modifications include (10):

- Minimum concrete compressive strength requirement should have 95% of all concrete tests meeting 35 MPa (5076 psi) to reduce cluster cracking due to low and heterogeneous tensile strengths.
- The steel content should be reduced from 0.85% of cross-sectional area to 0.59% with bar size reduced from 18 mm to 16 mm (#5) to provide greater bond surface and greater crack spacing.
- A 6 cm (2.4”) asphalt interlayer should be placed above a 25 cm (10”) cement-treated base to provide uniform friction, tight cracks and act as an erosion barrier for the 25 cm (10”) pavement above.

One of the primary differences noted between the concrete used in the older CRCP pavements constructed in South Dakota and the newer ones built since 1995 was the reduction in aggregate top size from 1½” nominal maximum size to 1”. Larger aggregate size results in an exponential increase in the energy required to cause fracture and, as such, can have a marked effect on cracking, especially early-aged cracking, in CRCP (11). Figure 4 illustrates the relationship for specific fracture energy versus aggregate size and shows a 20% increase in fracture energy in going from 1” to 1½”.

Another primary affect of aggregate size on reinforced concrete systems was demonstrated by Abrams in his seminal work on portland cement concrete (12). Figure 5 clearly demonstrates the effect that increasing coarse aggregate size has on bond strength development and compressive strength. Presumably, utilizing the largest top size coarse aggregate that still provides adequate workability, placement and consolidation would be of benefit in CRCP pavements

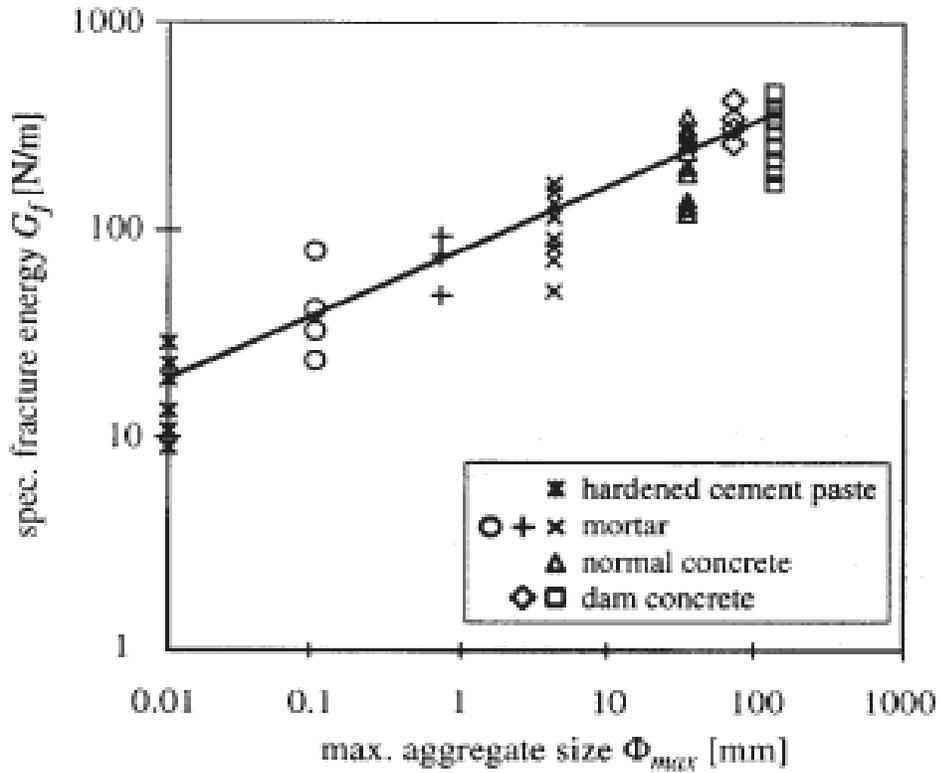


Figure 4: Specific Fracture Energy of Cement-Based Materials versus Maximum Aggregate Size

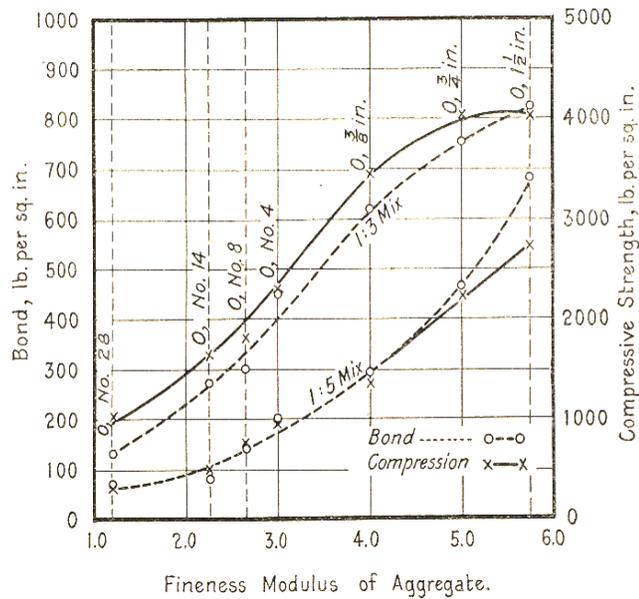


Figure 5: Effect of coarse aggregate size on bond and compressive strength

Analysis of CRCP Sections from the LTPP Database

In an effort to gain insight into the most critical of the numerous variables that may be contributing to undesirable performance in our newer CRCP pavements, data on 85 LTPP test sections from 29 states was analyzed using multiple regression techniques. An initial reading of the report, *Preliminary Evaluation of LTPP Continuously Reinforced Concrete (CRC) Pavement Test Sections* (13), issued in July, 1999 proved somewhat disheartening as the authors had conducted such an analysis without obtaining any statistically significant results and presented conclusions based on a direct comparison of the ten best performing sections versus the 13 worst performers in the database. They concluded that the worst performers had the following common characteristics:

- Larger crack spacing
- Greater depth to reinforcement
- High value of mean slab thickness
- Low values of elastic moduli for slab and base layer
- Low k-value for subgrade

The well performing sections, on the other hand, had the following in common:

- Smaller crack spacing
- Lower IRI (selection criteria)
- Shallow depth to reinforcement
- Thinner and stronger slab
- Stiffer base and subgrade layers

Unfortunately, these insights are of limited value with respect to newer CRCP performance in South Dakota as relatively smaller crack spacings of 1.5-4' are typical of newer sections indicating the possibility that the primary source of our problems may lie outside the realm of the variables examined in the LTPP analysis except for the reinforcement depth and slab thickness. The primary performance problem with CRCP pavements nationwide has been ascribed to too great a distance between transverse cracks resulting in excessive crack widths and accelerated distress. The increase in design slab thickness from 8" to 10-11" in some of the newer CRCP pavements may also be a factor but this is not supported by thinner slabs exhibiting the same cracking patterns.

In an effort to glean any useable results possible from the LTPP database, a preliminary assessment of the original data analysis methods was made. The first potential shortcoming of the 1999 analysis became obvious comparing crack spacings derived from both manual and PADIAS (Pavement Distress Analysis System) surveys. The PADIAS data is based on interpretation of 35 mm photographs, tends to be significantly different than the manual data and frequently has a lower value. The LTPP researchers used the highest value, either PADIAS or manual, for their analysis which presupposes at least equivalent accuracy for the PADIAS approach. All sections which had only PADIAS crack spacing values were excluded from the database used for the new analysis. Next, the researchers assumed that the initial

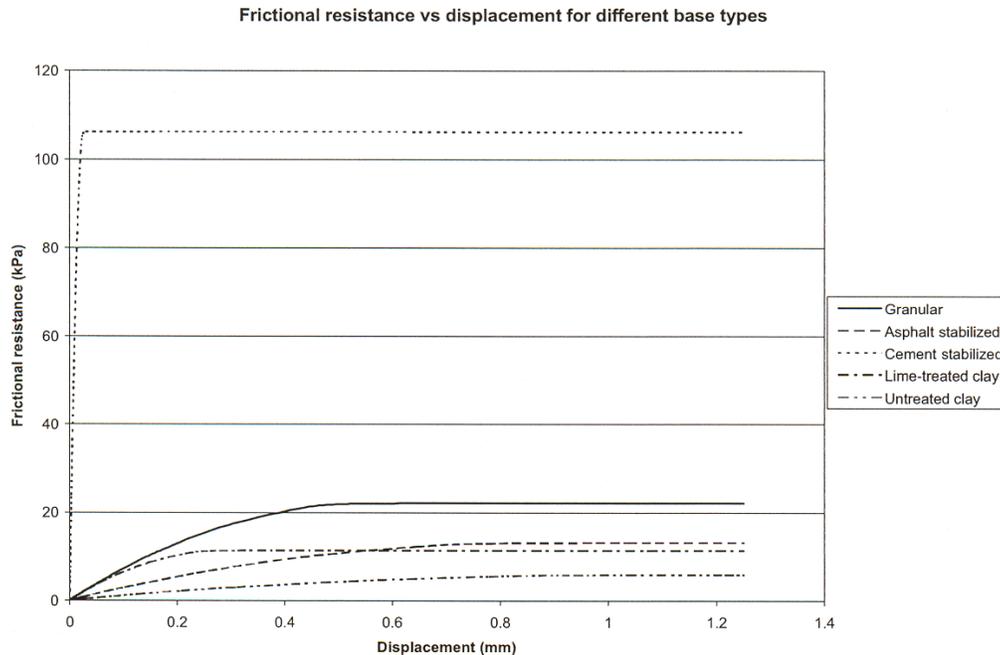


Figure 6: Typical Friction Resistance Relationships

cracking response from the individual LTPP sections could all be analyzed as a single dataset with no significant differences in cracking due to the base type for each section. Figure 6 (14) illustrates the significantly different frictional resistance response from various base types, especially cement stabilized ones where the adhesion between the base and portland cement concrete is so strong that large compressive forces build up, resulting in tensile stresses when the base acts to resist movement by the slab during expansion or contraction. The other base types yield at much lower levels of stress.

The database was separated into granular, asphalt-treated aggregate, cement-treated aggregate and cement/lime-stabilized soil subgroups and these were analyzed for significant independent variables contributing to transverse cracking in the LTPP CRCP test sections. In addition, the dependent variable used for the analysis was not the average crack spacing in feet for each 500' section but was, instead, the number of cracks counted in each section. Using the crack spacing creates problems with the analysis due to the fact that, in this form, it is inversely related to several of the independent variables. The results of these analyses are shown in Tables 1 and 2 and in Figures 7-10 and demonstrate the fact that statistically significant relationships between cracking and several independent variables do exist ($p < .05$) but the response to each factor is critically dependent on the base type. An additional factor in the analysis of each subgroup was the number of sections with sufficient available data inclusion. One section (46-5025) in the granular subgroup was mislabeled as a granular base when it actually had a lime-treated gravel cushion. It was included with the soil cement subgroup. Two sections, one each in the granular and ACM subgroups were excluded as extreme outliers with the result being an improved significance to the models with no substantial change in factors. The comparison of average values across all base types allows clarification of why some factors may be significant for one base type but not another.

The most interesting aspect of the statistical results is the verification that the cement-aggregate subgroup demonstrated a strikingly different model than either the granular or AC section models. Both the

granular and AC models showed expected relationships with slab thickness and % steel. Increasing slab thickness resulted in a decrease in cracking whereas increasing % steel correlated with an increase in cracking. The cement-aggregate sections, on the other hand, displayed exactly the opposite situation with an increase in slab thickness apparently contributing to more cracking and an increase in steel content reducing cracking. The soil cement, cement-treated and lime-treated subgrade subgroup showed a mixed response with increasing slab thickness reducing cracking and increasing steel doing the same thing. Obviously, the slab-base bond is directly affecting the early-age cracking patterns generated in the CRC.

Table 3: LTPP CRCP Test Section Variables

Factor	Base Type							
	Granular n=10		ACM n=32		SC, LT, CT n=6		CAM n=17	
	Average	S. D.	Average	S. D.	Average	S. D.	Average	S. D.
Number of Cracks	116.50	30.80	146.66	51.11	173.33	69.23	133.41	52.83
Age (years)	17.5	3.92	15.66	6.06	18.5	3.39	14.23	8.10
Slab Thickness (in.)	9.19	1.59	8.74	1.20	8.10	0.51	8.95	0.95
% Steel	0.613	0.049	0.593	0.084	0.638	0.058	0.597 n=16	0.074
Steel Depth (in.)	3.44 n=9	0.46	4.07 n=29	0.76	3.30	0.65	3.78 n=16	0.73
Steel Ratio	0.386 n=9	0.084	0.463 n=29	0.052	0.407	0.075	0.424 n=16	0.080
Lap Length (ft.)	19.0 n=8	1.77	28.27	16.92	22.00	6.93	21.00 n=16	12.03
Longitudinal Bar Diameter (in.)	0.663	0.103	0.678	0.077	0.638	0.020	0.685 n=16	0.065
Transverse Bar Diameter (in.)	0.644 n=7	0.253	0.514	0.041	0.562	0.087	0.535 n=11	0.081
Longitudinal Spacing (mm)	180.97 n=9	59.77	182.00	25.40	156.63	6.56	175.40 n=16	26.09
Transverse Spacing (mm)	999.12 n=6	307.57	892.61	188.29	944.92 n=5	198.74	1140.76 n=11	368.63
Temperature Difference (°C)	11.69	1.27	13.01	1.57	12.88	1.64	12.55	1.66
Concrete Modulus ((GPa))	32.31	7.79	31.88 n=12	5.84	27.05	6.63	31.44 n=16	6.37
Base Thickness (mm)	182.90	124.81	104.38	34.34	110.00	37.72	129.18	35.18
Base Modulus (GPa)	5.96	1.03	6.43	1.54	5.98	1.67	6.42 n=16	1.63
Subgrade k Value (MPa/mm)	57.70	23.07	89.84	38.48	59.33	32.73	80.25 n=16	38.09
Precipitation (mm)	367.4	308.31	184.63	245.57	232.67	274.25	215.53	250.78
Freeze Index (°C days)	1015.10	155.20	892.25	350.93	1013.83	401.00	968.65	407.90
ESAL's (18 kip)	16,004	19484	7422	10951	9938	15019	7620	5896
ACM-Dense-graded AC Hot Mix; SC-Soil Cement; CT-Cement-treated Subgrade Soil; LT-Lime-treated Subgrade Soil; CAM-Cement-Aggregate Mixture								

All factors listed in the LTPP report tables, as well as data taken from the LTPP DataPave online database, were examined for significance and the final models were formulated on a $\alpha < 0.05$ for inclusion. Actual slab thickness values from this database were used for analysis as well as lap length and longitudinal and transverse bar diameter. The steel ratio factor listed in the table below is the ratio between steel depth and slab thickness and was included as a factor in all analyses but only met the inclusion criterion for the largest dataset, the ACM sections.

Table 4: Statistical Model Results for LTPP CRCP Test Sections

Factor	Base Type							
	Granular n=8	p value/ F	ACM n=28	p value/ F	SC, LT, CT n=6	p value/ F	CAM n=16	p value/ F
Model R²	0.993	0/254.3	0.817	0/12.74	0.995	0.007/136.6	0.929	0/19.48
	Coefficient	p value	Coefficient	p value	Coefficient	p value	Coefficient	p value
Constant	-365.103	0	3520.53	0	2093.52	.002	-204.461	0.045
Slab Thickness	-21.79	0.002	-444.04	0	-137.508	0.003	21.754	0.008
% Steel	861.60	0	339.37	0	-1201.68	0.003	-505.302	0
Steel Depth	35.29	0	917.22	0	(-2.297)	(0.835)	(10.55)	(0.310)
Steel Ratio	(-359.29)	(0.243)	-7898.03	0	(-18.02)	(0.837)	(97.66)	(0.345)
Lap Length	(-0.681)	(0.522)	(0.006)	(0.990)	(0.782)	(0.705)	1.556	0.004
Placement	(1.799)	(0.375)	31.17	0.005	-	-	-31.98	0.047
Temperature Difference	(-1.122)	(0.536)	11.87	0.001	(7.082)	(.086)	28.463	0
Concrete Modulus	(0.066)	(0.756)	(6.5) (n=11)	(0.038)	(-1.147)	(0.147)	2.3	0.025
Base Thickness	(-0.007)	(0.798)	0.554	0.001	(-0.082)	(0.729)	(-0.096)	(0.764)
Subgrade k Value	(-0.115)	(0.117)	(-0.042)	(0.745)	-0.662	0.024	(-0.039)	(0.813)
ACM-Dense-graded AC Hot Mix; SC-Soil Cement; CT-Cement-treated Subgrade Soil; LT-Lime-treated Subgrade Soil; CAM-Cement-Aggregate Mixture								

The issue of slab-base bond strength is of importance with respect to the level of stresses generated within the CRCP pavement by hydration, steel-concrete bond development, shrinkage and temperature and moisture gradients. The cement-aggregate base forces the generation of relatively closely spaced cracks in the pavement and was specifically adopted for this reason due to performance problems related to widely spaced, open cracks in CRCP pavements that result in a reduction in service life under traffic. The high tensile stresses generated by the bonded base, as well as the cracking within the base layer, achieve the goal of reducing crack spacing by exacerbating the temperature response of the pavement section.

The statistical analyses of the ACM and CAM sections was only possible after including the temperature difference for the sections as an independent variable. Prior to this point, the independent variables meeting the acceptance criterion of $\alpha < 0.05$ could only explain less than half the variance. Interestingly, temperature difference is on the threshold of acceptance into the model for soil cement, cement-treated and lime-treated subgrades at $p=0.117$ (shown in parenthesis in Table I), even though the dataset consists of only six sections, implying that it probably is a significant independent variable masked by the lack of data. For all three base types, an increase in temperature difference results in an increase in cracking. Surprisingly, granular bases do not exhibit any marked response to temperature difference ($p=0.536$) and even show a negative coefficient, indicating that temperature difference affects this base type the least,

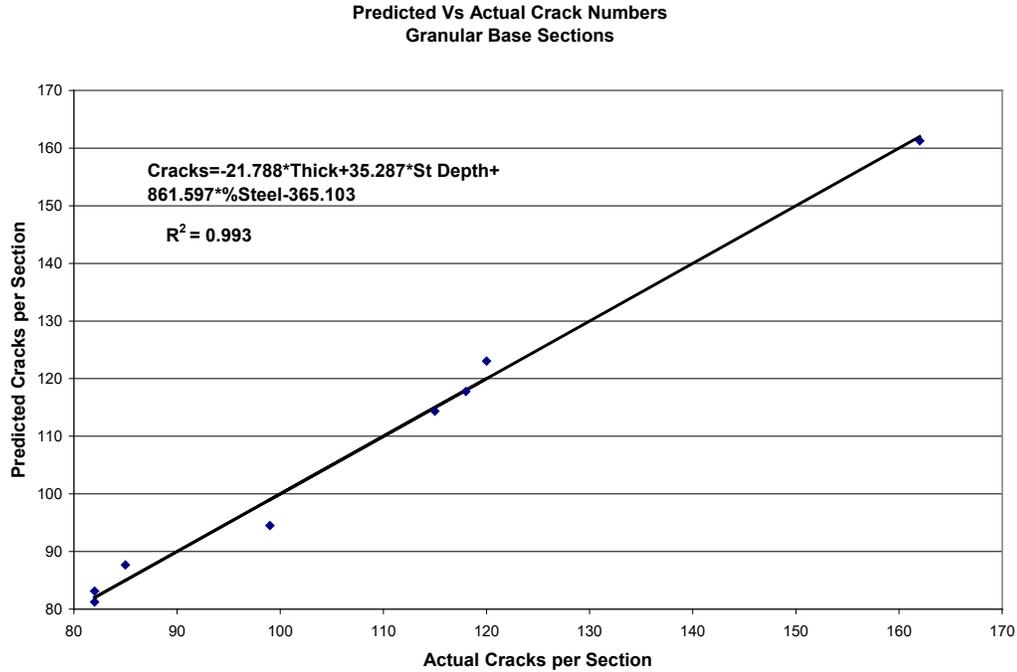


Figure 7: Granular Sections Model

implying that the majority of the cracking in CRCP with unbonded granular bases occurs at early ages. All coefficients and p values in parenthesis were forced into each model to see how close to significance they came but are not considered as part of the models.

The lack of data presents a problem in terms of evaluating the significance of other independent variables in the database. Only the ACM (n=28) and CAM (n=16) subgroups have sufficient data to allow the addition of other statistically significant factors. Interestingly, both these subgroups have placement as a significant contributor to transverse cracking with the only difference between the two being the signs are reversed. Three types of placement techniques were employed in constructing the pavement sections—chairs, mechanical and other. The reversal of the signs between these two groups is consistent with the signs for slab thickness and % steel also being reversed and lends support to the response difference being a real phenomenon and not a statistical fluke. This is borne out by lap length also being significant, but only for the CAM sections, even though there was no statistically significant difference between lap lengths in the CAM sections and the others (p = 0.256).

Steel depth is also interesting as a factor as it is only significant for the two sections (Granular and ACM) where % steel correlates with an increase in cracking but is not significant for the stabilized subgrade or cement-treated base sections. The ACM sections had a significantly higher steel depth than all other base types except stabilized bases (p = 0.011) whereas the granular sections had a significantly lower steel depth than the others (p = 0.034). The fact that these two base types represent the extremes of shallow and deep longitudinal steel may explain why this variable was important for them but not the other types. Considering each base type was analyzed separately, this does not explain the lack of significance of steel depth for the stabilized and CAM types as both of these had sufficient range in steel depth values for any significance to become apparent. This is also consistent with the strongly bonded bases changing the

interactions of the steel in the cracking process. The strong dependence of cracking response to concrete modulus also argues in favor of this difference as laboratory concrete modulus clearly provides a positive effect on transverse cracking. Both the ACM and stabilized subgroups verge on significance with respect to concrete modulus, and it probably would be significant if more data were available-only 11 of the 28 ACM sections in the analysis had laboratory modulus results. There is a good likelihood that the ACM model fit would be markedly improved with additional concrete modulus data. Again, granular proves the exception as the concrete modulus was not significant in the model, even though all sections used for the analysis had modulus values in the dataset.

Base thickness proved significant only for the ACM sections with none of the other subgroups even approaching the threshold of significance. This may be due to the fact that ACM base thicknesses averaged much lower than any other base type ($p = 0.015$). Subgrade k values were only important for the stabilized subgrade sections and, probably, for the granular sections, especially considering these values were not measured but were backcalculated from deflection data. It is not surprising that subgrade support is more critical for these two base types but these two base types also had significantly lower k values compared to the ACM and CAM sections ($p = 0.002$). Backcalculated base and slab modulus values were also included in the analyses but did not show any significance.

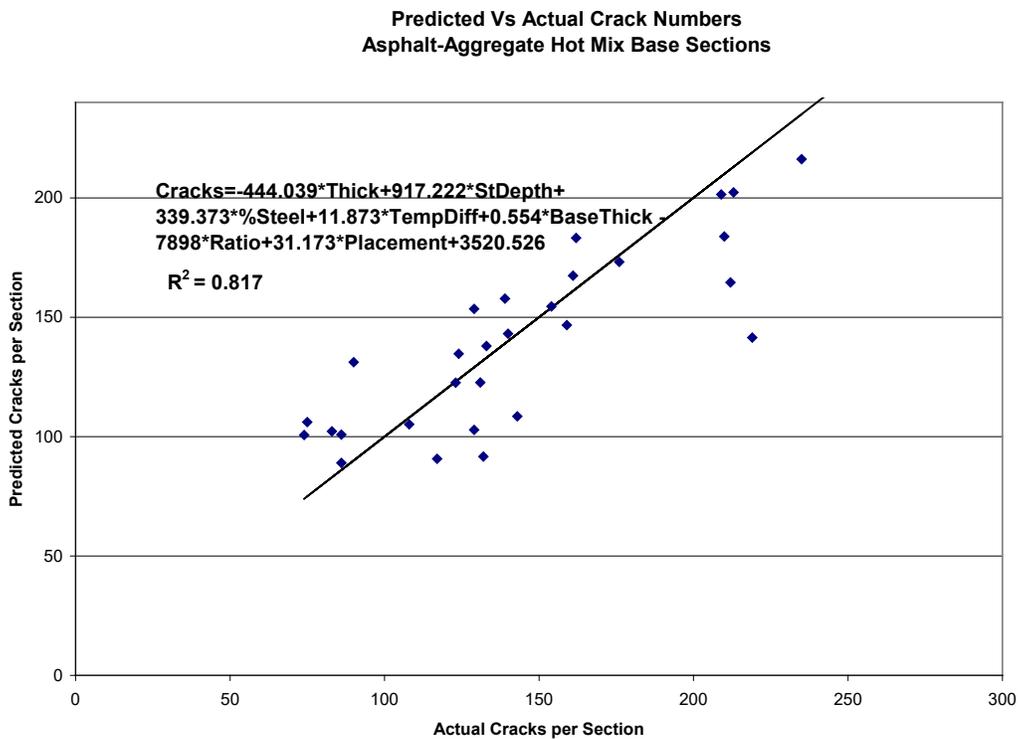


Figure 8: ACM Sections Model

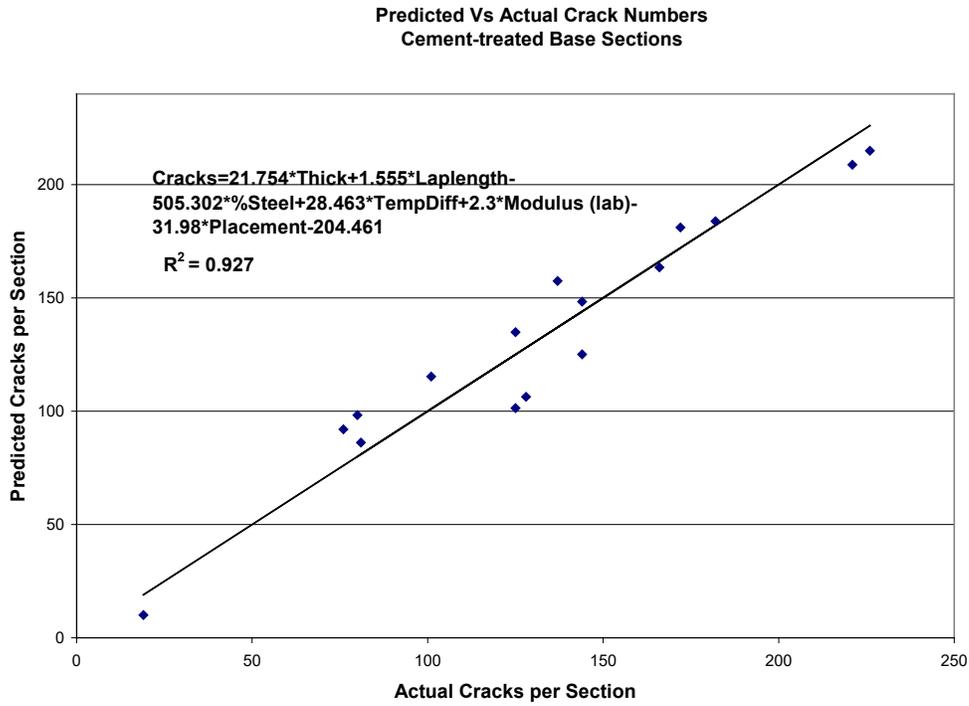


Figure 10: CAM Sections Model

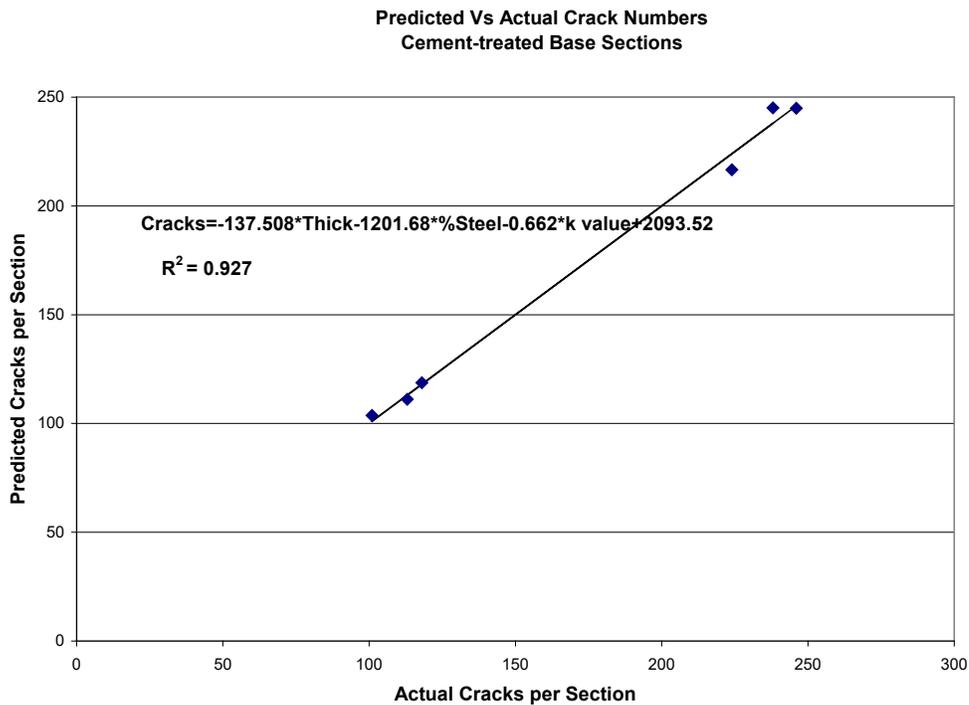


Figure 9: Stabilized Cement and Lime Subgrade Sections

TASK 3: CONTACT OTHER STATES

Through direct contact with other states—such as North Dakota, Illinois, Texas, Michigan, and Oregon—that have recently built CRCP pavements, as well as states that have decided to stop building CRC, evaluate their experience regarding the incidence and causation of unexpected distress.

Dr. Jeffery R. Roesler, University of Illinois, Urbana-Champaign was contacted with regard to performance results obtained to date on a series of CRCP test sections constructed as part of an Illinois DOT-sponsored research project. Although no report is available on the performance, he did send an advanced copy of a report detailing crack spacing and width measurements on the test sections indicating crack spacing decreased with increasing steel percentages as did crack width. The test pavements had an AC-stabilized base.

Texas DOT summarized 50 years of CRCP experience in a document posted on the CRSI web page entitled “*CRCP in Texas; Five Decades of Experience.*” In the document, they make a strong case for aggregate type and paving conditions being the primary factors for crack spacing, everything else being equal. They state that siliceous aggregate, with a higher coefficient of thermal expansion and modulus of elasticity tends to generate much closer crack spacing than limestone aggregate. They also state that lower compressive strength concrete also results in closer cracks. This could be a definite factor considering the use of 15-20% modified Class F fly ash in all new CRCP pavements in South Dakota. Texas DOT also noted a significant increase in y-cracking, network cracking and cluster cracking as well as a reduced crack spacing when paving occurred at temperatures >90°F.

Dr. Moon Won, formerly of Texas DOT and now at the University of Texas at Austin, is conducting a study on cracking in CRCP in Texas primarily attempting to understand and address the occurrence of crack spalling on new CRC. He believes the spalling is related to moisture and thermal stresses in the new CRCP and has carried out extensive measurements of stress levels, thermal gradients and moisture distribution in pavements in various climatic zones of Texas. He has been tasked with the development of tailored CRCP designs for different climate zones if the data indicates this will improve performance.

Mr. Harold D. French, Virginia DOT, stated that the current problem being experienced with CRCP in Virginia involves extensive longitudinal cracking on new projects which he tentatively ascribes to drying shrinkage. He characterized the cracking as follows:

- The longitudinal cracks are mostly in the middle of the lane, but are also near the pavement edge.
- The cracks seem to be very tight.
- The soil is very soft and the water table is pretty high—the subgrade is a soil-cement mixture.
- The crack spacing varies between 2-8 ft.
- A bit of meandering and Y-cracking, and an isolated incidence of possible pumping

TASK 4: ASSEMBLE RECORDS

Assemble and review available design and construction records for CRCP pavements constructed in South Dakota.

Available data was collected for projects built since 1995. Earlier CRCP projects built from 1968 to 1975 incorporated the same design elements throughout with a thickness of 8", 0.6% steel (#5 bars) a steel depth of 2½" (±½") with the only variable being base type. The earliest CRCP projects utilized a 9" granular base with later projects employing a 4" lime-treated gravel cushion. A summary of the available design data is shown below in Table 5 and discussed in Task 6. Daily paving reports and other project data were unavailable without exhaustive effort and, since manpower was unavailable, a decision was made to evaluate environmental, material and curing factors by concentrating on one project incorporating all other recommended design, material and construction modifications discussed in Task 5.

TASK 5: OBSERVE CRCP CONSTRUCTION

Through observation of active CRCP projects built in South Dakota during 2004 and early 2005, describe routine and exceptional construction processes that may contribute to unexpected levels of distress.

Construction of CRCP projects in Rapid City and Sioux Falls was monitored and measurements of crack evolution and distribution were made on a routine basis. Strain gauges were also installed to monitor early age stress. Lapping locations were mapped out along 1000' feet sections in each project to determine, as crack development proceeded, whether the lapping detail being used was contributing to cracking, possibly due to stress concentration. No direct influence of lapping on crack development was apparent. Alarmingly, the I-90 eastbound CRCP project, near Deadwood Avenue in Rapid City, exhibited small areas of spalling within three weeks of construction. The Sioux Falls project (I-29 at Madison Street) was instrumented with steel and concrete strain gauges as well as temperature probes to look directly at strain development with time during early ages. Unfortunately, a software glitch prevented the collection of any data during the critical early stages of strain development. The Sioux Falls project was also constructed using a concrete mixture with a larger maximum aggregate size (1½") than other projects built since 1995 but similar to the aggregate top size used in the older CRC. This mix was also formulated with an optimized gradation that minimizes cement content while providing uniform, workable concrete. Larger aggregate results in greater fracture resistance, bond strength and aggregate interlock in the concrete and the I-29 project provided invaluable information on how much of a contribution to poor performance the reduced maximum aggregate size has made. This project did not exhibit visible cracking one week after construction and 3 weeks after paving had cracks averaging 20' apart which were extremely tight as illustrated in Figure 11 indicating that the change in aggregate size does appear to have had a beneficial effect. Crack development over the winter months did not alter this improvement in cracking pattern.

The westbound I-90 project near Deadwood Avenue in Rapid City was constructed beginning in the spring of 2005. The only differences between the WB and EB sections were the increase in aggregate size and a reduction in cement content due to the optimized gradation used. Again, there was a substantial improvement in the cracking pattern generated compared to the eastbound lanes. Though the cracks were much tighter, however, they still exhibited a tendency toward clustering and excess wandering.



Figure 11: Early Cracking on I-29 Madison Street in Sioux Falls



Figure 12: Early Cracking on I-90 Deadwood Avenue in Rapid City

Analysis of Hanson County Pavement Cracking IM 90-8(43)344 EB

One of the principal tasks outlined in this research project was to examine the influence of concrete mixture design and environmental factors on cracking. This project, built in 2005, was selected as an ideal candidate for attempting to monitor the influence of these factors on cracking in view of the project length and pavement design. The concrete mixture design called for use of an optimized 1½” nominal mixture

with the steel content at 0.6% of the pavement cross-section. Using a reduced steel content and the larger aggregate concrete mixture should, it was hoped, allow a clearer indication of the effects of material and environmental properties on the ultimate cracking distribution. The adjacent westbound pavement, paved in 2004, would serve as a control using the original design and smaller aggregate mix.

Paving Sequence

Mainline paving for the project covered the period from August 10, 2005 to September 6, 2005. Paving began at the west end of the project and proceeded east until the last two days of paving. Paving was done from east to west on these days to insure paving away from the eastern terminal. Weather conditions for the time period involved were obtained from the Mitchell Airport database as this was the closest reporting station with information including relative humidity and dew point. There were no precipitation events during paving operations and temperatures ranged from a high of 91°F to a low of 50°F with an average of 71°F during this time. Wind speed ranged from 0 to 25 miles per hour and averaged a little over 9 miles per hour for the period. Relative humidity ranged from 18 to 97 % with corresponding dew point from 39° to 73°F.

The initial concrete mixture design for paving (Mix 1 in Table 5) is shown below. Unfortunately, the optimized 1½” aggregate laboratory approved mixture design proved unworkable and construction using it was promptly halted after only 930’ of pavement was placed on the first day as the mix proved too harsh to maintain normal paving operations in spite of increasing the water reducer in the mixture above manufacturer’s recommendations. The contractor switched to a conventional ¾” concrete mixture with a cement content of 480 lbs. to maintain paving operations (Mix 2). More than a mile and a half of pavement was placed using this mixture. Meanwhile, the concrete mixture design was modified after discussion resulting in a 25 lb. increase in cement content to 465 lbs./yd³. The aggregate gradation was also modified to eliminate 10% of the quartzite chip rock aggregate and the sand content was increased to 40% from 36% (Mix 3). The remainder of the mainline paving was accomplished using the modified mix except for a short section that was inadvertently paved with Mix 2.

Data Collection and Analysis

Crack frequencies were determined for each 500’ section of pavement throughout the project, excluding the first day’s paving and the section using Mix 2 paved on August 18, 2005. Transition areas between daily paving operations were also excluded to eliminate data overlap. Since Mix 2 was essentially identical to the mix used for the original control pavement in the westbound lanes and almost 10,800’ were paved using this mix, the control sections were changed to the EB sections for an optimum comparison of cracking frequencies between concrete mixtures. Air content, slump, unit weight, water-cement ratio, 28 day compressive strength, concrete temperature, air temperature, average daily paving rate, relative humidity, wind velocity, dew point and evaporation rate data were all compiled and averaged for each day’s paving prior to data analysis.

Table 5: Concrete Mixture Designs used on IM 90-8(43)344 EB

Ingredient	Original Job Mix		Normal Paving Mix		Modified Job Mix	
	Lbs./yd ³	(%)	Lbs./yd ³	(%)	Lbs./yd ³	(%)
Cement (Dacotah I-II)	440	11.3	480	12.6	465	12.0
Fly Ash (Coal Creek)	110	2.8	120	3.1	110	3.0
Water	212	5.4	243	6.4	227	5.8
Sand (Opperman)	1132	29.0	1236	32.4	1233	31.7
1½" (Spencer)	679	17.4	-	-	462	11.9
1" (Spencer)	878	22.5	1740	45.6	1165	30.0
Chip (Spencer)	452	11.6	-	-	222	5.7
Air Content	-	5.5	-	6.5	-	5.5
w/c	0.385		0.405		0.395	

A plot of cracking for each 500' section versus stationing (Figure 13) gives an idea of the range in cracking manifested over the course of mainline construction. The variable plotted on the right hand axis is concrete temperature, the independent variable with the strongest correlation, not surprisingly, to cracking. Two marked trends are immediately apparent from the graph besides the general correspondence with concrete temperature. The first is the substantial reduction in the level of cracking when the concrete mixture is changed from Mix 2 (¾") to Mix 3 (1½") at relatively similar concrete temperatures. The second, somewhat more surprising trend is the minimal cracking which occurred during the final day of paving at the east end of the project. Notably, this pavement section was on the east side of a small bridge and displayed one very striking visual difference compared to all the other sections surveyed. This was the only pavement section with curing compound residue still visible along the shoulders almost a year after paving was completed.

Figure 14 represents a normal distribution comparison of cracking between Mix 2 and Mix 3, generated as a result of a two sample t test of the sample means where the null hypothesis was that the cracking in the Mix 2 sections was equal to that in the Mix 3 sections. The null hypothesis was rejected ($p = 0.0042$) with the means equal to 219.8 for Mix 2 and 189.3 for Mix 3. The standard deviations for the two types were significantly different with Mix 2 exhibiting almost no variability (S.D. = 3.96) whereas Mix 3 had almost eight times the range (S.D. = 23.36). Because concrete temperature was identified as a prime factor in determining the level of cracking, the possibility that this difference in the means was actually due to the Mix 2 concrete temperatures being higher had to be tested. The results are shown in Figure 15 and show that there was no significant difference in concrete temperatures for the two mixes.

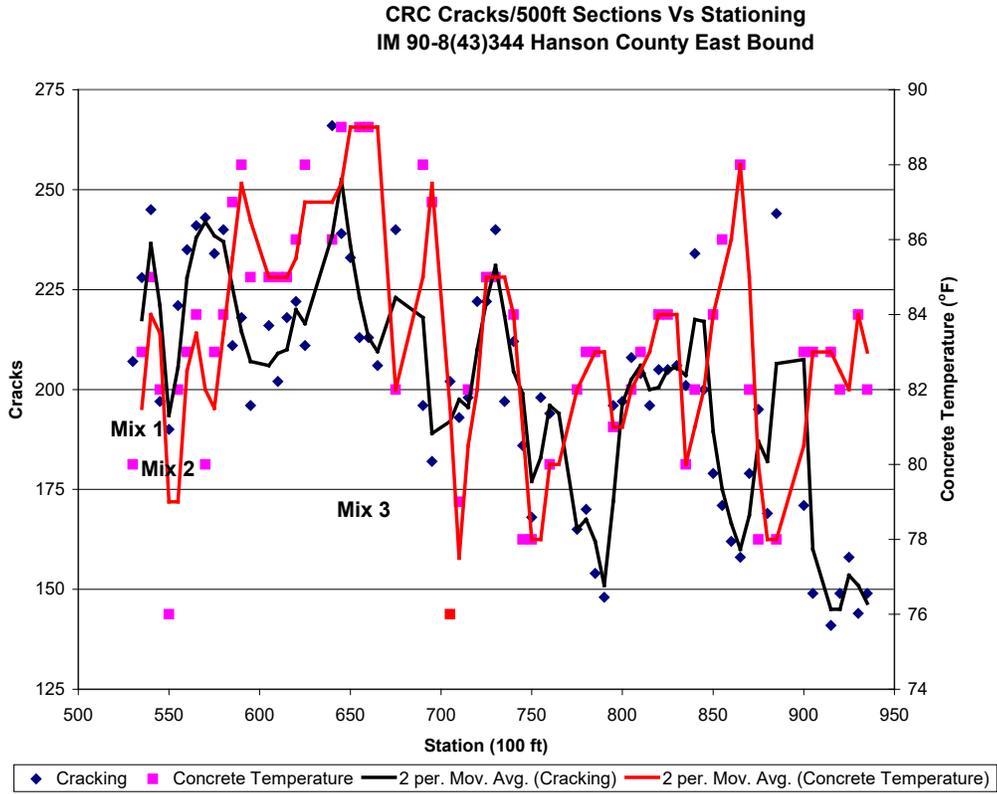


Figure 13: CRCP Cracking and Concrete Temperature versus Stationing

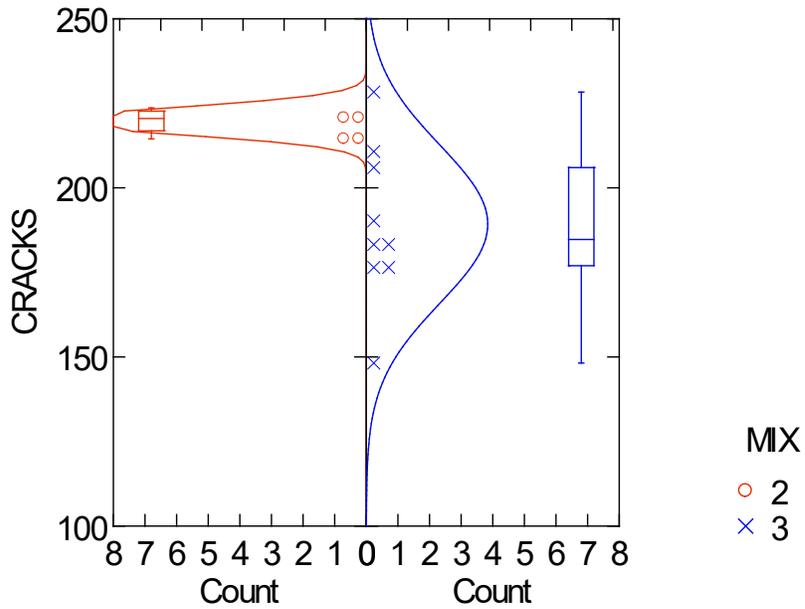


Figure 14: Normal Distribution Curves for Mix 2 and Mix 3 Cracking

Linear regression was used to analyze possible relationships between various concrete mixture and environmental factors for the project and the degree of cracking which occurred. In all cases, averages for each days concrete production as well as average weather condition parameters such as air temperature, wind velocity, etc. were used in the analysis. Acceptance of an independent variable in the final model was based on $\alpha < 0.05$ with p values greater than this rejected from consideration. Of the 13 independent variables tested, nine were concrete mix properties including air content, slump, unit weight, water-cement ratio, concrete temperature, 28 day compressive strength, water and cement content and average daily paving rate. Air temperature, the difference between air and concrete temperature, relative humidity, wind velocity, dew point and evaporation rate (as calculated using the ACI nomograph) were the environmental factors examined.

A satisfactory and somewhat puzzling model resulted from the analysis. Only five factors met the criterion for acceptance-average concrete temperature, water-cement ratio, wind velocity, air temperature and mix. Mix was included as a dummy variable as it was known to affect the cracking and using it allowed all mainline paving to be included in the analysis. The output for the model is shown below in Table 6. The most surprising aspect of the model is the effects certain variables have on cracking levels. To insure that the model was not a product of a small dataset and coincidence, a similar analysis was done on pavement sections where Mix 3 only was used in the concrete. The results were substantially the same with the coefficients almost identical.

The original analysis used air temperature as an independent variable yielding a significant correlation. To address the question of confounding due to colinearity, the temperature difference between concrete temperature and air temperature (Temp Diff in the model) was substituted with no change in the output beyond a shift in the coefficient, indicating that the temperature difference between the air and concrete was actually the critical variable. This difference was calculated by subtracting the average air temperatures during daily paving operations from the average concrete temperatures measured over the course of that day's paving. Only those hourly air temperatures for the time paving occurred were used. The fact that air temperatures higher than concrete temperatures result in a negative difference and less cracking indicates that the thermal aspects of the concrete are a significant contributing factor to crack propagation. This is not surprising considering that the Sioux quartzite coarse aggregate is one of the most thermally active aggregate in the country ($CTE \approx 9 \times 10^{-6}$ in/in) famous for blowups and sawing difficulties

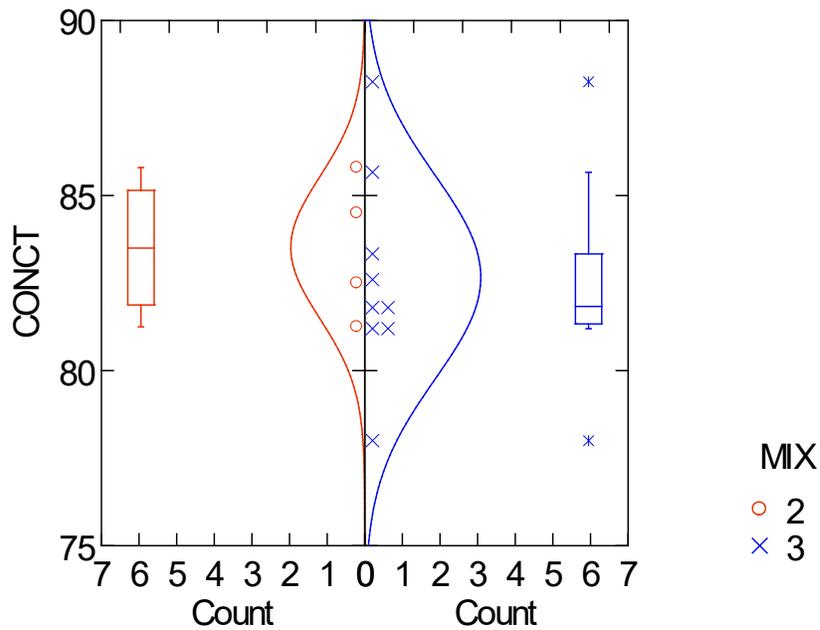


Figure 15: Comparison of Concrete Temperatures for Mix 2 and Mix 3

Table 6: Linear Regression Output for Cracking Model

Dependent Variable: CRACKS		N: 13		Multiple R: 0.983 Squared multiple R: 0.967	
Adjusted squared multiple R: 0.943			Standard error of estimate: 5.725		
Effect	Coefficient	Std Error	t	P(2 Tail)	
CONSTANT	525.76	133.09	3.95	0.00553	
CONCT	5.69	0.68	8.33	0.00007	
TEMPDIFF	2.47	0.40	6.20	0.00044	
WINDVEL	-1.73	0.44	-3.89	0.00597	
WC	-1486.57	275.88	-5.39	0.00102	
MIX	-55.51	6.10	-9.09	0.00004	
Analysis of Variance					
Source	Sum-of-Squares	df	Mean-Square	F-ratio	P
Regression	6761.29	5	1352.25	41.25	0.000048
Residual	229.44	7	32.78		

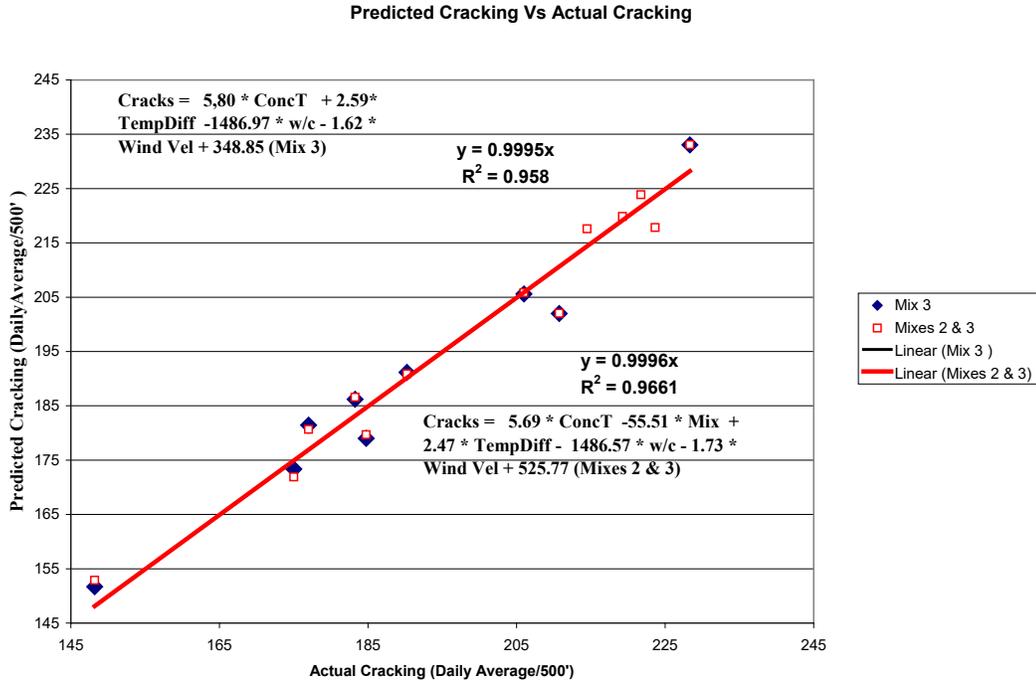


Figure 16: Comparison of Cracking Models

The effect of concrete temperature on cracking also argues for thermal effects being crucial to the level of cracking. The hotter the concrete as delivered to the paver, the more frequent the cracking which occurs. This is also in line with the wind velocity actually having a negative effect on cracking. Normally, higher wind speed means a greater evaporation rate and an increased likelihood of plastic shrinkage cracking at low relative humidity. The wind dries the concrete out but it also provides for more rapid equilibration between the air and concrete surface. This is consistent with the fact that air temperatures actually greater than the concrete temperature result in a reduction in cracking. A correlation between concrete temperature and air temperature using linear regression gave an R² value of 0.420. Including the average overnight air temperature (6 PM to 6 AM) to reflect the relative cooling of the aggregate stockpiles yielded an improved R² of 0.762, showing the not unexpected influence of the aggregate temperature on the concrete temperature and the decided benefit cooling the aggregate stockpiles down or adding ice to the mix water could have on cracking.

One other interesting aspect of the model was the effect of water-cementitious ratio on cracking. Normally, increasing the water content of the concrete increases the likelihood of cracking due to greater drying shrinkage. w/c values for the Hanson County project ranged from 0.418 to 0.451 with the higher water content mixtures associated with less cracking. The reason for this is probably related to the harshness of the concrete and concomitant difficulty in achieving a well consolidated, uniform pavement section. Mix 1 (Table 5) was abandoned as being too unworkable for adequate paving. Mix 2 was used as a stopgap until the original mix design could be modified to yield a workable concrete. The shift downward in cement content, from 510 lbs/yd³ with 112 lbs/yd³ of Class F ash to 440 lbs/yd³ with 110 lbs/yd³ of Class F ash, has resulted in a innately harsher mix with much less leeway with respect to controlling workability by adjusting w/c. The increased water in the mix should allow for easier

consolidation and a reduction in the number of voids and other defects in the pavement that can act as stress concentrators and lead to more rapid, uncontrolled cracking.

Discussion

The cracking frequencies obtained for each day of mainline paving on the Hanson County project only give an indication of the underlying causes for different levels of cracking to occur. They do not indicate the severity of cracking with respect to undesirable cracking such as wide cracks, cluster cracking, network cracking and minor spalling. The cracking exhibited over the 8.21 miles of this project covers a gamut from detrimental to desirable with the level of severity generally decreasing from west to east. Unfortunately, the shift from Mix 2 (¾") to Mix 3 (1½"), although beneficial, did not eliminate undesirable cracking though it did reduce its extent and severity. There is, however, a direct corollary between cracking performance and cracking frequency that provides some insight into what factors are potentially contributing to deleterious cracking.

During the 2005 construction season, test sections with 0.54 and 0.60% steel with 1½" optimized concrete mix were also built at the Tea interchange on I-29. These sections have significantly reduced cracking but, more importantly, the cracks are uniformly tight and there is no evidence yet of any appreciable problem cracking. Unfortunately, these sections were extremely short and did not have sufficient development length for the early stress concentrations that can occur in longer sections of CRCP pavement. The primary portion of the pavement in Hanson County is continuous, with no terminals, for over 7 miles from the west end of the project until the structure at station 894+15. The best performing pavement sections are on the east side of the structure but the run of CRCP pavement in this stretch is less than a mile long. The possibility exists that the primary reason for the differences in cracking is due to the reduced development length also resulting in much lower levels of stress and much more controlled cracking. Fortunately, the last section of the continuous run west of the structure also exhibited improved cracking performance even though it is contiguous with the poorly performing sections further west. Presumably, any environmental or concrete mix factors these segments share which differ substantially from the sections further west could provide insights into the differing performance.

Table 7 lists these parameters for each days paving on the Hanson County project. The marked reduction in average concrete temperature, air temperature and temperature difference for the easternmost sections is readily apparent and, conceivably, a reasonable explanation for the difference. The initial impression that the cracking occurring on our CRCP sections happens early in the life of these pavements is bolstered by these results and the somewhat surprising relationships evident from the cracking model seem to have a rational basis. The strong influence of the thermal environment on crack propagation argues that it is at least as significant, if not more significant, as drying shrinkage in determining the extent and nature of the cracking which occurs. In other words, excessive thermal effects seem to result in uncontrolled cracking whereas, if these effects are minimized, much more controlled and uniform cracking more directly related to drying shrinkage and restraint, occurs. The need to minimize thermal effects as part of normal construction practice on CRCP projects becomes very apparent. Figures 17 and 18 illustrate the difference between acceptable and unacceptable cracking possibly due to these thermal effects.



Figure 17: Cracking in Mix 2



Figure 18: Cracking in Mix 3

Table 7: Paving Parameters for IM90-8(43)344 EB

Date	Begin Station	End Station	Mix	Air	Slump	Average Concrete Temp	Average Air Temp	Temp Difference	w/c	Average Paving Speed	28 Day Compressive Strength	Average Cracking (500')
08/12/2005	527+85	549+74	2	6.53	1.69	82.50	74.25	8.25	0.449	4.92	4440	219.25
08/15/2005	549+74	569+46	2	6.05	1.44	81.25	80.75	0.50	0.432	4.11	4690	221.75
08/16/2005	569+46	602+62	2	6.36	1.42	84.50	84.33	0.17	0.444	4.95	4530	223.70
08/17/2005	602+62	635+80	2	6.38	1.44	85.80	78.75	7.05	0.451	4.81	4950	214.50
08/18/2005	635+80	673+85	2,3	5.30	1.50	88.25	85.50	2.75	0.421	5.54	5145	228.30
08/19/2005	673+85	704+46	3	4.77	1.42	85.67	86.33	-0.67	0.419	6.44	4750	206.00
08/22/2005	704+46	744+54	3	5.08	1.29	81.83	76.17	5.67	0.418	6.12	4980	210.75
08/23/2005	744+54	750+98	3	5.15	1.50	78.00	66.50	11.50	0.422	4.77	--	177.00
08/24/2005	750+98	796+86	3	5.50	1.35	81.80	74.60	7.20	0.428	6.65	4875	175.00
08/25/2005	796+86	835+95	3	5.10	1.05	82.60	74.40	8.20	0.434	6.30	5435	190.25
08/29/2005	835+95	876+95	3	5.23	1.04	83.33	83.50	-0.17	0.431	5.99	4665	184.75
08/30/2005	876+95	894+15	3	5.87	1.50	81.33	79.33	2.00	0.419	5.55	4370	183.25
09/01/2005	945+10	914+65	3	5.05	1.38	81.20	86.25	-5.05	0.430	5.16	4330	148.20

Many of the hot weather concreting techniques used on structural pours would not lend themselves to CRCP paving operations due to economical constraints or constructability issues. Typical measures to minimize thermal effects from high ambient temperatures include night pours, adding ice to the mix water and wetting down aggregate piles prior to mixing, typically the night before. The first two measures are undesirable, if avoidable, but the third offers a practical and inexpensive means of cooling the aggregate stockpiles. In addition, the potential cooling and water retention benefits of curing compound applied in a timely manner at a rate sufficient to insure maximum reflectivity should be explored.

The reduced steel percentage being employed on new CRC, coupled with the larger top size aggregate concrete mix have resulted in significant improvement of excessive and uncontrolled cracking. Not unexpectedly, they are not the entire answer to addressing the problem and optimizing new CRCP cracking and performance. The Hanson County project provides clear evidence of the influence of thermal effects on uncontrolled cracking as well as insights as to how these effects may be minimized. The ultimate modifications to design, materials and construction practices to achieve this goal must be derived from a systematic approach to provide an unambiguous picture of benefit for each change. Otherwise, the interactions of all the factors involved make arriving at a clear understanding of the processes involved extremely difficult.

The initial success of the increased aggregate size in controlling crack development also argues strongly in favor of analyzing the effects steel size and percent are having on the early strength development, especially bond strength, in the pavements. We have gone to a minimum steel size of 3/4" (#6) from the 5/8" (#5) used in all older CRCP pavements. One significant advantage of using smaller diameter steel reinforcement is the reduction in bond stress compared to larger diameter bars. During early strength development, this greater bond stress, coupled with slower strength development due to fly ash concrete, could lead to a greater incidence of uncontrolled cracking or the possibility of localized stress concentration resulting in spalling. One frequently observed distress involves what appears to be crushing failure of concrete along transverse cracks with spalls developing as the smaller pieces of concrete break away from the surface. Construction of further CRCP pavements with larger aggregate under more

normal ambient conditions than those experienced on Madison Street should help to clarify how critical these factors may be.

Ground Penetrating Radar Survey Results

The GPR surveys of several projects immediately yielded results that led to a design change in the spacing of chairs across the pavement section. The transverse steel depth profile on all projects surveyed showed a gradual increase in steel depth from the pavement edge toward centerline, as illustrated in Figure 19. The sagging of steel was presumed to be directly related to construction where the concrete was being dumped at centerline prior to spreading. The revised transverse bar assembly shown in Figure 20 was adopted to increase chair support across the centerline and minimize the likelihood of sagging. The surveys also demonstrated an association of transverse cracks with transverse steel, with as many as 70% of the #4 transverse bars having cracks directly above them. This was consistent with the cracks forming almost immediately after construction.

Figure 19: Typical Transverse Profile from GPR Survey

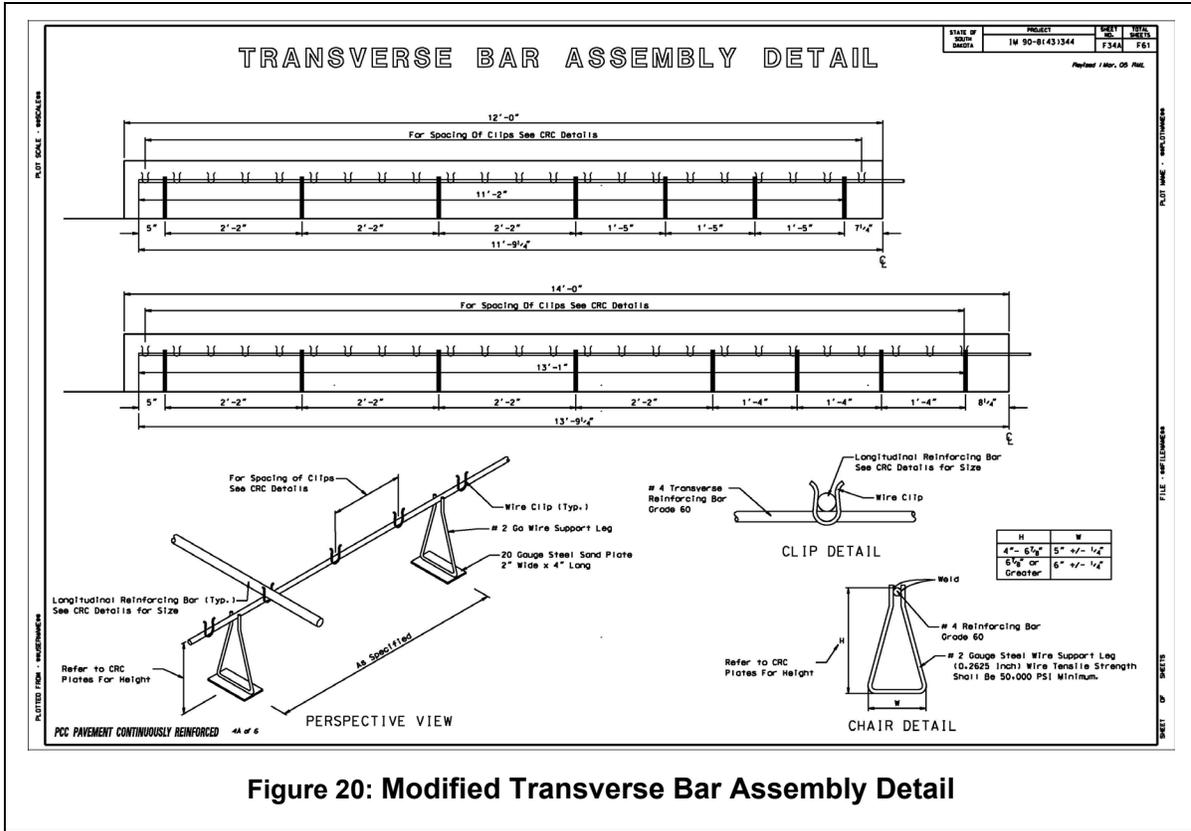


Figure 20: Modified Transverse Bar Assembly Detail

Strain Gauge Measurements

Strain gauges were installed in the southbound lane of I-29 near the Tea intersection north of the bridge on project IM 29-2(52)72 in the control section with optimized 1 1/2" coarse aggregate concrete mixture but incorporating 0.7% steel at 3 1/2" nominal depth. The purpose of this installation was to observe the early age strain behavior of the steel and concrete, measure any differences in strain across a lap section and attempt to confirm that early age stresses are of sufficient magnitude to initiate uncontrolled cracking before the concrete has reached adequate strength levels to effectively resist the stresses. Figure 21 shows the installation of embedded strain gauges (Geokon 4210 Concrete Embedment Strain Gauges) in the CRCP before paving. In all, 14 vibrating wire strain gages were installed—twelve weldable and two embedded—including four on each side of a lap, top, bottom and sides, two at the lap and two on the transverse steel. Figure 22 shows the early strain histories for a few of the gauges, primarily highlighting the response differences between the gauges installed on the steel and those embedded in the concrete.



Figure 21: Embedded Strain Gauge Layout I-29 Tea Interchange

The transverse steel shows the expected response, being perpendicular to the direction of concrete movement resulting in cracking. The transverse strains also cycle with temperature, as expected. The embedded concrete gauges, on the other hand, go into tension almost immediately and achieve high levels of total strain, especially above the steel, during the first day. The magnitude of these strains is sufficient to explain the uncontrolled cracking, wide cracks and spalling which is occurring. This is consistent with the factors affecting crack width in CRCP where cracks which occurred during the first three days were typically wider than cracks formed at later ages (16), especially considering the quartzite coarse aggregate used which also contributes to wider cracks. For comparison purposes, Figure 23 was taken from Nam, et al. (17) and shows the early strain development in a CRCP pavement in Texas. The researchers were able to separate the stress dependent strain (elastic and creep strain) from the stress independent strain (thermal and shrinkage strain), as shown in the figure, but the total strain progression is directly comparable and totally different than that obtained at the I-29 Tea Interchange. The significant early tensile strain in the concrete is consistent with the early cracking being more like tearing and further supports the lack of restraint from the granular base and its effects on accelerating early cracking.

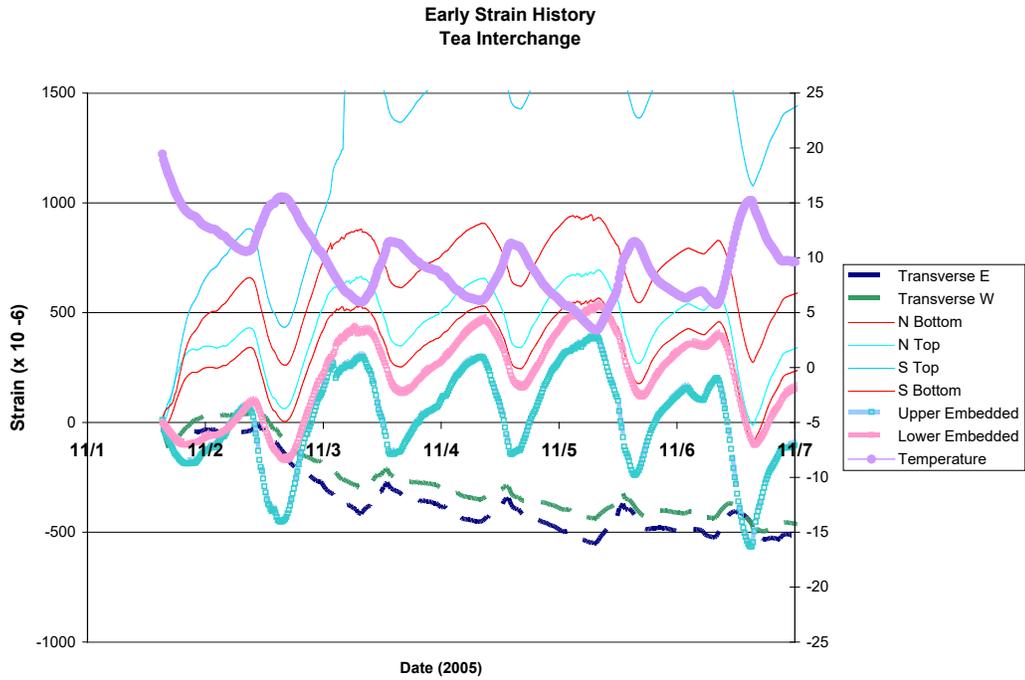


Figure 22: Early Strain Histories I-29 Tea Interchange

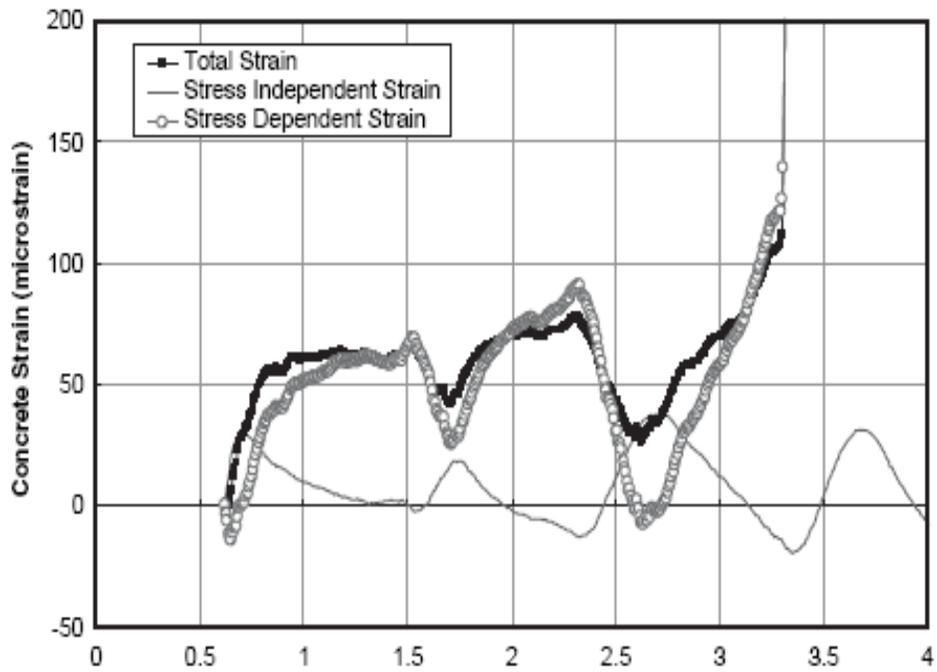


Figure 23: Concrete Strain Variation (after Nam, et. al. 2006)

TASK 6: DESCRIBE AND COMPARE DISTRESS SEVERITY

Through observations and measurements of in-service CRCP pavements, describe the character, extent, and severity of cracking and distress in CRCP pavements constructed since 1995 and compare these observations to concrete constructed prior to 1995 and to desirable CRCP characteristics.

Surveys of CRCP projects are complete with the initial results used to develop refinements in survey procedures. One outgrowth of this effort has been the purchase of a Ground Penetrating Radar (GPR) unit for determining steel depth on select projects. A general observation is that, not surprisingly, quartzite concrete tends to crack at much shorter intervals than limestone. Results to date indicate that there is a wide distribution of performance in newer projects ranging from nominal (slight spalling, narrow crack widths) to unacceptable (severe spalling, wide cracks, pumping) with some projects exhibiting anomalous cracking that may involve steel displacement, early age construction traffic or significant lack of support.

The plan for project surveys involved locating three survey sections of 500' at random and determining average crack spacing. An earlier plan called for measurement of separate sections paved under cool, moderate and high temperature conditions to determine the following:

- crack distribution and orientation
- crack width measurements at shoulder and mid-slab
- steel depths and lapping locations (GPR)
- concrete distress (spalling, network cracking, cluster cracking, corrosion, etc.)
- FWD testing to determine relative base and subgrade support
- condition severity (based on a weighted ranking using the above information)

Initial results from the first survey of pavement sections built at three different temperatures (60°F, 75°F and 100°F near Vivian on I-90) indicate that temperature is indeed an important factor as there was a significant difference in performance in the three sections. Surprisingly, the higher the temperature, the lower the rate of spalling. The actual role of temperature was examined more clearly as discussed in the Hanson County project above.

Project data as well as survey and other testing information were entered into a database for comparison and trend analysis. The results from data collection and processing were used to try and understand the complex interactions between all the performance variables involved.

The initial project surveys involved a crack survey in late fall or early winter on three randomly selected 500' pavement sections, excluding areas near terminals as they exhibited anomalous crack spacings compared to the rest of the pavement. Also, crack width measurements were randomly taken at five locations throughout each section. These measurements were discontinued due to safety concerns and the fact that the results were surprisingly uniform and disappointing. The average crack width value was 20 mils with a standard deviation of only 1.94, indicating that all newly constructed CRCP projects had unacceptably high crack width values. For comparison, the CRCP test sections built on I-90 in 1963, monitored over a period of 5 years, had average crack widths of 5 mils with the maximum crack width measured only 18 mils (15). Also, eliminating crack width measurements broadened the timeframe for project surveying as there was no longer a need to collect data only during the cold months. Initially,

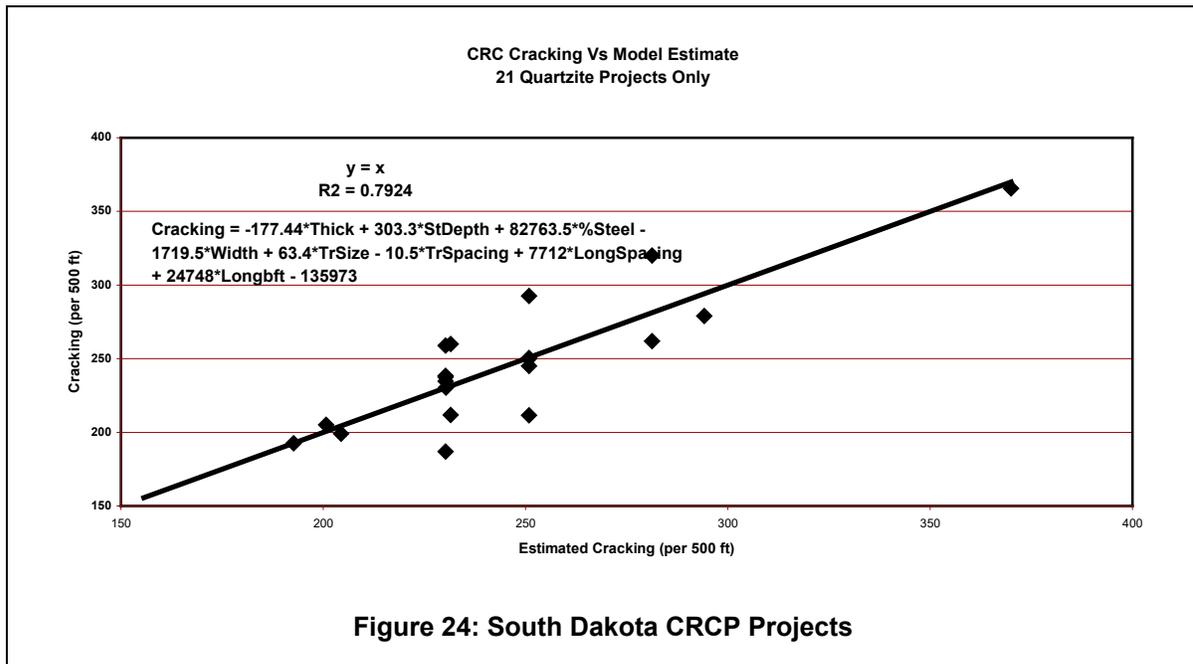
cracking data was collected in terms of number of cracks, number of cracks with spalls and occurrences of Y or network cracking in each 500' section. Y, cluster and network cracking were pervasive, as was spalling, though the spalls were generally quite small. In fact, slight to moderate spalling occurred on anywhere from 25 to 100% of the cracks surveyed statewide. These performance issues were considered a direct result of the design/materials/construction issues that were yielding such undesirable cracking in all new CRCP and no longer monitored so as to speed up the data collection process. The primary variable obtained from the surveys was the number of transverse cracks in 500', as shown in Table 8, and this was used to analyze the effects of various design parameters in a manner similar to the regression analyses conducted on the LTPP CRCP datasets.

Table 8: Design Data for Newer CRCP Projects Surveyed

Quartzite														
Highway	Year	Direction	MRM	Thickness	Width	% Steel	Tr Size	Tr Strength	Tr Spacing	Lap	Stagger	Lg Size	Lg Spacing	Cracking
90	1996	EB	251	10	24	0.7	6	30	48	20	48	6	6.5	320
90	1997	WB	251	10	24	0.7	6	30	48	20	48	6	6.5	262
29	2000	SB	85	8	26	0.66	4	30	48	20	36	6	8	205
90	1995	EB	265	9.5	24	0.7	6	30	48	20	48	6	6.5	365.66
90	1995	WB	265	9.5	24	0.7	6	30	48	20	48	6	6.5	279
90	1997	WB	403	8	26	0.66	4	30	36	20	36	6	8	250.5
90	1998	EB	403	8	26	0.66	4	30	36	20	36	6	8	245
90	2002	WB	334	10.5	26	0.7	4	60	42	25	173.75	6	6	211.7
29	2000	SB	61	11	26	0.7	4	30	48	20	48	6	6.5	199
90	2003	EB	338	10.5	26	0.7	4	60	42	25	197.75	6	6	260
90	2001	WB	213	10	26	0.7	4	30	48	20	48	6	6.5	237.5
90	2000	EB	213	10	24	0.7	4	30	48	20	48	6	6.5	230.33
29	1997	SB	97	8	26	0.66	4	30	36	20	36	6	8	211.66
29	1999	NB	85	8	26	0.66	4	30	36	20	36	6	8	250
29	1998	NB	97	8	26	0.66	4	30	36	20	36	6	8	292.66
29	2001	NB	61	11	26	0.7	4	30	48	20	48	6	6	147
90	2001	WB	353	10	26	0.7	4	30	48	20	48	6	6.5	187
90	2002	EB	353	10	26	0.7	4	60	42	25	197.75	6	6.25	192.66
90	1999	EB	210	10	26	0.7	4	30	48	20	48	6	6.5	234.66
90	1999	WB	210	10	26	0.7	4	30	48	20	48	6	6.5	238.3
29	2001	SB	27	10	26	0.7	4	30	48	20	48	6	6.5	259
Limestone														
Highway	Year	Direction	MRM	Thickness	Width	% Steel	Tr Size	Tr Strength	Tr Spacing	Lap	Stagger	Lg Size	Lg Spacing	Cracking
90	2004	EB	53	11.5		0.7	4	30	36	30	48	7	6.25	134
90	2000	WB	19	10	26	0.7	4	30	48	20	48	6	6.5	146
90	2001	EB	19	10	26	0.7	4	30	48	20	48	6	6.5	121.88

Although three CRCP projects constructed with limestone coarse aggregate were surveyed, they were excluded from the analysis due to their limited number and the fact, as mentioned previously, that they tended to crack much less than the quartzite projects due to limestone's low coefficient of thermal expansion.

A cracking analysis was performed on data from 21 CRCP pavements in South Dakota, constructed since 1995. Slab thickness, % steel and steel depth were all significant independent variables in the model ($p < .05$) with the coefficients having the same signs as those in the LTPP model for granular bases, as shown in Figure 24. Other significant factors included pavement width (24 and 26 feet), transverse spacing and size and longitudinal spacing. The analysis was somewhat confounded by multiple design changes occurring over the course of construction but the results clearly indicate that CRCP pavements with granular bases do not respond the same as those with other base types.



Although the results of this analysis do not directly address the undesirable cracking occurring on our newer CRCP sections, they do provide insight into possible causes. It is interesting to note that, despite the fact that granular base CRCP sections exhibit the least cracking of any pavement section type in the LTPP database, we are experiencing severe, closely spaced cracking on virtually all our newer CRCP pavements, even though the majority of base types are granular. The analysis strongly supports the idea, initially buttressed by the Madison Street I-29 project where larger top size coarse aggregate was used, that the cracking in the newer CRCP pavements is being driven by early age stresses that would normally be insufficient to create unacceptable cracking patterns but are acting on the concrete before sufficient concrete-steel bond formation. Both the reduced top size coarse aggregate and the 20% Modified F fly ash in the concrete are slowing bond strength development and fracture resistance and the cracking exhibited in the CRCP pavements reflects this by its wandering nature and excessive width. If significant volume change is occurring in recently poured slabs with insufficient restraint due to steel-concrete bond strength being inadequate and base friction not fully developed, the cracks formed are both wide and uncontrolled. The additional presence of both cluster cracking, Y-cracking and network cracking as well as crack spalling all argue in favor of the concrete being unable to resist the early tensile stresses adequately.

An example of these conditions is the newly constructed stretch of I-90 eastbound near Exit 55 in Rapid City, SD, which consists of an 11.5" CRCP pavement. Crack mapping was begun soon after construction began and Table 9 below details the results up until the end of October 2004. The progression of spalling

and Y-cracking on this project so soon after construction is somewhat alarming, especially considering the coarse aggregate used is limestone which has a low coefficient of thermal expansion and should minimize any thermal contributions to early age stresses. All of these cracks are also extremely wide as the left image in Figure 25 shows and the spalls appear to be due to initial stresses exacerbated under traffic. This pavement had a nominal longitudinal reinforcing depth of 4" due to its thicker section. The image on the right in Figure 25, on the other hand, is from I-29 Madison Street and shows a much tighter crack even though the pavement thickness was 12" and the nominal steel depth was 4" with #7 longitudinal reinforcement. Since greater steel depth contributes to an increase in transverse cracking as does increasing steel content, based on the granular model above, one change in current design requirements which could be beneficial with no extra cost involved would be to require a nominal steel depth of 3¾" for all new CRCP no matter its depth. Reducing the size of the steel to #5 for all CRCP may also be beneficial as this will also increase crack spacing, if this can be done without reducing the cross sectional area too greatly.

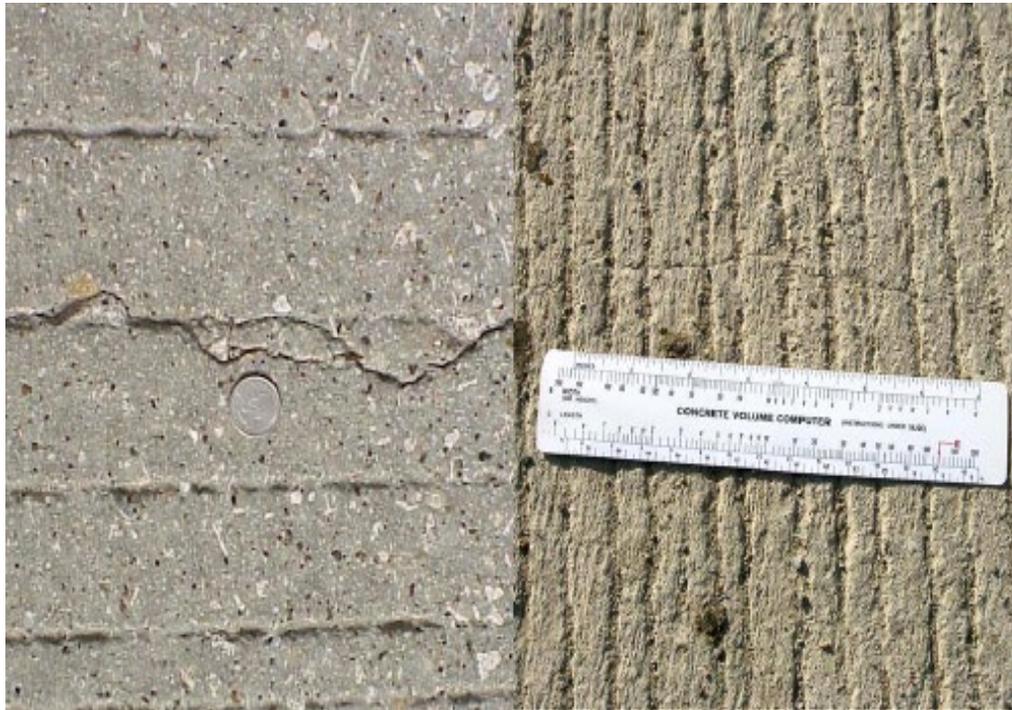


Figure 25: Crack Comparison Between I-90 ¾" Limestone (left) and I-29 1½" Quartzite (right)

Table 9: Crack and Damage Progression on I-90 MRM 55 Section

Feature	7/7/2004	8/12/2004	10/29/2004
Number of Cracks	42	58	95
Crack Spacing	15.96	10.91	6.67
Spalls	0	1	49
Y-cracks	0	1	4

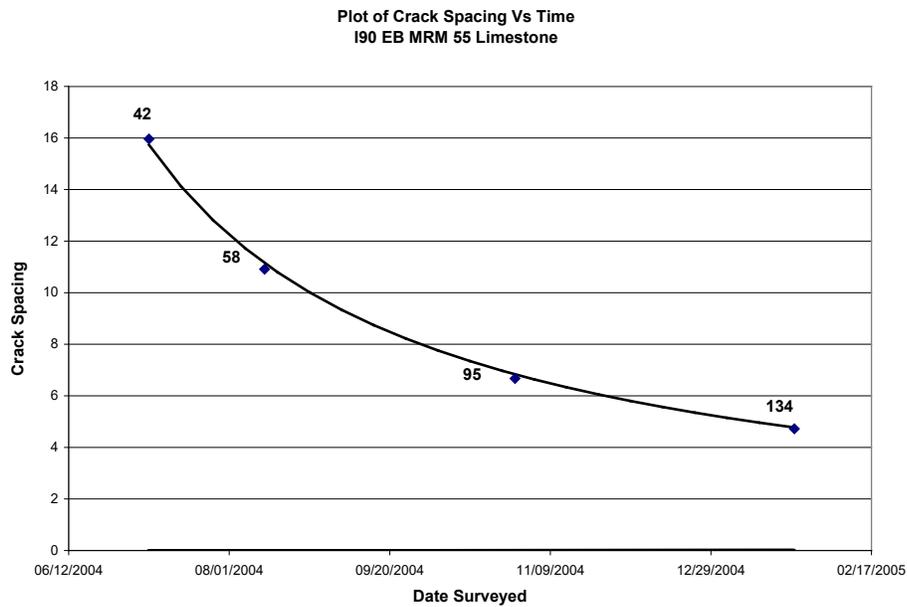
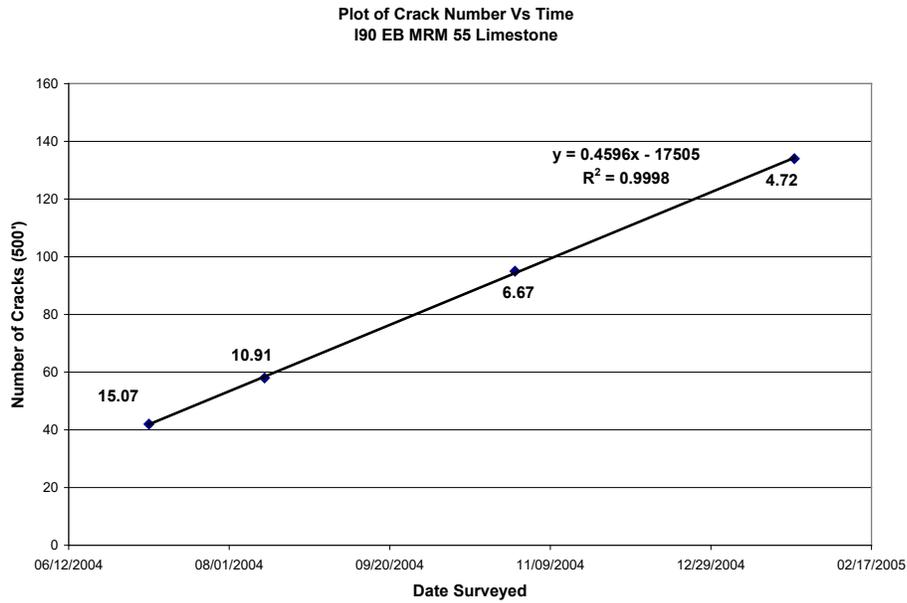


Figure 26: Number of Cracks and Crack Spacing versus Time for Deadwood Avenue

Figure 26 shows plots of crack development with time at I-90 westbound Deadwood Avenue and confirms the prudence of using the number of cracks per 500' for modeling purposes (left plot) instead of the crack spacing as was done with the Mechanistic Empirical Design Guide.

One primary factor which has to be taken into consideration when reviewing existing CRCP design guidelines and practices is the driving force behind many of the recommendations for everything from base type to steel content. The draft mechanistic-empirical design software from the 200X AASHTO Design Guide was used to model the effects of base type and friction coefficient on punchout performance and cracking. The results of a series of analysis for granular bases with frictional coefficients ranging from 2 to 8 are shown in Figure 27. Table 10 below is taken directly from the design guide and illustrates the ranges and averages for friction coefficients for various base types. The modeling parameters characterized the base as fairly erodable but the modeling results clearly show the profound effect of the friction coefficient on projected performance. To obtain a punchout rating of 10/mile (the default failure threshold) at a reasonable age of 40+ years it was necessary to use a friction coefficient of 8 which is twice the high value for granular base shown in the table. Considering the fact that the first two experimental CRCP pavement test sections built in South Dakota are 45 years old and have performed flawlessly with granular bases over their entire lives, the performance basis for the Design Guide model may not truly reflect performance issues for South Dakota CRCP pavements, especially considering the pavement sections used to validate the model were primarily from Illinois and had cement-treated bases. The design guide does require local calibration and adjustment of the model response to actual performance before full implementation.

Table 10: Typical Values of Base/Slab Friction Coefficient Recommended for Design

Subbase/Base type	Friction Coefficient (low – mean – high)
Fine grained soil	0.5 – 1.1 – 2
Sand*	0.5 – 0.8 – 1
Aggregate	0.5 – 2.5 – 4.0
Lime-stabilized clay*	3 – 4.1 – 5.3
ATB	2.5 – 7.5 – 15
CTB	3.5 – 8.9 – 13
Soil cement	6.0 – 7.9 – 23
LCB	3.0 – 8.5 – 20
LCB not cured*	> 36 (higher than LCB cured)

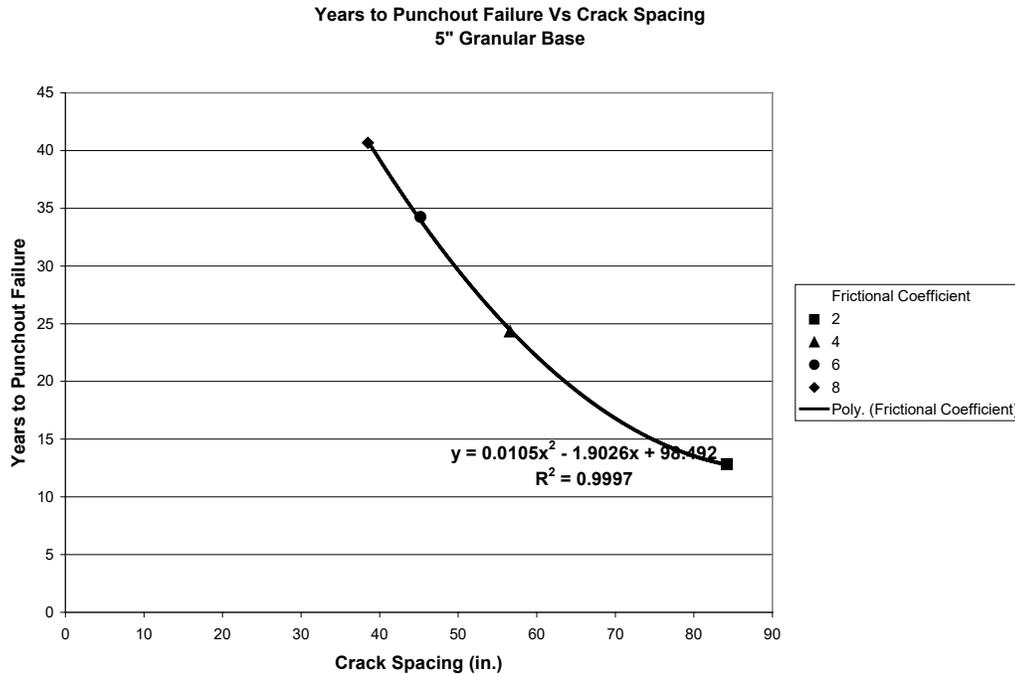


Figure 27: Years to Punchout Failure versus Crack Spacing

Surveys on projects and GPR data indicates that much of the problem with the newer CRCP is manifesting shortly after construction is completed. Initial GPR results from two projects on I-90 near Vivian show a tendency for steel depth to decrease toward centerline and show wandering transverse cracks associated directly with transverse steel. In three sections examined using GPR between 70% and 85% of transverse steel had cracks directly over the steel or within a few inches of it. The steel settlement can be ascribed to concrete placement techniques where the greatest concrete load is placed on centerline before being dispersed by the paving operation. The association of transverse steel with cracks, although not completely unexpected, is a direct indication of cracking occurring at early ages due to stresses inadequately restrained by steel bond or concrete strength as well as later drying shrinkage. Cracking directly over steel is not strongly associated with the temperature gradient-induced cracking that relieves stresses in the pavement. This is also consistent with the wandering nature of the cracks, the tendency for network cracking and wide cracks. Transverse bar spacing was included in the LTPP analysis discussed above for all base types and did not correlate with transverse cracking in any model.

TASK 7: INTERIM REPORT

Submit an interim report summarizing the results of preceding tasks and offering interim recommendations.

An interim report was submitted on April 17, 2005 with reasonable recommendations based on a preliminary assessment of the factors involved for inclusion into construction documents for the 2005 construction season. Recommendations included:

1. CRCP projects for 2006 should be designed with 0.6% steel as a maximum. A shift to higher steel contents can be made next year, if warranted, but the significant improvements apparent from the reduction in steel and the shift to #5 reinforcement with its greater bond surface area argue strongly in favor of staying at this reduced level until further data collection and analysis is completed.
2. The minimum cement content for CRCP pavement concrete should be raised from 440 lbs/yd³ to 500 lbs/yd³ to insure rapid strength development during the critical early stages of crack initiation. This will also provide the concomitant benefit of reducing the likelihood of voids occurring above the steel due to the reduced workability of the concrete mix, a widespread problem during the 2005 construction season and an undesirable problem for long term performance.
3. The project exhibiting the worst performance, I-29 SB in Lincoln County should be treated with one or more corrosion inhibitor/concrete crack repairing chemicals, including Hycrete, during the summer of 2006 and monitored for reduction in crack permeability and corrosion. This may provide a treatment for existing CRCP pavements with unacceptably wide cracks to slow down the impact of deicing chemicals initiating corrosion of the reinforcing steel.

TASK 8: ESTIMATE IMPACT ON PERFORMANCE AND LIFE

Estimate the impact of observed levels of cracking and distress on long-term performance, serviceability, maintenance costs, and pavement life.

Based on the results from the literature review, discussions with other states, project surveys and punchout occurrences and repair costs on existing newer CRCP pavements, judgments of impacts with respect to reduced service life and increased maintenance costs were developed and used to assess expected performance for CRCP pavements with various levels of distress. Initially, the performance curves developed as part of SD97-05 *Statistical Methods for Pavement Management Curve Building & Other Analysis* were used for comparison as a reflection of performance levels achieved in older CRCP pavements. The Pavement Condition Survey results for the various CRCP projects built since 1995 shown in Table 11 were used as inputs for developing the modified performance curves for three types of distress: 1) punchouts, the primary failure mechanism; 2) spalling, a reflection of both the excessive crack width and potential to trap water and deicing salts; and 3) ASR/D-cracking, a rating of excessive and undesirable cracking not truly related to either ASR or D-cracking distress modes but suitable for capturing the level of undesirable cracking. For all three categories, a rating of 5 is nominally ideal. These performance curves are shown in Figures 28, 29 and 30 below.

The results for all three performance estimators are disappointing in terms of the implications for CRCP service life. They indicate that the newer CRCP pavements will suffer a much more rapid rate of deterioration and require maintenance at much earlier ages than the older CRCP did. For instance, the performance curve for punchouts demonstrates an approximately 25% reduction in performance with the onset of significant repair requirements after only 30 years instead of 40. Even more problematical is the impact of deicing chemicals on these newer projects which may not be apparent in terms of performance as of yet. This could mean that the above estimate is extremely optimistic and this is borne out by the

performance curve for spalling. The rate of spall development on the newer projects appears to be accelerated by a factor of 1.8 compared to the old CRCP.

Table 11: Severity Ratings for Old and New Pavements 2006

Interstate	Year	Direction	MRM	Thickness	D Cracking	Spalling	Punchout	Aggregate
90	1996	EB	251	10	4.24	4.99	5	Quartzite
29	1997	SB	97	8	4.98	4.63	4.98	Quartzite
29	1998	NB	97	8	4.88	4.99	4.98	Quartzite
29	1999	NB	85	8	4.98	4.89	5	Quartzite
29	2000	SB	85	8	5	4.71	5	Quartzite
29	2000	SB	61	11	4.55	4.08	5	Quartzite
29	2001	NB	61	11	5	4.57	5	Quartzite
29	2001	SB	27	10	5	4.82	5	Limestone
90	1995	EB	265	9.5	4.22	4.6	5	Quartzite
90	1995	WB	265	9.5	4.4	5	5	Quartzite
90	1997	WB	251	10	4.17	4.77	4.95	Quartzite
90	1997	WB	403	8	4.23	4.87	5	Quartzite
90	1998	EB	403	8	4.24	4.91	5	Quartzite
90	1999	EB	210	10	5	4.79	5	Quartzite
90	1999	WB	210	10	4.97	4.94	5	Quartzite
90	2000	EB	213	10	4.97	4.99	5	Quartzite
90	2000	WB	19	10	5	5	5	Limestone
90	2001	EB	19	10	5	5	5	Limestone
90	2001	WB	213	10	5	5	5	Quartzite
90	2001	WB	353	10	5	4.94	5	Quartzite
90	2002	EB	353	10	4.94	4.93	5	Quartzite
90	2002	WB	334	10.5	5	5	5	Quartzite
90	2003	EB	338	10.5	4.87	4.98	5	Quartzite
90	2004	EB	53	11.5	5	4.75	5	Limestone
Older CRCP Projects								
29	1968	EB	110	8	3.94	3.44	5	Quartzite
29	1968	EB	121	8	3.6	3.61	4.9	Quartzite
29	1968	WB	110	8	3.39	4.01	5	Quartzite
29	1968	WB	121	8	2.81	3.79	4.98	Quartzite
29	1975	EB	151	8	3.27	3.12	5	Quartzite
29	1975	WB	151	8	2.5	4	5	Quartzite
90	1968	EB	133	8	2.98	5	5	Limestone
90	1968	WB	131	8	4	5	5	Limestone
90	1970	EB	189	8	3.46	4.91	5	Limestone
90	1970	WB	189	8	4.06	4.72	4.94	Limestone
90	1972	WB	10	8	3.63	4.73	4.97	Limestone
90	1973	EB	10	8	4.16	4.99	5	Limestone
90	1974	EB	112	8	3.09	5	5	Limestone
90	1974	EB	121	8	3.83	4.8	5	Limestone

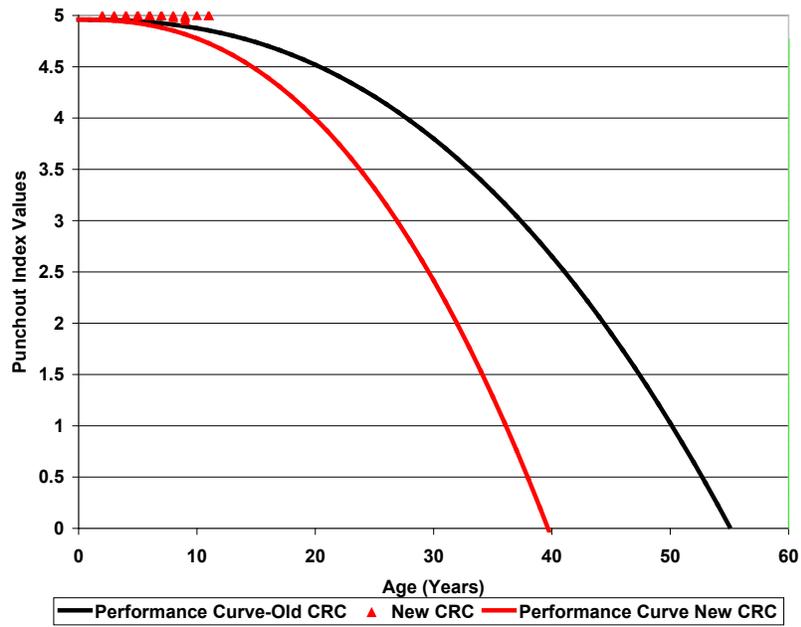


Figure 28: New CRCP Performance Curve Punchouts

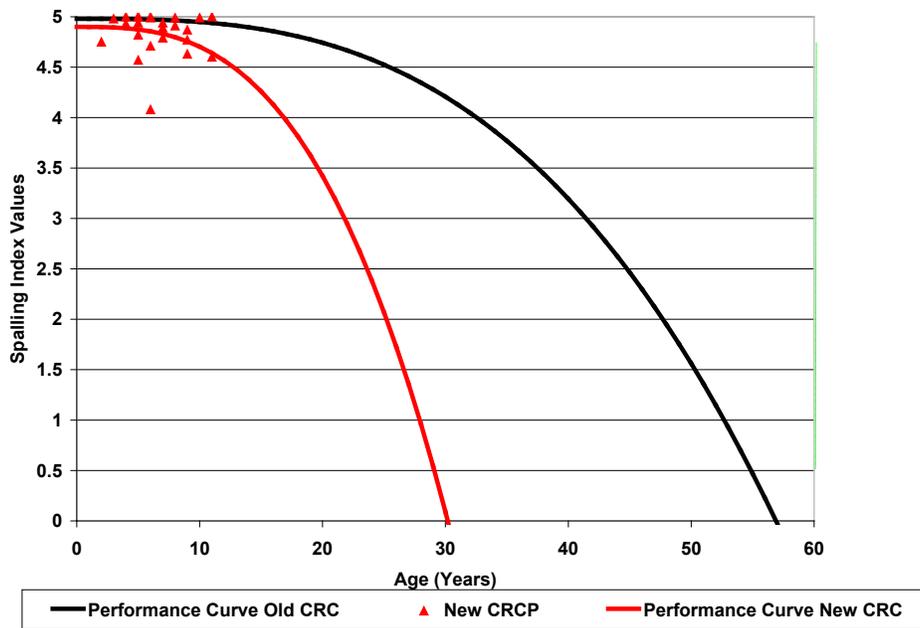


Figure 29: New CRCP Performance Curve Spalling

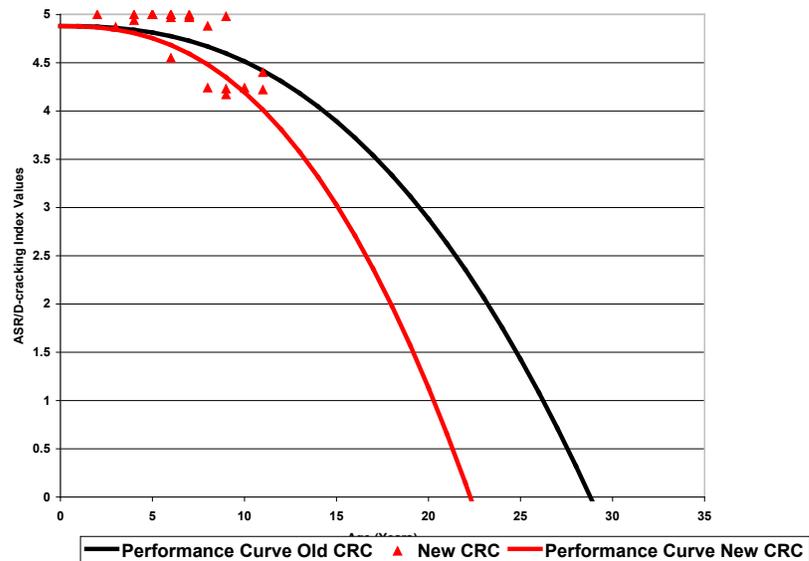


Figure 30: New CRCP Performance Curve ASR/D-Cracking

TASK 9: IDENTIFY CONTRIBUTING FACTORS

Identify factors that potentially contribute to unexpected levels of cracking and distress and, through analysis of design and construction records and observations and measurements of in-service pavements, evaluate their contribution. Include interactions among factors in the analysis.

The literature lists a number of specific factors that have been related to crack spacing, spalling and punchouts. When attempting to assign cause and effect to South Dakota pavement performance using results from other state's research and experience, it is necessary to utilize this information cautiously as the complexity of the interactions between known factors and the sequence of design and construction changes makes assessing relative importance difficult. Initial data indicates that the poor performance of some of our newer CRCP pavements is complicated, to say the least. There is strong evidence of inadequate subbase support on several projects where pumping and some punchouts are evident. Many of the older CRCP pavements were constructed on lime-treated gravel cushions (3"), asphalt-treated base (2") or relatively thick (6-8") granular bases constructed on relatively weak subgrades in many cases. The current design calls for a 5-6" granular base, if reconstructed, which should be sufficient support. The fact that pumping is occurring indicates that there is a significant change in the overall response of the pavement that may have something to do with base aggregate size and construction. The relative performance of the newer CRCP pavements across a spectrum of design, materials and construction variables should allow some clarification of significant contributing factors and how they interact, both positively and negatively.

TASK 10: INTERIM REPORT 2

Submit a second interim report summarizing the results of preceding tasks and offering interim recommendations.

An abbreviated second interim report was submitted in April of 2007, recommending that experimental features including higher cement content and double curing compound application rates be incorporated into the Union County CRCP project discussed in Task 9.

TASK 11: RECOMMEND SPECIFICATION CHANGES

In light of factors contributing to unexpected cracking and distress, recommend changes to design and construction specifications and procedures to minimize their occurrence and severity.

Recommended changes to design and construction specifications as well as construction procedures have been developed throughout the course of the research, evaluated for effectiveness, if possible, through the incorporation of test sections during the 2005, 2006 and 2007 construction seasons and proposed for adoption if they are practical, economically feasible, and there is sufficient evidence from the literature, test section performance or data analysis of existing CRCP project performance to justify incorporating them into future design and construction practices.

Some of the test sections included:

- Optimized gradation 1½” concrete aggregate on all future CRCP projects.
 - IM 29-3(38)79 Madison Street
 - IM-BRF-90-1(106)52 Deadwood Avenue
 - IM 29-6()151 Watertown
- Incorporation of a reduced steel content, modified centerline chair assembly and shallower steel on two projects:
 - IM 29-2(52)72 Tea Interchange
 - IM 90-8(41)344 Hanson County
- Evaluation of environmental factors on one project, already discussed above:
 - IM 90-8(41)344 Hanson County
- Evaluation of cement content and curing compound application rate
 - IM 29 1(74)4 NB

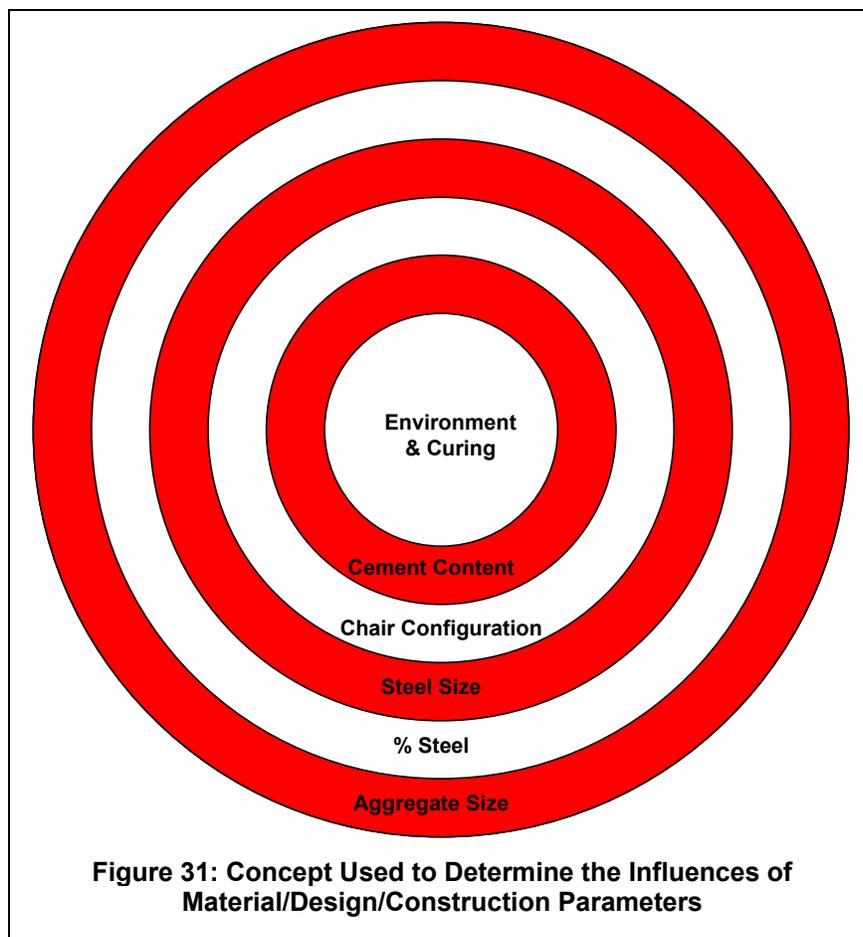


Table 12 illustrates the experimental plan and the design factors being evaluated. The approach used was a systematic modification of material properties and design parameters to evaluate potential benefits in a methodical manner and eliminate, as much as possible, any uncertainties with respect to obtaining clear indications of any benefit. After the larger top size aggregate proved successful at reducing uncontrolled cracking in CRCP pavements incorporating limestone and quartzite coarse aggregate, the 1½” optimized gradation concrete mix design was adopted for all future projects. After the effects of reduced steel content were compared in the Tea I-29 project, a 0.6% steel content was similarly adopted. The Hanson County project, with both increased aggregate size and reduced steel, clearly illustrated the need to minimize environmental effects to insure optimum cracking patterns and was used to develop test sections on the Union County project the following year that provided the final pieces of the puzzle in terms of developing recommendations for all future CRCP construction in South Dakota.

Figure 31 illustrates the approach used and, it must be emphasized, the results are directly applicable for CRCP constructed using South Dakota’s design and materials. For instance, the reduction in steel content from a high of 0.85% to an across the board 0.6% was not a major issue as the older CRCP performed well at this same level of steel content and achieved its expected design life. Traffic levels on South Dakota’s interstate are not excessive and are almost two orders of magnitude less than some of the CRCP pavements in other states. Higher steel contents could be used with granular bases but would require a significant increase in cement content and provisions for curing to achieve satisfactory cracking patterns.

Additionally, an asphalt or cement stabilized base could also be employed to provide increased restraint and control cracking at higher steel contents. The results from these experimental construction projects were used to formulate the changes in design, materials and construction procedures listed in the recommendations.

Many of the design changes incorporated into CRCP projects were based on an analysis of the Long Term Pavement Performance CRCP data as well as survey results from South Dakota CRCP projects discussed earlier. The first item selected for modification, however, was the maximum aggregate size. This decision was based on the known effect of aggregate size on both steel bond strength and concrete fracture resistance. The concrete mixture design for the northbound (NB) lanes of a project being constructed on I-29 in Sioux Falls during the late summer of 2004 was shifted from a maximum coarse aggregate size of 1 in to 1½ in. No other design changes were introduced and the southbound (SB) lanes, constructed during the same year by the same contractor, provided an ideal control section for comparison. The pavement was 12 in thick with a steel content of 0.75% requiring #7 longitudinal steel. The initial results were striking with the NB section exhibiting an average of only 162 cracks per 500 feet (crack spacing 3.09 ft) versus the SB section's more typical average of 228 cracks per 500 feet (crack spacing 2.19 ft) after the first winter. The cracks in the pavement with larger aggregate were much more uniform and tight with much less incidence of cluster, network and Y-cracking. A similar project on I-90 in the westbound (WB) lane near Rapid City with 0.75% steel, #7 bars and a thickness of 11.5 in was constructed with larger aggregate in 2005. The adjacent eastbound (EB) pavement had been built in 2004 by the same contractor using the smaller top size aggregate. The cracking after the first winter was far better in the WB lanes (106) compared to the EB (134). The significantly lower cracking incidence, even in the conventional concrete section, was primarily due to the use of limestone coarse aggregate. As a result, larger aggregate concrete mixtures were adopted for all future CRCP projects.

The next factor to be considered was the steel percentage (% steel). Direct comparison of the impact of both steel content and depth was achieved by incorporating parallel test sections into an interchange project just south of Sioux Falls where two experimental and two control sections were built. All sections were constructed with larger aggregate using the same concrete mixture design. The NB lanes on either side of the interchange bridge were designed with 0.54 and 0.6% steel. The SB lanes contained the standard 0.7% steel recommended for the 11 in pavement. In addition, nominal steel depth was specified at 3½ in and 4 in for the two experimental and control sections. Again, the results were striking with the test sections exhibiting excellent cracking behavior with the cracks uniform and tight and no undesirable cracking after the first winter. The standard sections, on the other hand, although they exhibited improved cracking over prior CRCP projects due to the larger aggregate, had significantly wider and more frequent cracks. The test sections had 78 (0.54% steel, 3½ in depth) and 87 (0.6% steel, 4 in depth) cracks per 500 feet compared to the control sections which had 188 (3½ in depth) and 140 (4 in depth) cracks per 500 feet. Since the CRCP built in South Dakota during the late 1960's and early 1970's had a steel content of 0.6%, this content was provisionally adopted for all future CRCP construction unless further results indicated the need for a higher steel content. The older CRCP had shown exemplary performance in spite of insufficient thickness and steel content based on current CRCP design parameters and should be sufficient for the traffic levels encountered on Interstate highways in South Dakota. The design and material parameters as well as cracking performance are shown in Table 12.

Table 12: Union County Test Sections

Section	Cement/Fly Ash Content (lbs./yd ³)	Curing Rate (gal/125 ft ²)	Cracks (#/500 ft)	Type of Cracking
Control	461/115	1	140	Random, wandering
Double Cure	461/115	2	145	Tight, uniform
High Strength	510/112	1	126	Tight, uniform
High Strength Double Cure	510/112	2	108	Tight, uniform

A project on I-90 in Hanson County in eastern South Dakota, built in 2005 and discussed previously, provided an excellent illustration of the complexity and interactions occurring with regard to the cracking patterns which develop in CRCP. This project incorporated both the larger aggregate and reduced steel content modifications as well as a portion with smaller aggregate due to constructability issues. The range in cracking exhibited on the project was directly due to thermal effects and aggregate size and was correlated with concrete temperature, water/cement ratio, aggregate size, wind velocity and the temperature difference between concrete and ambient temperatures. The variations in crack widths and the propensity for undesirable cracking in some portions of the project prompted the inclusion of several experimental sections in a Union County project constructed in 2006. The primary variables included on this project were cement content and curing compound application rate. A series of 1 mile sections were paved sequentially incorporating the parameters shown in Table 13. The results indicated the importance of cement content (early strength development) and adequate curing (minimum thermal stresses) on cracking performance. An additional project, built later in 2006 by the same contractor employed a double cure throughout and did not exhibit any undesirable cracking. This outcome can be ascribed to the use of the double cure and the fact that the project was paved during the fall when cooler temperatures prevailed. The project did not incorporate higher cement content.

TASK 12: RECOMMEND MAINTENANCE AND REHABILITATION STRATEGIES

Recommend cost-effective maintenance and rehabilitation strategies to maintain and extend the life of in-service CRCP pavements exhibiting unexpected levels of cracking and distress.

The primary focus of this effort was directed at the possible application of corrosion inhibitors in solution to CRCP pavements with wide cracks and heavy winter deicing. One product, Hycrete, was identified as a potential inhibitor for topical application as an aqueous or alcohol solution and a test section on the shoulder was applied at MRM 22 westbound on a CRCP section near Whitewood in June, 2005. The material proved too slippery for practical use on pavements as the inhibitor is essentially a soap with all the issues that entails. Also, initial research on this product was producing mixed results at the time so a decision was made to defer further investigation into this task by recommending a separate research project be developed at the completion of this research.

TASK 13: RECOMMEND MONITORING STRATEGY

Recommend a strategy for monitoring the condition of in-service CRCP pavements to better discern their condition and remaining life.

The existing road service indices were used for projecting future CRCP performance and appear to be adequate to track the decline of serviceability with time for the newer CRCP without introducing new,

more time consuming evaluation procedures. All that would be required would be the separation of pavement performance estimates into older and newer CRCP families instead of using the combined population for determining any changes in the performance projections over time. In fact, given the limited remaining life of the original CRCP pavements in South Dakota, all the projections could be based on the pavements built since 1995 though data would still be collected on the older CRCP pavements until they are rebuilt or overlaid to allow prioritization for maintenance and reconstruction purposes.

Table 13: Experimental Construction Projects and Factors

Factor	Project	Year	Lane	Concrete Aggregate	Slab Thickness	% Steel	Bar Size	Steel Depth	Base Thickness	Chairs	Cracking	Effects
	IM 29-3(38)79 Madison Street	2004	NB	Quartzite Optimized 1½"	12"	0.75%	#7	4"	5"	Normal	162	Agg Size
			SB	Quartzite Normal ¾"	12"	0.75%	#7	4"	5"	Normal	228	Control
	IM-BRF-90-1(106)52 Deadwood Avenue	2005	WB	Limestone Optimized 1½"	11.5"	0.75%	#7	4"	Variable	Normal	106	Agg Size
		2004	EB	Limestone Normal ¾"	11.5"	0.75%	#7	4"	Variable	Normal	134	Control
	IM 29-2(52)72 Tea Interchange	2005	NB North	Quartzite Optimized 1½"	11"	0.60%	#5	4"	5"	Normal	87	Agg Size % Steel Bar Size
			NB South			0.54%	#5	3½"	5"	Normal	78	Agg Size % Steel Bar Size & Depth
			SB North			0.70%	#7	3½"	5"	Normal	188	Agg Size
			SB South			0.70%	#7	4"	5"	Normal	140	Agg Size
	IM 90-8(43)344 Hanson County	2004	WB	Quartzite Normal ¾"	10.5"	0.70%	#6	3¾"	5"	Normal	212	Control
	IM 90-8(41)344 Hanson County	2005	EB	Quartzite Optimized 1½"	10.5"	0.60%	#5	3½"	5"	Special	212	Agg Size % Steel Bar Size & Depth
	IM 29-6(21)151 Watertown	2006	NB	Quartzite Optimized 1½"	10"	TBD	TBD	TBD	Variable TBD	Special	91	Agg Size Base
	IM 29-6()151 Watertown	2004	SB	Quartzite Optimized ¾"	10"	0.70%	#6		10"	Normal	262	Optimized

The idea of using Pathview images for monitoring the newer CRCP was not explored do to delays in the technology being fully implemented. The results of the projections using existing performance indices were sufficiently consistent to preclude the use of this approach until such time as it supersedes current pavement evaluation procedures.

TASK 14: PREPARE FINAL REPORT

Prepare a final report and executive summary of the research methodology, findings, conclusions, and recommendations.

A final report and executive summary detailing the results of the literature review, research methodology, findings, conclusions, and recommendations was submitted at the conclusion of the project.

TASK 15: MAKE EXECUTIVE PRESENTATION

Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

A PowerPoint presentation will be presented to the SDDOT Research Review Board.

SUMMARY AND CONCLUSIONS

The primary goal of this research effort, an attempt to optimize South Dakota's CRCP design, materials and construction methods, was essentially successful. Given the economic constraints involved, the focus of the research was not to adopt design details used successfully by other states such as asphalt and cement stabilized bases but to minimize uncontrolled cracking in new CRCP constructed on a granular base with the least increase in cost

The first requirement for recommending specification changes to alleviate undesirable cracking was to understand the mechanisms responsible for initiating the cracking. The cracking was clearly demonstrated to occur at very early ages, effectively beginning immediately after the concrete had hardened and proceeding much more rapidly than is consistent with desired behavior. Although the numerous design changes made to our CRCP since 1995 were all in agreement with accepted design parameters, the combination of materials changes coupled to the increased steel contents and pavement thicknesses employed resulted in high levels of stress in the pavements at very early ages. The increasing steel contents could only be accommodated by increasing reinforcement size which had the unfortunate side effect of decreasing steel surface area and early bond strength while increasing stress. The use of modified Class F fly ash in the concrete, at first at a rate of 18.75% substituted for 15% cement and later at a straight 20% substitution, was prompted by concerns with Alkali-Silica Reactivity (ASR) but had the concomitant effect of slowing down strength development in the concrete at exactly the point where rapid strength gain was critical to generate uniform, tight transverse cracking in the pavement.

This research was directed at modifying design and material parameters for optimizing the transverse cracking patterns developed on CRCP in South Dakota. These parameters were examined in both existing pavements and in new construction to methodically determine what changes were necessary to yield the most acceptable crack spacing and width while minimizing undesirable cracking patterns such as cluster, network and Y-cracking. That being said, the results obtained apply directly to CRCP built on granular or unbonded base, not on stabilized bases. The degree of restraint provided by the base used in CRCP has a direct influence on how much cracking occurs, its spacing and how long it takes to develop.

CRCP built on granular base develops significant cracking almost immediately and is therefore subject to crack-inducing stresses while the concrete-steel bond is inadequately developed and the concrete itself has insufficient strength to resist these stresses, especially when fly ash is used in the concrete. This can also contribute to undesirable crack spalling and extremely wide cracks. Increasing coarse aggregate size and decreasing steel content had beneficial effects on cracking due to the increase in early bond strength and the greater fracture resistance of the concrete with bigger rock. Decreasing steel percentage and thus size actually increased the bond area available while reducing the concrete-steel stresses due to the smaller bar diameter used. The purpose of these changes was not directly aimed at increasing the crack spacing on new CRCP as tight, uniform cracks spaced far enough apart as dictated by the aggregate used will provide excellent performance. Instead the intent was to slow down the process of crack propagation so that cracks formed in a more controlled manner over a longer period of time with a minimum of spalling due to unbalanced early stresses. The final modifications to design parameters, material requirements and construction procedure modifications insure that any future CRCP pavements built in

South Dakota using granular base and minimal cement content will perform as effectively as those originally constructed almost forty years ago.

The results of this research indicate just how complicated CRCP cracking can be and how much the thermal environment can affect cracking, even when an optimal design is used. Every effort needs to be made to minimize thermal, moisture and shrinkage stresses during the early stages of hydration so that cracking is not random, uncontrolled and threatens the long term performance of the CRCP, especially where deicing chemicals are employed for winter maintenance. CRCP is probably the most superlative portland cement concrete pavement that exists but it becomes apparent that the one size fits all changes in CRCP design that have developed over the last decades may not always be appropriate. The changes in CRCP design parameters employed over the course of rebuilding the Interstate in South Dakota beginning in 1995 were based on recommended design standards. Unfortunately, these standards were inadequate with respect to insuring pavements with acceptable cracking behavior. Modifications to the design and the materials used in CRCP construction should be optimized for the environment the pavement will be built and expected to perform in.

RECOMMENDATIONS

Recommendations developed as a result of this research include the following:

1. Our current nominal steel depth, ranging from 3¼” for 8” thick CRCP to 4” for pavements ≥ 11 ” should be modified to 3¾” for all pavements ≥ 9 ” in thickness. This will insure minimal crack widths without affecting performance.
2. A maximum cross sectional steel content of 0.6% utilizing #5 reinforcement should be adopted for all new CRCP pavements, regardless of thickness or traffic.
3. Optimized concrete mixture designs with at least 10% of the coarse aggregate retained on the 1” sieve should be employed for all new CRCP construction. This approach should also be investigated for jointed plain concrete pavements as well as it should help in reducing curling and warping in these slabs at early ages.
4. The modified chair support design should be adopted for all new CRCP pavements to minimize steel subsidence due to additional loading during paving operations.
5. Curing compound should be applied to the pavement within 30 minutes at a minimum rate of 1.5 gallons/125 ft². Project Engineers should insure that curing compound application is uniform and meets the maximum time and minimum coverage rate requirements. This approach should also be investigated for jointed plain concrete pavements as well as it should help in reducing curling and warping in these slabs at early ages.
6. When concrete temperatures are anticipated to be in excess of 80°F the following day and air temperatures are sufficiently high to result in hot aggregate in the stockpiles, the stockpiles should be wetted down at the conclusion of daily production to provide evaporative cooling of the stockpiles overnight.
7. The grade and steel should be moistened directly ahead of the paving operation to cool the steel and minimize water loss from the concrete due to dry grade.
8. Cement content should be increased to 510 lbs/yd³ with modified Class F fly ash addition at 112 lbs/yd³ for all new CRCP projects to minimize the likelihood of uncontrolled cracking due to slow strength development.
9. Pavement Management should continue to collect performance data on CRCP pavements using the same parameters as in the past but should develop and maintain performance curves incorporating data from the newer CRCP pavements built since 1995 and excluding older CRCP sections to more accurately track CRCP serviceability and potential maintenance requirements.
10. A separate research project should be developed to examine methods for mitigating the deterioration of our newer CRCP pavements, including an investigation of whether the topical application of a corrosion inhibitor could be effective. This effort should also include a reassessment of the levels of deterioration of CRCP projects based on the recent manifestation of severe longitudinal cracking on a significant number of projects that was not apparent at the time of the original distress surveys done in 2004-2005.

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