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# Performance of Continuously Reinforced Concrete Pavements, Volume VII—Summary

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Turner-Fairbank Highway Research Center  
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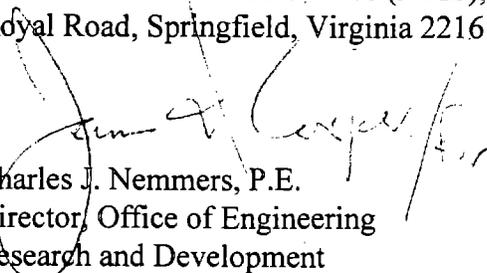


## FOREWORD

This report is one volume of a seven-volume set presenting the results of a study to provide the state-of-the-art for the design, construction, maintenance and rehabilitation of Continuously Reinforced Concrete Pavements (CRCP). Through a thorough literature review of current and past research work in CRCP and extensive field, and laboratory testing of 23 in-service CRC pavements, the effectiveness of various design and construction features were assessed; performance of CRCP was evaluated; and procedures for improving CRC pavement technology were recommended. The 23 test pavements were located in six states that participated in this national pooled fund study. In addition, the data available for 83 CRCPs included in the General Pavement Study (GPS) Number V of the Long Term Pavement Performance (LTPP) Program was presented and analyzed. A number of CRCP maintenance and rehabilitation techniques have been used over the years, including joint and crack sealing, cathodic protection of reinforcing bars, full-depth patching, resurfacing, etc., were also evaluated. This report will be of interest to engineers and researchers concerned with the state-of-the-art design, construction, maintenance and rehabilitation of CRCP, including predictive models. The study was made possible with the financial support of Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, Texas, and Wisconsin.

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<b>16. Abstract</b> This report is one of a series of reports prepared as part of a recent study sponsored by the Federal Highway Administration (FHWA) aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of continuously reinforced concrete (CRC) pavements. The scope of work of the FHWA study included the following: <ol style="list-style-type: none"> <li>1. Conduct of a literature review and preparation of an annotated bibliography on CRC pavements and CRC overlays.</li> <li>2. Conduct of a field investigation and laboratory testing related to 23 existing in-service pavement sections. This was done to evaluate the effect of various design features on CRC pavement performance, to identify any design or construction related problems, and to recommend procedures to improve CRC pavement technology.</li> <li>3. Evaluation of the effectiveness of various maintenance and rehabilitation strategies for CRC pavements.</li> <li>4. Preparation of a Summary Report on the current state of the practice for CRC pavements.</li> </ol> Each of the above four items is addressed in a separate report. The following reports have been prepared under this study: <ul style="list-style-type: none"> <li>Performance of CRC Pavements                         <ul style="list-style-type: none"> <li>Volume I - Summary of Practice and Annotated Bibliography</li> <li>Volume II - Field Investigation of CRC Pavements</li> <li>Volume III - Analysis and Evaluation of Field Test Data</li> <li>Volume IV - Resurfacings for CRC Pavements</li> <li>Volume V - Maintenance and Repair of CRC Pavements</li> <li>Volume VI - CRC Pavement Design, Construction, and Performance</li> <li>Volume VII - Summary</li> </ul> </li> </ul> This report is Volume VII in the series.		<b>14. Sponsoring Agency Code</b>	
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# SI\* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS FROM SI UNITS

APPROXIMATE CONVERSIONS TO SI UNITS		APPROXIMATE CONVERSIONS FROM SI UNITS						
Symbol	When You Know	Multiply By	To Find	Symbol	When You Know	Multiply By	To Find	Symbol
<b>LENGTH</b>								
in	inches	25.4	millimeters	mm	millimeters	0.039	inches	in
ft	feet	0.305	meters	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	meters	1.09	yards	yd
mi	miles	1.61	kilometers	km	kilometers	0.621	miles	mi
<b>AREA</b>								
in <sup>2</sup>	square inches	645.2	square millimeters	mm <sup>2</sup>	square millimeters	0.0016	square inches	in <sup>2</sup>
ft <sup>2</sup>	square feet	0.093	square meters	m <sup>2</sup>	square meters	10.764	square feet	ft <sup>2</sup>
yd <sup>2</sup>	square yards	0.836	square meters	m <sup>2</sup>	square meters	1.195	square yards	yd <sup>2</sup>
ac	acres	0.405	hectares	ha	hectares	2.47	acres	ac
mi <sup>2</sup>	square miles	2.59	square kilometers	km <sup>2</sup>	square kilometers	0.386	square miles	mi <sup>2</sup>
<b>VOLUME</b>								
fl oz	fluid ounces	29.57	milliliters	mL	milliliters	0.034	fluid ounces	fl oz
gal	gallons	3.785	liters	L	liters	0.264	gallons	gal
ft <sup>3</sup>	cubic feet	0.028	cubic meters	m <sup>3</sup>	cubic meters	35.71	cubic feet	ft <sup>3</sup>
yd <sup>3</sup>	cubic yards	0.765	cubic meters	m <sup>3</sup>	cubic meters	1.307	cubic yards	yd <sup>3</sup>
<b>MASS</b>								
oz	ounces	28.35	grams	g	grams	0.035	ounces	oz
lb	pounds	0.454	kilograms	kg	kilograms	2.202	pounds	lb
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
<b>TEMPERATURE (exact)</b>								
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
<b>ILLUMINATION</b>								
fc	foot-candles	10.76	lux	lx	lux	0.0929	foot-candles	fc
fl	foot-Lamberts	3.426	candela/m <sup>2</sup>	cd/m <sup>2</sup>	candela/m <sup>2</sup>	0.2919	foot-Lamberts	fl
<b>FORCE and PRESSURE or STRESS</b>								
lbf	poundforce	4.45	newtons	N	newtons	0.225	poundforce	lbf
lbf/in <sup>2</sup>	poundforce per square inch	6.89	kilopascals	kPa	kilopascals	0.145	poundforce per square inch	lbf/in <sup>2</sup>

NOTE: Volumes greater than 1000 l shall be shown in m<sup>3</sup>.

\* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised September 1993)

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## INTRODUCTION

Continuously reinforced concrete (CRC) pavement is a portland cement concrete pavement type containing continuous longitudinal reinforcement that consists of a pattern of regular occurring transverse cracks that effectively serve as contraction joints. CRC pavements are designed to develop naturally a pattern of cracks at certain transverse intervals. The concept behind the use of CRC pavement is the “so-let-it-crack” philosophy rather than avoiding cracks at long intervals or controlling cracks to optimum intervals. The alternative is often the use of jointed concrete pavement systems intended to control cracking at longer spacing and reduced longitudinal steel requirements. Whether a joint occurs as a random or a controlled crack, its intended purpose is to possess high load transfer and stress relief characteristics that minimize the potential for fault or spall development and eventual loss of ride quality. Due to the features of this characteristic crack pattern, CRC pavements maintain a relatively low state of stress under load and consequently can provide excellent levels of performance, but they typically have been accepted as having greater initial cost than jointed concrete pavement (JCP) systems.

Most of the cracking pattern in CRC pavement forms shortly after construction. However, many cracks may not become evident at the pavement surface for several months after construction. Transverse cracks in a properly designed and constructed CRC pavement are not objectionable to motorists since many of them are not noticeable from inside a moving vehicle. The cracks are held tightly by the continuous reinforcement to ensure the integrity of the aggregate interlock and load transfer at the crack face. The riding quality of CRC pavements is typically very good and will remain high as long as the structural continuity of the transverse cracks remains high. Several aspects related to the design, construction, and evaluation of CRC pavements have been suggested and addressed in the previous volumes of this report. This summary volume is to serve as a synthesis of CRC pavement performance, design, and rehabilitation.

### **Factors Associated With Quality Performance of CRC Pavements**

Key to the quality performance of CRC pavement is the provision for non-erodible, uniform support and the prevention of either widened transverse cracks or punchouts or both. Each of these distress types is indelibly associated with the characteristics of the crack pattern and the quality of the support system. Prior to discussion of them, the factors associated with the development of the cracking pattern will be briefly described. Initial cracking in CRC pavements is primarily due to stresses that are the result of restraint to temperature and moisture variation within the slab that tend to induce both horizontal and vertical displacements. Field observations suggest that the cracks induced by this restraint form within the first 3 to 7 days after placement of the concrete. Typically, 80 percent of these cracks are induced by drying shrinkage in the concrete.

The performance of CRC pavement has been studied over a period of several years in the form of special field sections, performance studies, and construction projects that have contained a wide range of design and construction variables. Documented evidence of these studies has provided a wealth of knowledge leading toward improved techniques and

practices for CRC pavement design and construction. This section provides a summary of the key factors critical to the performance of CRC pavements. As previously alluded to, these factors may be categorized as follows:

- Development of uniform crack patterns.
- Provision for erosion resistant, drainable, uniform support conditions.

A key element to the development of a uniform crack pattern is balance between the following factors, listed in their order of importance:

- Concrete drying shrinkage.
- Ratio of steel bond area to concrete volume ( $Q$  value =  $4p/d_b$ ).
- Concrete aggregate/paste bond strength.
- Season of construction.
- Concrete thermal coefficient of expansion.
- Subbase friction characteristics.

Other factors related to the formation of the crack pattern in CRC pavements but not discussed in detail are: concrete strength, concrete modulus of elasticity, concrete creep characteristics, lapping of reinforcing bars, slab thickness, shoulder type, rebar corrosion, deicing chemicals, and traffic.

As discussed later, the crack pattern indirectly affects crack width and steel stresses which affect the degree of load transfer provided at the transverse cracks. Ultimately, the average and distribution of resulting crack widths control the life and the quality of CRC pavement performance. Since the factors noted above affect the resulting crack widths, it is important to discuss each of these factors relative to how they influence the development of the crack pattern and a balance that should be maintained between them.

### Concrete Drying Shrinkage

Loss of moisture is a characteristic of concrete that is related to the environmental conditions at the time of construction and the porosity of the concrete matrix. Loss of moisture can affect concrete not only in terms of induced strain, referred to as drying shrinkage, but also its strength. Drying shrinkage depends to a great extent on the amount of moisture loss and the quality of curing and on the water cement-ratio used to place the concrete pavement. The amount of drying shrinkage typically ranges from 400 to 600  $\mu\epsilon$  but may be greater, particularly under hot weather paving conditions. Drying shrinkage is the primary contributor to the development of cracking and must be controlled such that it is not too high or reduced to where it is too small. Although the amount of drying shrinkage that concrete will ultimately develop is difficult to predict, if evaporation rates are too low then a potential for long crack patterns exists. If the evaporation is too high then short crack patterns and reduced concrete strengths can result. It is evident that the minimum and maximum shrinkage limits need to be established for optimum crack development and performance of CRC pavement systems.

### Ratio of Steel Bond Area to Concrete Volume (Q)

The reinforcement in CRC pavement causes a restraining effect to contraction strain that increases as the percentage of steel increases and, relative to the Q value ( $4p/d_b$ ), is one of the major factors affecting crack development. Decreased crack spacing is associated with increased steel percentages.<sup>(1,2)</sup> U.S. experience has indicated that steel percentages of 0.55 to 0.70 have provided suitable cracking patterns and performance in CRC pavement systems. However, European experience has generally indicated steel percentages 15 to 20 percent greater than those that are used in the United States. The percentage of longitudinal steel also affects the crack width, which in turn influences aggregate interlock, load transfer, and stiffness at the transverse cracks. Field observations, in addition to theoretical design theories, verify the fact that crack width in CRC pavements decreases with increase in percentage of longitudinal reinforcement.<sup>(2)</sup> Practically speaking, this effect apparently has an upper limit associated with it since it has been noted that the average crack interval does not significantly decrease with steel amounts above 1 percent while average cracking intervals increase dramatically with steel amounts below 0.4 percent. Season of placement and construction weather may override the effect of steel reinforcement on crack spacing. Key elements in the development of the crack pattern are steel design (p and Q) and weather conditions at the time of construction.

Stress development in CRC pavements occurs through the transfer of stress from steel to the concrete at and in the vicinity of the transverse cracks where the bond-slip regions exist. The stress transfer from the longitudinal steel to the concrete is related to the reinforcing steel surface area and the characteristics of the rebar pattern. For the same percent of longitudinal steel, the smaller size bar provides a larger steel surface area, which in turn increases stress transfer from the steel to the concrete. Smaller bar sizes are associated with smaller crack spacing. McCullough and Ledbetter<sup>(3)</sup> found that the crack spacing was inversely proportional to the ratio of the bond area to concrete volume. Analysis and experience have indicated that rebar bond area to volume of concrete (Q) will affect the crack spacing in CRC pavement and that the parameter Q is related to the time of year of construction. As a result, minimum Q values of 0.03 for summer construction and 0.04 for fall or winter conditions are recommended. Although no guidelines are available, it is suggested that these factors be increased 10 percent for epoxy-coated reinforcement. It is pointed out, however, that one study indicated epoxy-coated reinforcement has little effect on CRC pavement crack patterns.<sup>(4)</sup> Based on the equation for Q, it is evident how the value of Q can be held constant for various combinations of steel percentage and the diameter of the reinforcing steel. The relationship of these rebar parameters suggests their sensitivity to crack pattern development. Studies have documented this sensitivity, which can also be related to transverse crack widths. However, coarse aggregate and effects due to construction weather, as noted above, may significantly influence this sensitivity.

An aspect related to the area of bond of steel is the use of two layers of longitudinal steel. The position of the top layer of steel has been shown to be significant in past studies, and the use of two-layer placements has been adopted in TxDOT construction standards<sup>(5)</sup> for pavements thicker than 330 cm (13 in) in order to maintain optimum steel bond area to

concrete volume ratios. Thicker pavements may experience a greater degree of volumetric restraint because of a reduced depth of cover caused by the use of two layers of reinforcing steel. Two layers of reinforcing steel also requires two layers of transverse steel, which tends to cause a weakened plane in terms of transverse crack induction. A high incidence of transverse cracking coincidental with the position of the transverse steel was noted on projects in Texas<sup>(6)</sup> that used two layers of reinforcing steel where the transverse bars in each layer were vertically aligned.

### Concrete Aggregate/Paste Early-Age Bond Strength

The strength of the bond between the coarse aggregate and the paste at an early-age is critical to the development of the crack pattern in a CRC pavement system. The early bond strength of the coarse aggregate is primarily mechanical in nature; however, a physical adhesion may play some role in the overall strength of the bond. From a mechanical standpoint, crushed aggregates typically manifest greater bond strengths at an early-age than rounded aggregates. In terms of cracking patterns, smooth river gravel coarse aggregate concrete mixtures typically develop cracking patterns of greater crack density and at closer cracking intervals than those made with crushed coarse aggregates. Because of the difference in bond strength of the coarse aggregate and the difference in cracking behavior, the percentage of reinforcement should be reflected in the overall design of the pavement system as a function of the type of coarse aggregate used in the concrete. River gravel concrete should require less steel reinforcement than a CRC pavement made with limestone concrete. Typically, the design steel percentage is based on the thermal characteristics of the concrete at a mature age although it is the bond behavior of the coarse aggregate controlling the cracking behavior at an early-age. The thermal effect of the coarse aggregate plays a role in the development of the crack pattern after creep effects of the concrete are diminished due to ageing and maturing of the concrete.

### Season and Time of Construction

The magnitude of a change in temperature will affect crack pattern development in CRC pavements in primarily two ways: (1) due to thermal gradients in the slab and (2) due to uniform changes in temperature at a given point in the slab. Naturally, as geographic location changes, the climate to which the concrete is exposed also changes. Temperature ranges (highest annual temperature minus lowest annual temperature) can be as large as 65.5°C (150°F), depending on the location and climate. Historical temperature records may also be used to establish the temperature difference for design; however, normal temperature ranges may not be as severe as indicated by such records. Maximum and minimum yearly temperatures are used because they correlate with the evolution of average crack spacing and in determination of the amount of linear slab movement and prediction of crack width at the transverse crack. However, recent studies in Texas<sup>(6)</sup> have indicated that such correlations may not be applicable with respect to prediction of early-aged crack widths.

Concrete strength gain rates due to environmental conditions during fall and winter construction periods are lowest when the prevailing temperatures are typically the lowest. Therefore, concrete placed in the fall may have less time to develop sufficient concrete strength before maximum cracking stress occurs than concrete placed in the spring or summer. Concrete pavement placed in the fall may develop a shorter crack spacing than pavement placed in the spring because of the relatively lower concrete strengths caused by typically lower ambient temperatures. However, this effect may be somewhat offset because the reference temperature (on which the concrete stresses are based) is also lower in comparison with construction periods at hotter times of the year. CRC pavements, particularly those placed with limestone coarse aggregates, constructed under cool weather conditions develop longer crack spacing but smaller crack widths than those placed in the summer months under warm weather conditions. Because of the greater drying shrinkage under hot weather conditions, CRC pavement performance may be significantly affected due to the effect the seasonal conditions have on the resulting crack widths.

The cracking process in CRC pavement involves crack development at an early-age and at later ages. It is important to point out that most cracks initiate at an early-age but may not become evident at the surface for several months or years. The time of day that concrete is placed also has an effect on CRC pavement cracking behavior. Concrete placed in the morning typically sets at a higher temperature and consequently develops greater stresses and more early-aged cracking than concrete placed in the afternoon. The effect is that concrete placed in the morning has shorter crack spacings than concrete placed in the afternoon. Large temperature drops and moisture loss are conducive to rapid crack development. This can occur under summer weather and windy conditions where concrete pavement is placed in the morning hours, leading to maximum setting temperatures and stresses that can cause cracking as early as the next day or later (2 to 3 days) depending on the type of coarse aggregate used in the concrete. Concrete placed in the afternoon typically displays a delayed early-aged cracking because of a buildup of drying shrinkage, in combination with temperature change effects, sufficient to cause cracking. Equations that indicate the percentage of steel required to hold shrinkage and temperature cracks intact to prevent yielding of the steel were presented in volume 6 of this report.

Another factor that affects the development of cracking in CRC pavement is the curing methods used during the paving process. The degree of early-aged cracking may be related to some extent to how concrete is cured. It is generally accepted that the more the water loss from the concrete mixture during the hardening process the greater will be the shrinkage and the lower the degree of hydration. Therefore, concrete shrinkage stress will have a greater potential to exceed the concrete strength, inducing early-aged cracks in the CRC pavements. Curing of CRC pavements is a crucial step in controlling early cracking potential. Common methods for curing concrete pavements are: (1) membrane curing compound; (2) polyethylene film curing; and (3) cotton mat curing. The research conducted by Zollinger et al.<sup>(6)</sup> revealed that both cotton mats and polyethylene film reduced daily temperature variation and reduced moisture loss from the pavement surface. Accordingly, the numbers of surface cracks in pavements that develop initially with cotton

mat or polyethylene curing are much lower than those in pavements cured with membrane compound.

### Coefficient of Thermal Expansion

Thermal coefficient of expansion of concrete is a parameter that is related to the volumetric change hardened concrete will undergo due to a temperature change. As previously pointed out, although the thermal-related behavior of concrete does not contribute to the development of the crack pattern to the same extent as drying shrinkage or possibly the strength of the aggregate/paste bond, it is used, mostly as a matter of convenience, in design to predict the average cracking behavior of CRC pavement. The two main constituents of concrete, cement paste and coarse aggregate, although dissimilar thermal coefficients, combine in terms of a composite coefficient of expansion of concrete. Since more than half of the concrete volume is coarse aggregate, the biggest factor influencing the coefficient of thermal expansion of concrete appears to be the type of coarse aggregate. As the siliceous gravel content decreases the thermal coefficient value decreases. A study conducted by Brown<sup>(7,5)</sup> showed that the effect of silica content in the aggregate on the thermal coefficient of the concrete is very significant. The higher the silica content, the higher the thermal coefficient. Thermal effects are greatly manifested in the daily variation in the opening and closing of the transverse cracks. This opening and closing contributes to the stress in the reinforcing steel but only to the extent that the thermal coefficient of the steel reinforcement is greater than the coefficient of expansion of the concrete. Consequently, this effect is often much less than the effect due to drying shrinkage. However, the opening and closing of the cracks is a factor in performance since the degree of load transfer is directly related to the width of the cracks. Therefore, crack widths should be restricted to certain limits.

### Subbase Friction Characteristics

The direction of frictional resistance provided by the subbase is opposite to that of concrete displacement. Subbase friction depends on the subbase material type. As concrete contracts, the subbase friction and the steel resist the concrete displacement, thereby increasing the level of concrete tensile stress, which contributes to the resultant crack spacing. The resistance to the concrete contraction through bond stress and subbase friction causes the concrete tensile stress to build up and the concrete displacement to be reduced. If the resultant concrete stress exceeds the concrete tensile strength, a crack will develop.

### **Widened Transverse Cracks**

One of the frequently occurring distresses in CRC pavements is in the form of excessively widened transverse cracks. The development of this distress type is often associated with reinforcing steel that has been exposed to severe corrosion that has resulted in a reduced cross-section of the steel. The reduced cross-section of the steel results in a localized reduced tensile capacity of the pavement section. The reduced capacity leads to yielding or complete rupturing of the reinforcing steel, which allows the transverse cracks

to open excessively. Once the distress has advanced to this stage, faulting of the crack can readily, and often does, develop causing the pavement surface to become rough.

In terms of a mechanism of distress development, widened transverse cracks are typically associated with widely spaced transverse cracks (typically 3 to 4 m or greater) that inherently will manifest a wider range of crack opening and greater steel stresses in the vicinity of the crack than cracks spaced at closer intervals. The wider crack widths allow greater access of moisture over time and temperature cycles (and deicer salts) that eventually provides an avenue for moisture to reach the level of the reinforcement and initiate corrosion. Over time and temperature cycles, corrosion progresses at the wider cracks that causes a reduction in the cross-section of the reinforcement and ultimately causes failure of the steel to occur.

An effective method to prevent the development of widened transverse cracks is to prevent widely spaced or clustered crack patterns. This can be achieved to some extent through maintaining an appropriate balance between the design Q value of the steel, the bond strength of the coarse aggregate, the method of curing, and the construction weather at the time of placement of the concrete. Design Q values and coarse aggregate type should be selected relative to each other in light of the expected season of construction. Elaborating further, limestone coarse aggregate CRC pavement placed in the fall construction season may require greater Q values than the same CRC pavement placed in the summer construction season. Concretes consisting of coarse aggregates with lower bond strengths will manifest closer cracking intervals unless this characteristic is offset by more effective methods of curing that would be expected to result in a greater average cracking spacing. Although improved methods of curing will minimize the loss of moisture due to evaporation during the hardening stages of the concrete, excessive restriction of the loss of moisture and drying shrinkage may result in low stress development and wider than expected cracking intervals for some coarse aggregate and Q value combinations.

The discussion above is based on the assumption that the subbase material does not create such a high level of friction that it significantly affects the development of the cracking pattern. As will be discussed subsequently, subbase materials must provide certain support related functions but should not be strongly bonded to the bottom of the concrete slab. A variety of materials are available for subbases under CRC pavement systems, all of which manifest different degrees of friction with the concrete slab depending on the characteristics of the material. Subbases under CRC pavements should provide flexible, erosion resistance support and provide a certain amount of drainability. Drainability can be achieved by grading the subbase aggregates to provide sufficient permeability and void space in the aggregates but must be utilized in such a manner without causing or creating an interlocking with the bottom of the concrete slab. For this purpose, many CRC pavement systems include an asphaltic interlayer that serves to break the bond between the subbase and the concrete slab. In this fashion, the effect of the subbase on crack development can be minimized yet meet the support requirements for CRC pavements.

## **Punchout Distress**

The second form of distress is the loss of load transfer on adjacent transverse cracks, leading to the development of a punchout—the greatest concern of designers of CRC pavements. The punchout process is associated with load transfer mechanisms inherent to the behavior of CRC pavement. Certainly a widened crack results in a significant decrease in load transfer but punchout distress is always associated with aggregate interlock wearout and the loss of load transfer on two adjacent, closely spaced cracks. The focus of identified failure modes of the punchout process is consequently closely aligned with the load transfer, crack width, and the effective slab bending stiffness of adjacent transverse cracks characteristic to CRC pavement, as discussed below. Detailed field and laboratory study<sup>(1)</sup> has clearly indicated that punchouts are initiated as a result of lost or reduced pavement support rather than as a result of ruptured steel reinforcement, as commonly heretofore assumed. Relative to punchout formation, rupturing of the steel reinforcement does not (if it does at all) occur until well into the final stages of the punchout process and, consequently, is only an artifact of the loss of support, load transfer, and pavement stiffness. As previously noted, steel rupturing is a factor primarily in cases of widened transverse cracks where advanced corrosion has severely reduced the cross-sectional area of the reinforcement. The ruptured steel in this instance results in cases of widened transverse cracks that leads to faulting of the transverse crack where punchouts frequently occur in the absence of widened transverse cracks.

### Basic Failure Modes Leading to Punchout Distress

Punchout development in CRC pavement systems is closely tied to the degree of support provided in the pavement structure. Although punchouts are recognized as the primary form of distress in the performance of CRC pavements, CRC pavements in the 200 to 230 mm (8 to 9 in) thickness range have performed very well (with no punchouts) sustaining several million ESALs. Even though performance of this level of traffic can be achieved with good design practice and adequate crack widths, it is still important to consider the mechanisms associated with this form of distress.

Four failure modes relating to punchout distress have been identified (and verified in this study) on the basis of field observations<sup>(1)</sup> as the fundamental failure mechanism in CRC pavements developing punchout distress. The development of these failure modes is based a priori on uniform support conditions. The failure modes are illustrated in figure 1 in typical developmental sequence. The first three modes of failure are associated with factors contributing to the loss of load transfer across the transverse crack. Mode I focuses on concrete fracturing associated with the reinforcing steel at the crack face. Cracking with this form is due to reinforcing bar pullout from the surrounding concrete. Fracturing of this nature has been noted in concrete pullout tests<sup>(8,9)</sup> and develops in the concrete at a steel stress range of 96.5 to 124.1 kPa (14 to 18 ksi). Field measurements of steel strains at the crack face indicate that this range of stress is frequently exceeded in the colder months of the year. Cyclic bond stresses in the concrete induced from environmental factors can result

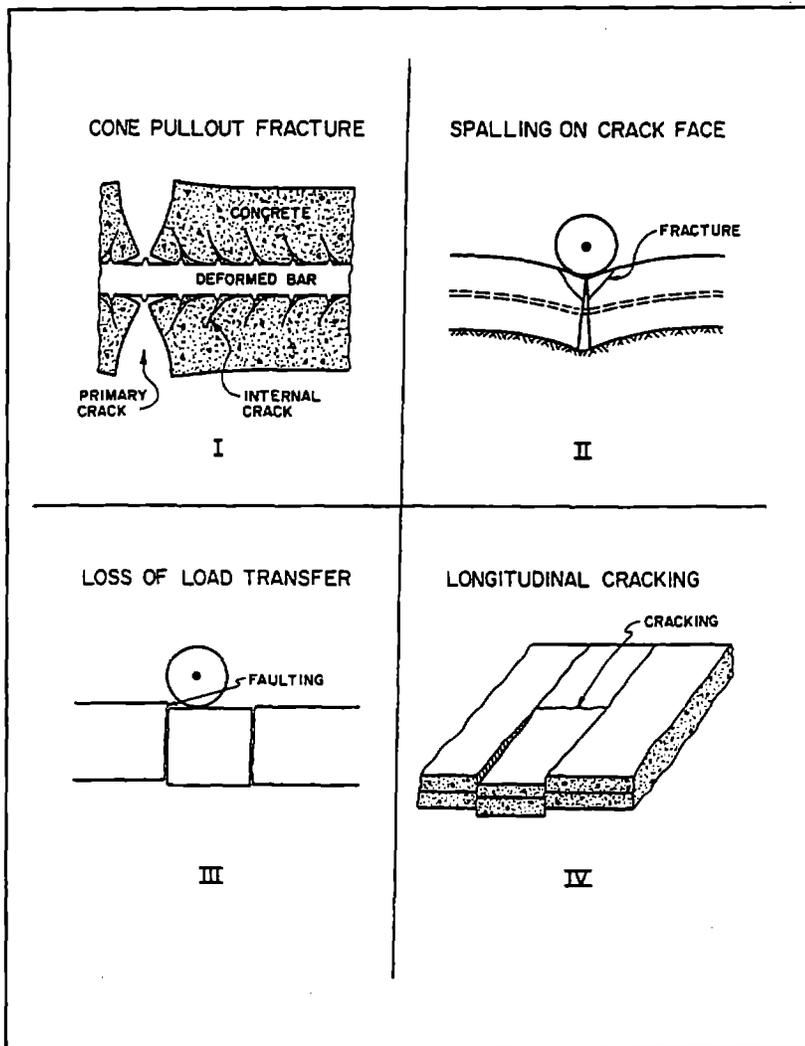


Figure 1. Failure modes related to punchout distress in CRC pavement.<sup>(1)</sup>

in a crack growth process, noted in the field study, around the reinforcing bar — effectively destroying the load transfer capability of the bar as a void develops. Additionally, a loss of bond stiffness<sup>(10)</sup> and pavement bending stiffness occurs. Bearing failure or rebar looseness can also lead to a void around the reinforcement and can have a detrimental effect on the pavement performance similar to what the pullout fracture does. Pullout failure may be difficult to avoid since the threshold stress is frequently exceeded. In any case, the load transfer contribution of the reinforcing steel (relatively small bearing areas and small diameters) should be ignored in design.<sup>(1)</sup> This emphasizes the importance of crack width on pavement stiffness and performance.

Mode II failure, spalling of the transverse crack, affects the pavement stiffness at the transverse crack. Because of the development of voids around the reinforcing steel described above, pavement stiffness is significantly reduced. As pointed out below with regard to Mode III failure, a reduction in pavement stiffness at the cracks may also develop due a gradual loss of aggregate interlock and load transfer efficiency.<sup>(1)</sup> The pavement stiffness cycles between high and low, mostly as a function of the temperature and the concomitant opening and closing of the transverse cracks. The reduced stiffness behavior, which occurs on a daily basis, can be assumed to predominate during the winter season. Reduced pavement stiffness is not only a function of the crack width<sup>(11)</sup> but also of the position of the reinforcing steel<sup>(12)</sup> among other factors discussed in earlier volumes of this research. The narrower the transverse cracks the stiffer the overall pavement system, which in turn lowers spalling related stresses. This mode of failure is a visual sign of progressive punchout development.<sup>(1)</sup>

Failure Mode III, shown in figure 1, is a loss of load transfer along transverse cracks due to wear out of the aggregate interlock. Since the reinforcing steel provides little load transfer, the load transfer of the crack is solely a function of the crack width. Given a constant crack width, the load transfer will decrease under repetitive loading. Loss of support due to erosion plays a major role in accelerated wear out of the aggregate interlock along a transverse crack.

The final mode of failure, Mode IV, is related to bending stresses in the transverse direction. These stresses typically are not significant in CRC pavement so long as there is a high load transfer across the cracks (prior to spalling), a high quality of support, or the crack spacing is greater than 1.2 m (4 ft).<sup>(1, 13)</sup> The process relative to CRC pavement design can be optimized with respect to crack spacing and crack width. Obviously, an erosion resistant subbase system is required to ensure quality performance for CRC pavements. This normally requires that stabilized subbases consist of approximately 8 percent cement. As previously pointed out, AC interlayers provide the optimal combination of bond and friction to develop desirable crack patterns in CRC pavement. Excessive bonding of the slab to cement stabilized subbases often results in poor crack patterns and wide crack widths.

### **Improved CRC Pavement Design Concepts**

Present design methodology for CRC pavements focuses on the determination of reinforcing steel requirements relative to achieving an average crack spacing for a given set of temperature conditions. However, slab thickness determinations are only remotely related to the criteria used for the steel design or for transverse crack spacing requirements. In this sense, improvements are warranted in the design of CRC pavement systems particularly in terms of maintaining a balance, previously referred to, among all of the factors that control the CRC pavement performance relative to slab thickness requirements. Early thickness design for CRC pavement was based on the premise that CRC thicknesses did not need to be as great as jointed concrete pavement thicknesses due to a certain equivalence in structural capacity (approximately 70 to 80 percent). Past and present thickness design procedures consider several factors associated with the prediction of the average crack spacing and steel stress due to contraction restraint. Crack prediction methods included in these procedures are based on environmental stresses and material thermal properties of the concrete and steel. The design crack spacing is limited to certain criteria to minimize the potential of punchout distress, which indirectly allows the design engineer to arrive at a design thickness. On the basis of the factors that control punchout performance indicated previously, it is apparent the thickness design of CRC pavement should consider more directly the effect of the width of the transverse crack and its associated load transfer characteristics relative to the associated mechanism of punchout development. Although very important to the performance of CRC pavement, present design methodology currently ignores crack width requirements as far as they pertain to the degree of load transfer afforded by a transverse crack. Therefore, a relationship between pavement thickness, load transfer, crack width, and the percentage of reinforcement for a given crack spacing is needed for design purposes. Underlying such a relationship is the

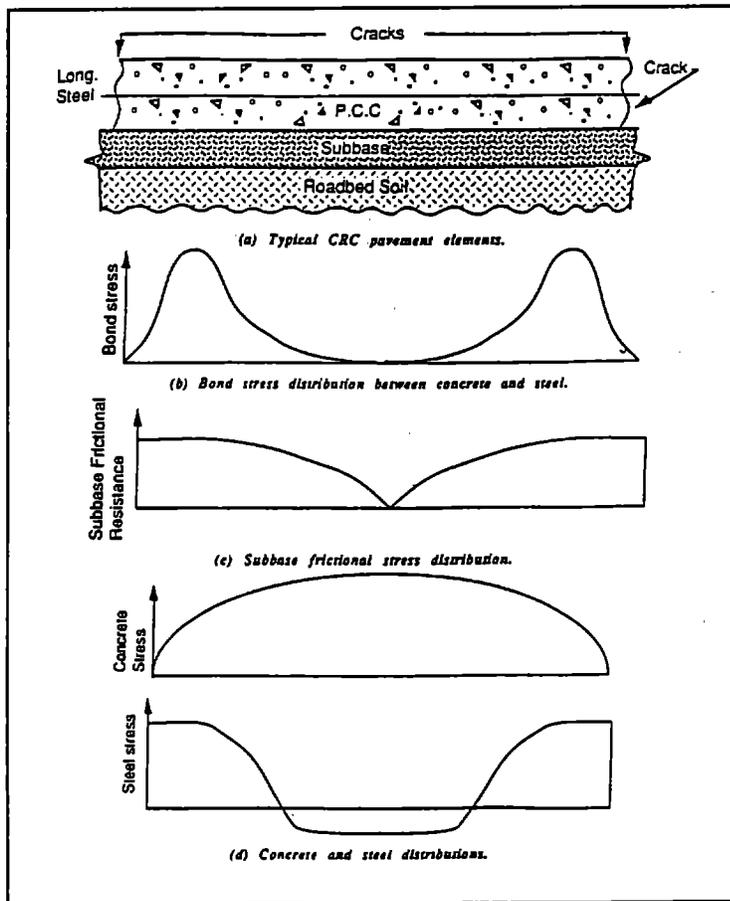


Figure 2. CRC pavement elements and distributions of various stresses.<sup>(5)</sup>

fact that the control of the transverse crack width is the key to achieving good performance of CRC pavement as facilitated through uniformly and optimally spaced transverse cracks.

The design of CRC pavement can be divided into the consideration of environmental and wheel load related cracking. Crack development due to environmental stresses may be thought of in two phases: initial crack development and secondary crack development. Initial cracking, which occurs rapidly, should be nearly equal to or less than  $4.4 \ell$  where  $\ell$  is the radius of relative stiffness of the pavement surface layer. The crack pattern tends to stabilize after the development of secondary cracking, which is largely a function of the factors discussed above.

During the placement of CRC pavements, the concrete is typically subjected to non-uniform/non-linear (from top to bottom) volumetric changes that result in stress development due to temperature, moisture, and shrinkage effects. The resulting stresses caused by these effects are relieved by the formation of secondary transverse cracks. Figure 2 (a) shows a typical CRC pavement section between two adjacent transverse cracks.<sup>(5)</sup> When the pavement experiences a change in temperature or a change in drying shrinkage, the concrete movement in the longitudinal direction is restrained by the longitudinal steel and subbase friction.

The reinforcing steel that is embedded in the concrete behaves, stress and strain-wise, in a different manner than the concrete. This behavior results in interfacial shear stress (referred to as bond stress) at the interface between the steel bar surface and the concrete. The magnitude of the bond stress depends on the concrete strength and mechanical shape of the bearing face of the ribs on the longitudinal bar. These factors have been the subject of recent improvements in the design of reinforcing steel rib patterns.<sup>(14)</sup> Because of the anchor and lug characteristics of the reinforcing steel that promote a strong bond between

the concrete and the embedded steel, a bond stress will develop. Figure 2 (b) shows a typical bond stress distribution between concrete and steel<sup>(5)</sup> over a segment of cracked CRC pavement.

The direction of frictional resistance provided by the subbase is opposite to that of concrete displacement. Subbase friction depends on the subbase material type; when the concrete contracts, the subbase friction and the steel resist the concrete displacement, thereby increasing the level of concrete tensile stress, which contributes to the resultant crack spacing. Figure 2 (c) shows a typical distribution of frictional resistance.<sup>(5)</sup> The resistance to the concrete contraction through bond stress and subbase friction causes the concrete tensile stress to build up and the concrete displacement to be reduced. Figure 2(d) illustrates the concrete and steel stress distribution along the CRC pavement slab.<sup>(5)</sup> If the resultant concrete stress exceeds the concrete tensile strength, a crack will develop. Past performance data have indicated that a dense graded asphaltic concrete (AC) interlayer provides the most desirable subbase frictional characteristics. Although not shown in figure 2, it is good design practice to incorporate an AC interlayer between the CRC layer and the subbase—particularly where stabilized bases are used. CRC pavements that perform well have non-erodible support conditions while maintaining minimal bonding conditions. Open-graded, permeable bases, in combination with AC interlayers, have also provided non-erodible support while maintaining minimal subbase friction.

### Environmentally Induced Transverse Cracking

As noted above, several factors have been identified that affect how cracks form in CRC pavements. Initial cracking in CRC pavements may be due to environmentally induced temperature and moisture gradients related to the slab  $\ell$ -value and its curling and warping behavior. Field observations of initial or primary cracks suggest that these cracks form within the first 3 to 7 days after placement of the concrete. Secondary cracks form due to the continuity of reinforcement (i.e., internal restraint), which inhibits free movement of the concrete after the formation of primary cracks. Stresses that develop at this stage are referred to as restraint stresses. According to data recently obtained in Texas,<sup>(15)</sup> primary cracks constitute the rapidly evolving crack pattern at intervals of approximately  $4.4\ell$  (radius of relative stiffness) or less, which then form the beginning secondary crack intervals and lead to the development of a stable cracking pattern.

A significant contribution was made by Vetter,<sup>(16)</sup> who developed relationships for crack spacing in reinforced concrete illustrated in stress diagrams for drying shrinkage and temperature drop, shown in figures 3 and 4 ( $L$  is the crack spacing and  $u$  is the bond stress). After the formation of the first crack due to restrained shrinkage, a new state of equilibrium and strain compatibility develops. The restrained shrinkage is accommodated by the crack, by the bond slip, and by the uncracked concrete. The following equations for the average crack spacing are derived from Vetter's basic assumptions,<sup>(16)</sup> namely, secondary cracks form within the initial crack interval. A formula for the average crack spacing due to shrinkage is shown below:

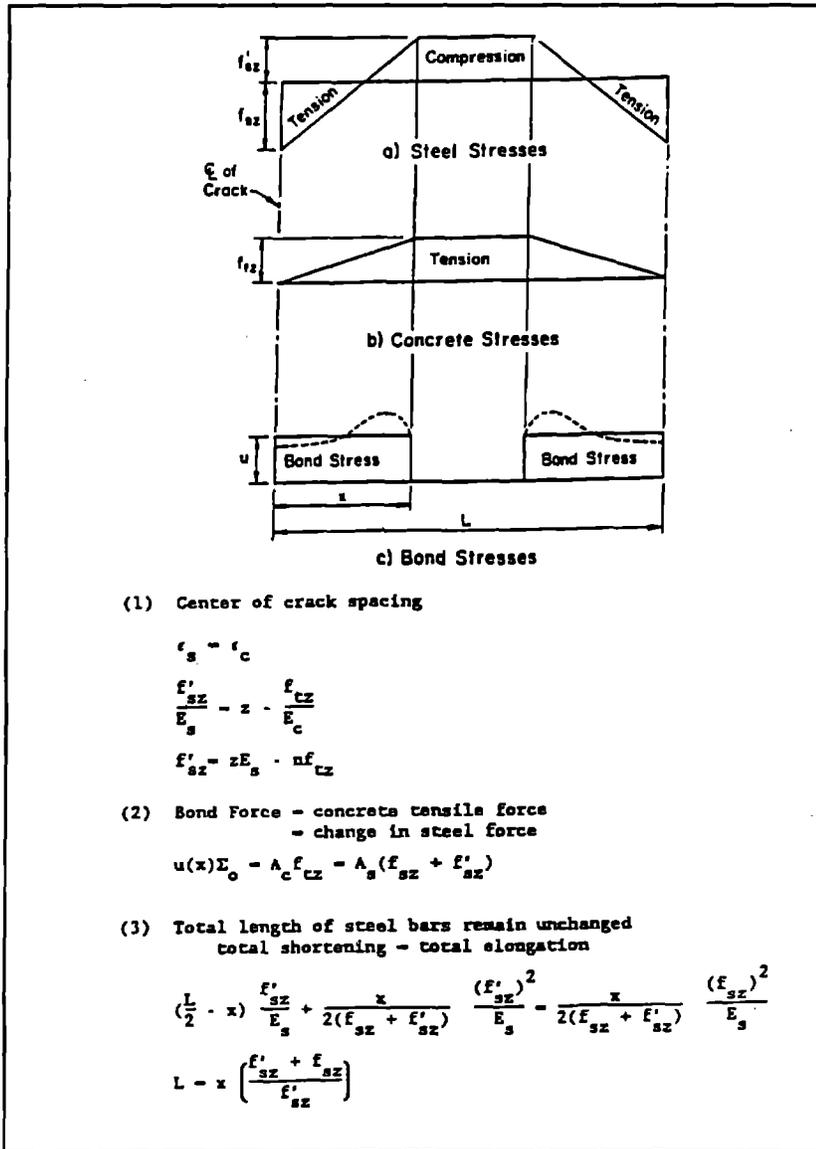


Figure 3. Stress distribution between cracks of CRC member subject to shrinkage.<sup>(16)</sup>

$\alpha_s$  = coefficient of thermal expansion of steel  
 $t_m$  = temperature drop on the surface of the pavement (°F)

A formula for the average crack spacing when both shrinkage and temperature drop occur simultaneously is later derived<sup>(16)</sup> by considering the combined stress diagram for steel and the concrete that is expressed in a simplified form as:

$$L = f_t^2 / \{Q \cdot u \cdot p (E_s \alpha_s t_m + z \cdot E_s - n \cdot f_t)\} \quad (3)$$

$$L = f_{tz}^2 / \{Q \cdot n \cdot p \cdot u (z \cdot E_c - f_{tz})\} \quad (1)$$

where

$L$  = crack spacing (L)  
 $f_{tz}$  = concrete tension stress due to shrinkage strain at the center of crack (F/L<sup>2</sup>)  
 $Q$  = ratio of bond area to concrete volume  
 $u$  = average bond stress (F/L<sup>2</sup>)  
 $p$  = percent reinforcement  
 $n$  = modular ratio ( $E_s/E_c$ )  
 $E_c$  = elastic modulus of concrete (F/L<sup>2</sup>)  
 $z$  = drying shrinkage

A formula for the average crack spacing is also derived for temperature drop in a similar manner:

$$L = f_{t\phi}^2 / \{Q \cdot n \cdot p \cdot u (\alpha_s t_m E_c - f_{t\phi})\} \quad (2)$$

where

$f_{t\phi}$  = concrete tension stress due to temperature drop at the center of the crack spacing (F/L<sup>2</sup>)

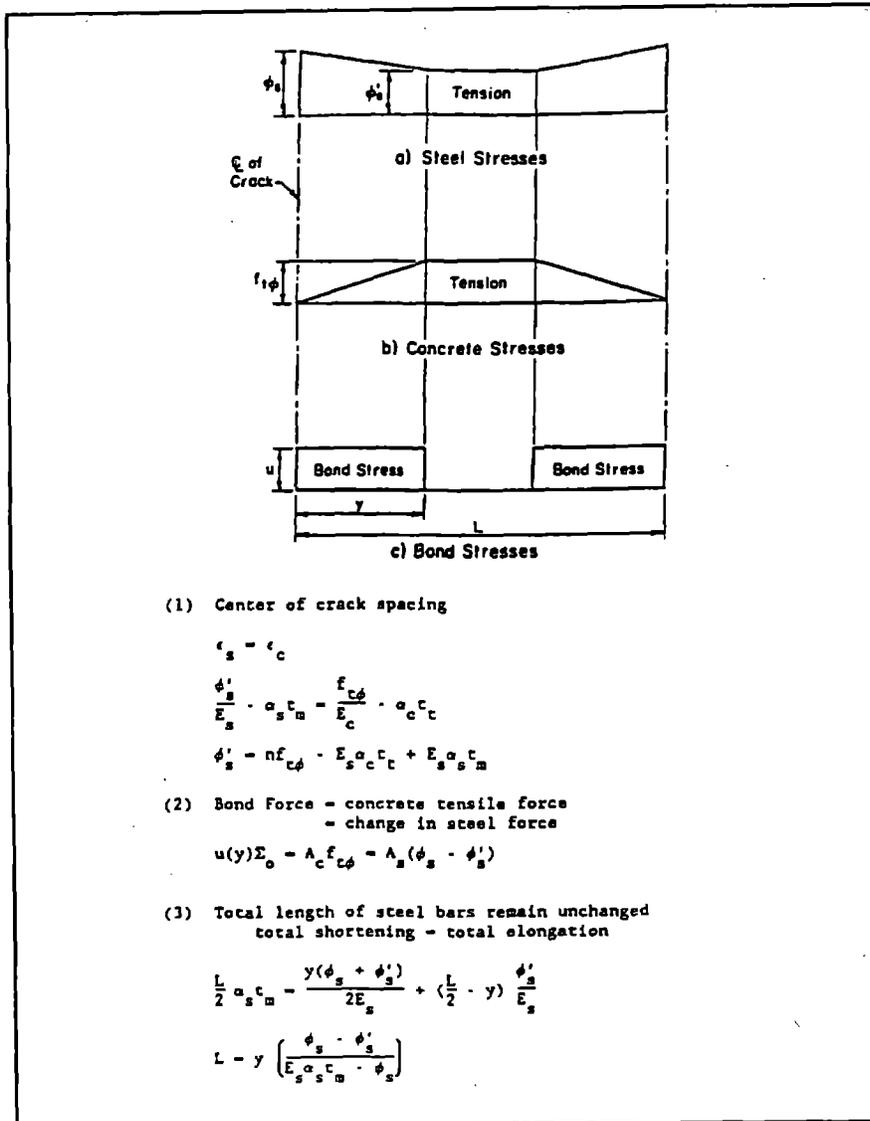


Figure 4. Stress distribution between cracks of CRC member subject to temperature drop.<sup>(16)</sup>

longitudinal reinforcement expressed as the ratio of area of steel reinforcement to the area of concrete ( $A_s/A_c$ ). The percentage of steel reinforcement has been listed as one of the most significant factors affecting crack spacing (as included in the Q factor). As previously noted, many CRC pavements in the United States contain reinforcement in a range of 0.5 to 0.7 percent. In some northern regions percentages in the higher end of the range have been used. However, it is reemphasized that environmentally induced cracking is largely controlled by the construction weather conditions, which can many times dominate how the reinforcement interacts with the concrete pavement cracking behavior. Many punchout distresses may be a function of crack width in which crack spacing is considered to have a major influence on crack width. Although the principal purpose of the reinforcement is to maintain tight crack spacing and good aggregate interlock, little information is available as to the actual role the reinforcement plays in the load transfer developed in CRC pavement.

where

$f_t$  = total tension stress in concrete (which for CRC pavement analysis is assumed to be equal to the tensile strength of concrete)

All the other terms are as defined in equations 1 and 2. Equation 3 indicates a close crack spacing may be obtained by a high bond stress. The same effect can also be obtained through increasing the percentage of reinforcement or using smaller diameter bars.

### CRC Pavement Reinforcement Considerations

A major factor in the crack development of CRC pavement is the percentage of

### *Design Percentage of Reinforcement*

Most theoretical relationships for the determination of reinforcement are based on the yield strength of the steel ( $f_y$ ). Vetter<sup>(16)</sup> originally developed two expressions for the percentage of reinforcement ( $p = A_s/A_c$ ) in reinforced concrete under fully restrained conditions for volumetric changes. One expression is in terms of drying shrinkage and the other is in terms of temperature drop, respectively:

$$p = f_t / (f_y + zE_s - nf_y) \quad (4)$$

and

$$p = f_t / (f_y - nf_y) \quad (5)$$

where

$$\begin{aligned} f_t &= \text{concrete tensile strength} \\ f_y &= \text{yield strength of steel reinforcement} \end{aligned}$$

The above equations were developed for unbonded or low friction subbase interfaces. They have been modified (using a multiplication factor of  $1.3 - 0.2\mu$  where  $\mu$  is the coefficient of friction) to account for coefficients of subbase friction other than a coefficient of friction of 1.5, which was apparently associated with unbonded subbase conditions. The advantage of using the multiplication factor is to increase the percentage of reinforcement under longer crack spacings that may result from lower values of friction coefficients. However, the effect of subbase friction on the design percentage of steel is rather insignificant and experience has indicated that the percentages predicted by these expressions are suitable for friction coefficients up to 3.0. On this premise, a multiplication factor of  $1.3 - 0.1\mu$  may be more appropriate. Since it is recommended that subbase interfaces with friction coefficients greater than 3.0 be avoided, any further adjustments to the design percentage of steel based on subbase friction is not warranted.

Vetter rationalized that the above expressions formed the basis for minimum reinforcement. He showed that the maximum shrinkage that can be sustained by the concrete without cracks forming (ignoring creep) is  $z = S_t/E_c$ , where  $S_t$  is the tensile strength of the concrete; upon substitution in the first of the two above equations, the sum of the last two terms of the denominator is zero and  $p$  becomes equal to  $S_t/f_y$ . Equation 5 represents the minimum limit of steel requirements if the shrinkage is zero and the temperature drop ( $T$ ) does not exceed a critical amount in which the total bond development length is greater than the crack spacing. In such case:

$$p = S_t / 2(f_y - T\alpha_s E_s) \quad (6)$$

Under minimum steel conditions ( $f_t = S_t$ ), equation 6 is equivalent to equation 5. Equation 6 only pertains to the case where the crack spacing is less than or equal to two times the bond development length. The percentage of steel calculated by equation 6 assumes the steel to be at the elastic limit and gives results greater than those determined by equation 5 as long as the steel stress is below the yield strength at a crack spacing of twice the bond

development length or less. Consequently, equation 6 is not frequently applicable since these conditions (crack spacing and temperature drop combined) are rarely met.

The role of the stress in the concrete and the reinforcement is demonstrated in the above equations in which the amount of reinforcement is minimized if yielding of the reinforcement occurs. Equations 4 and 5 are also useful in determining the level of stress in the reinforcement (at the crack) given the percentage of reinforcement (p):

$$f_s = f_t(1/p + n) - zE_s$$

The stress equation for temperature drop independent of shrinkage is found by dropping out the shrinkage term. A Vetter type equation can be developed for a combination of shrinkage contraction and temperature drop by accounting for a difference in thermal coefficients for the concrete and the steel reinforcement (shown previously):

$$f_s = f_t(1/p + n) + E_s\{(t_m\alpha_s - t_c\alpha_c) - z\}$$

where

$t_m$  = temperature drop at mid-depth of slab

$t_c$  = temperature drop

Past American Association of State and Highway Transportation Officials (AASHTO) design guides have recommended a limiting stress criterion for the reinforcement of 75 percent of the ultimate tensile strength of the steel.<sup>(17)</sup> Based on stress predictions of in-service CRC pavements that have shown good performance, it was concluded that yielding of the steel occurs. This has led to a reconsideration of the criterion to allow for a small amount of permanent deformation in the steel reinforcement. McCullough et al.<sup>(17)</sup> suggested maximum rebar stresses based on the premise of allowing some permanent deformation and increased crack width of 0.25 mm (0.01 in). The plastic strain deformation was calculated for a gauge length corresponding to the stress range that exceeds the yield stress. This resulted in a relationship for maximum allowable stress as a limiting criterion as:

$$\sigma_{\max} = (0.19E_s\sqrt{f_c}/d_b 2f_y) + (f_y \times 0.75)$$

However, it should be pointed out that other limiting crack width criteria have been suggested by McCullough et al. based on spalling, steel corrosion, and subgrade erosion induced by excessive crack widths. The crack widths should be limited to 0.61 mm (0.024 in) for spalling considerations and to 0.20 to 0.25 mm (0.008 to 0.010 in) to minimize steel corrosion. These were considered to be too conservative for reasonable reinforcement design. Therefore, it was suggested the limiting design criterion should be based on the selection of a temperature drop below the construction temperature corresponding to a crack width of approximately 0.64 mm (0.025 in). This temperature drop should not be exceeded 95 percent of the time. The 1986 AASHTO Design Guide allows a design crack width of 1.0 mm (0.040 in), which suggests that some corrosion of the reinforcement may be expected.

## Wheel Load Induced Cracking

Wheel loading on a CRC pavement system causes a bending stress that contributes to fatigue cracking as a function of the slab thickness and the level of load transfer efficiency of the transverse crack as a part of the development of punchout distress. Bending stresses associated with fatigue cracking are closely tied to load transfer efficiency and the degree of support at each transverse crack. Load transfer efficiency is a function of the crack width and shear capacity of the transverse cracks. The crack width depends on the crack spacing, the thermal coefficient of expansion of the concrete, and the design steel percentage. This reemphasizes that the spacing between individual transverse cracks is of vital interest in design since maintaining a high level of load transfer will be largely dependent on the width of individual transverse cracks.

On the basis of recent developments in CRC pavement construction technology relative to improved crack patterns, the crack pattern can be controlled through the use of early-aged saw cutting to preselected intervals or allowed to occur randomly, as is the current practice in CRC pavement construction technology. In the case of the latter, the mean crack spacing may be used to estimate the mean crack width. Otherwise, the design crack spacing as generated from the incorporation of early-aged saw cutting technology is used to estimate the crack width.

The basic design process can focus on the prediction of longitudinal cracking prerequisite to the formation of punchout distress in the form of a Weibull-related distribution cracking function:

$$\%C = 100 \cdot e^{-\left(\frac{D}{\alpha}\right)^{\beta}} \quad (7)$$

where  $D$  is the accumulated fatigue damage (due to slab bending in the transverse direction) and  $\alpha$  and  $\beta$  are punchout calibration constants. The fatigue damage due to wheel load and environmentally related stress can be accumulated according to Miner's Damage Hypothesis<sup>(18)</sup> by summing the damage over the entire design period. The applied traffic is computed using traffic data for the design period. Load equivalency ratios are applied to the seasonal and daily breakdown of the traffic to obtain the number of load applications (bending) for design analysis. The method is similar to that used to calculate the accumulated fatigue damage for jointed concrete pavement.

The allowable axle load applications ( $N_f$ ) are estimated from  $\text{Log } N_f = 17.61 - 17.61 \cdot R$ , where  $N_f$  = number of allowable load applications;  $R$  = ratio of applied wheel load stress to modulus of rupture (stress ratio =  $\sigma_{\text{tot}}/\text{MOR}$ ); and  $\text{MOR}$  = modulus of rupture. Fatigue damage analysis, discussed in volume 6, is facilitated by the use of equivalent damage ratios (EDR)<sup>(19)</sup> for CRC pavement. EDR are useful in determining the percentage of traffic applied to the design wheel load location to cause the equivalent amount of fatigue damage as that caused by the entire distribution of traffic. The applied stress is the total of (1) wheel load stress and (2) environmentally induced (curling and warping) stress. These

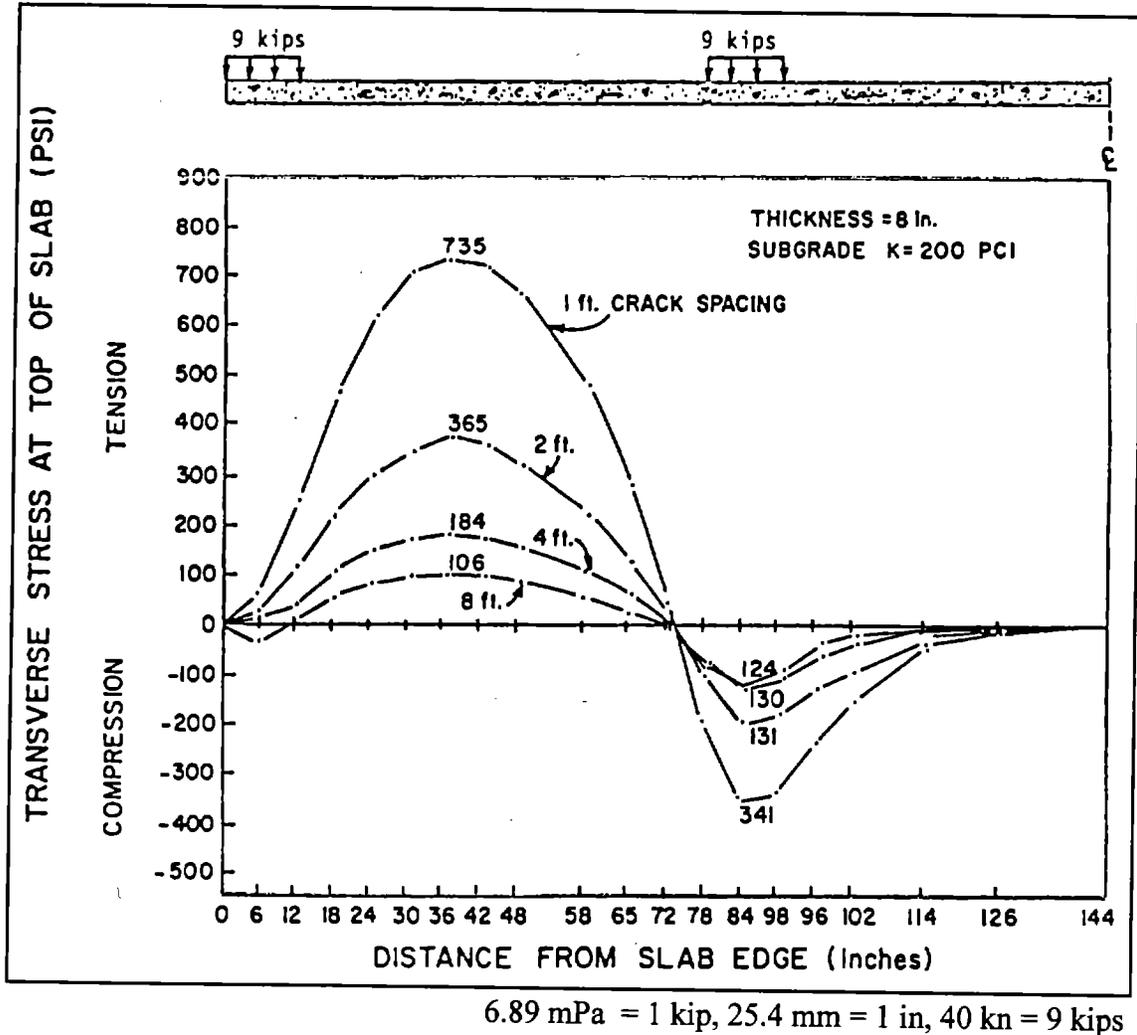
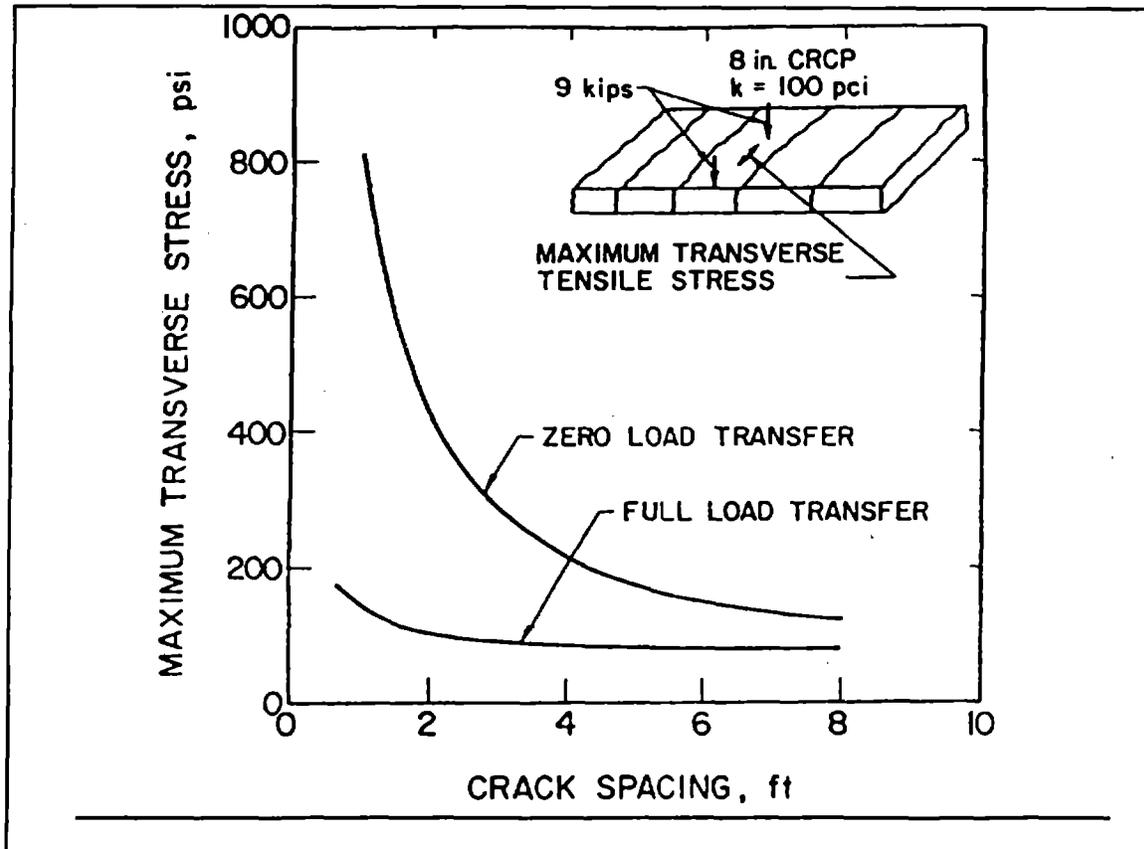


Figure 5. Effect of crack spacing on maximum tensile stress (0 percent LTE).<sup>(20)</sup>

stresses, which are discussed next, vary depending on the base type, shoulder configuration, the level of load transfer efficiency (LTE), crack spacing, and crack width.

#### *Wheel Load Stress - Transverse Bending*

The formation of longitudinal cracking (toward the development of punchout distress) by lateral stresses due to wheel load has been thoroughly reviewed by others.<sup>(20)</sup> Crack spacing has been shown to significantly affect the magnitude of the lateral stresses illustrated in figure 5 and as shown, these stresses increase with decreasing crack spacing. However, a more important parameter is the load transfer across the crack shown in figure 6. Transverse bending stresses ( $\sigma_a$  illustrated in figure 7 between the wheel load positions) are low at high LTE and are high at low LTEs. These stresses are significant below an LTE of 80 percent. In comparison, the longitudinal bending stresses ( $\sigma_b$ ) are relatively low but



(6.89 kPa = 1 psi, 27.1 kPa/mm = 100 pci, 40 kn = 9 kips, 25.4 mm = 1 in, 0.305 m = 1 ft).

Figure 6. Effect of load transfer efficiency across transverse cracks on maximum transverse stress in CRC pavement.<sup>(20)</sup>

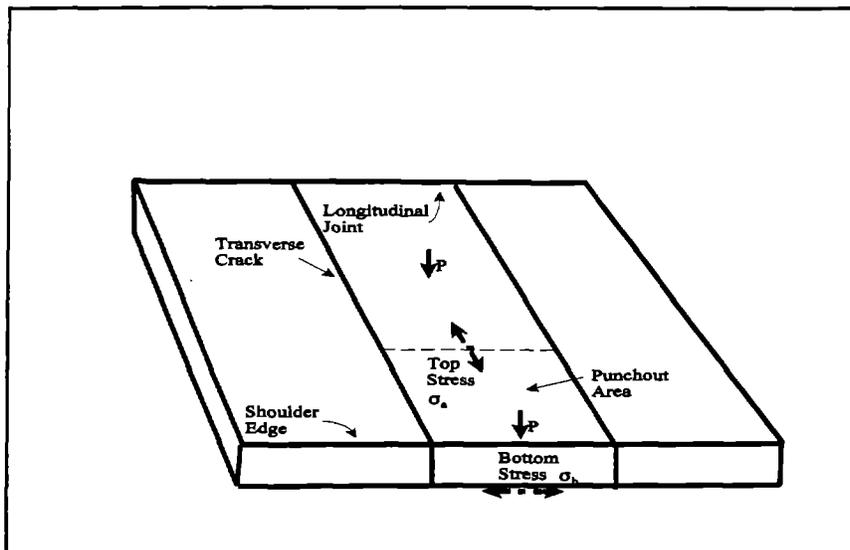
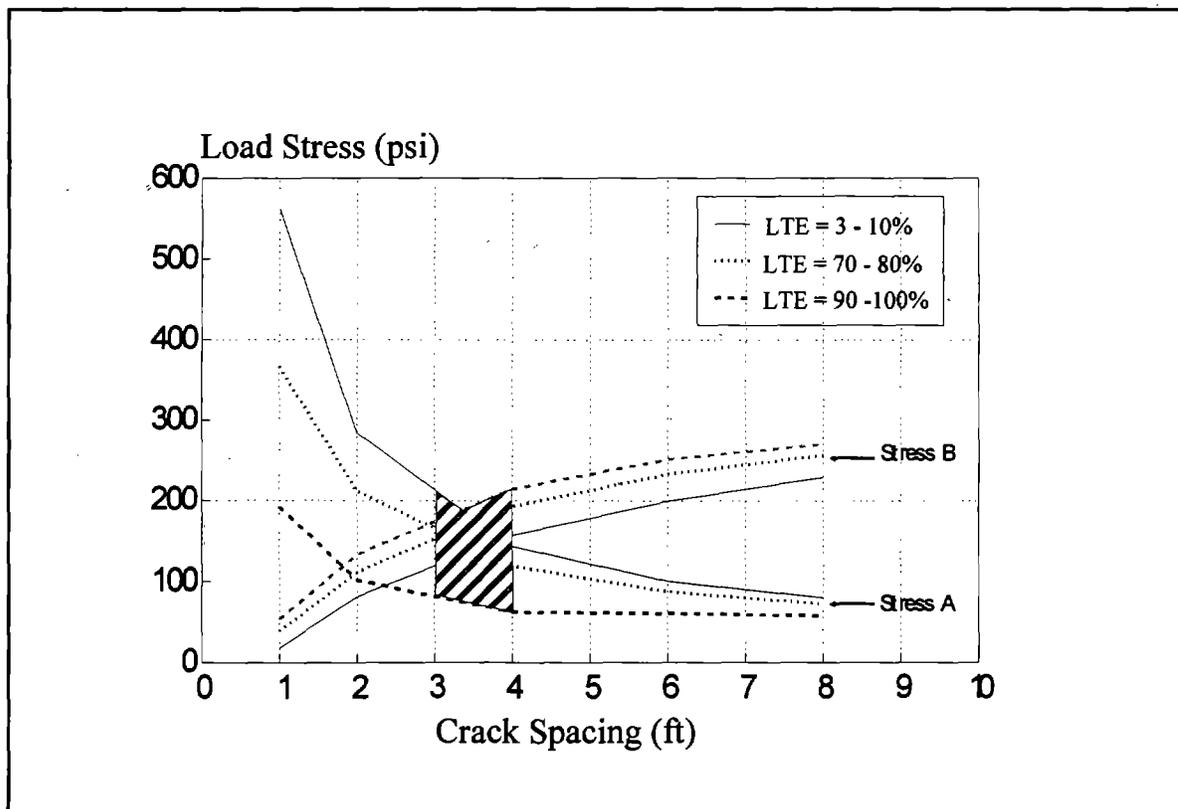


Figure 7. Wheel load stresses in a loaded CRC pavement system.<sup>(1)</sup>

may contribute to some extent to further transverse cracking as part of the overall cracking pattern. Interestingly enough, analysis tends to indicate that the effect of loss of support by itself on  $\sigma_a$  and  $\sigma_b$  stresses is surprisingly small. However, if LTE is diminished because of excessive shear stresses induced by poor support then

these stresses are significantly affected. This means that loss of support acts as a catalysis that precipitates the loss of LTE. Observations of punchouts in field studies were always accompanied by severe subbase erosion and loss of support. Consequently, loss of load transfer is really the dominant effect on excessively high bending stresses that are accelerated due to loss of support and relatively unaffected by environmentally induced slab curling and warping. Coupled with loss of load transfer, curling and warping effects will contribute significantly to longitudinal cracking stresses. However, loss of load transfer is the most significant factor which reemphasizes the importance of considering aggregate wear-out across the transverse cracks.

Figure 8 illustrates a comparison between  $\sigma_a$  and  $\sigma_b$  shown in figure 7 that provides some basis as to how these stresses vary with crack spacing. Stress  $\sigma_b$  decreases with decreasing crack spacing as long as the load transfer remains high. For a CRC pavement with a bituminous shoulder and a given level of aggregate wear-out and loss of load transfer, a crack spacing between 0.9 and 1.2 m (3 and 4 ft) may be the most optimal crack spacing. Within this cracking interval, if the LTE remains high, neither the transverse nor the longitudinal stresses are excessive. However, other shoulder conditions will dictate



0.305 m = 1 ft, 6.89 kPa = 1 psi

Figure 8. Comparison of  $\sigma_a$  and  $\sigma_b$  with crack spacing for a 254-mm (10-in) pavement thickness<sup>(1)</sup> with a bituminous shoulder.

different optimum cracking intervals due to the effect of the shoulder configuration on the load behavior of CRC pavement.

Previous studies<sup>(1)</sup> have indicated that non-uniform supported conditions in CRC pavements seem to have a greater effect on transverse shear stresses than on transverse bending stresses. A more severe shear stress condition will increase at the rate of load transfer loss, which will result in increased bending stresses and greater potential for punchout distress. The shear stresses are reduced with either a 0.6-m (2-ft) extended or a 3-m (10-ft) tied shoulder if sufficient load transfer on the longitudinal shoulder is provided.

Transverse wheel-load stresses can be included in the design process for CRC pavement systems. Using ILLISLAB analysis,<sup>(13)</sup> a data base of maximum transverse wheel-load stresses were generated for a CRC pavement system (under a free edge condition) for a variety of thicknesses, LTEs, and crack spacings. A typical pattern of maximum stresses is shown in figure 9. The contribution of bending stresses to fatigue damage is negligible prior to wear-out of the aggregate interlock and concomitant loss of load transfer. The level of load transfer may also affect the maximum stress location in a CRC pavement system consisting of a bituminous shoulder and to a lesser degree with other shoulder types. The variation of wheel-load stress with LTE and thickness illustrated in figure 9 is based on a cracking interval of 0.6 m (2 ft). Transverse wheel-load stresses in a CRC pavement system are, therefore, at a minimum a function of crack spacing and shoulder configuration. A stress function for transverse wheel-load stresses can be configured as follows:

$$s = \{a + b \ln (L/\ell)\}^{-1} \quad (8)$$

where

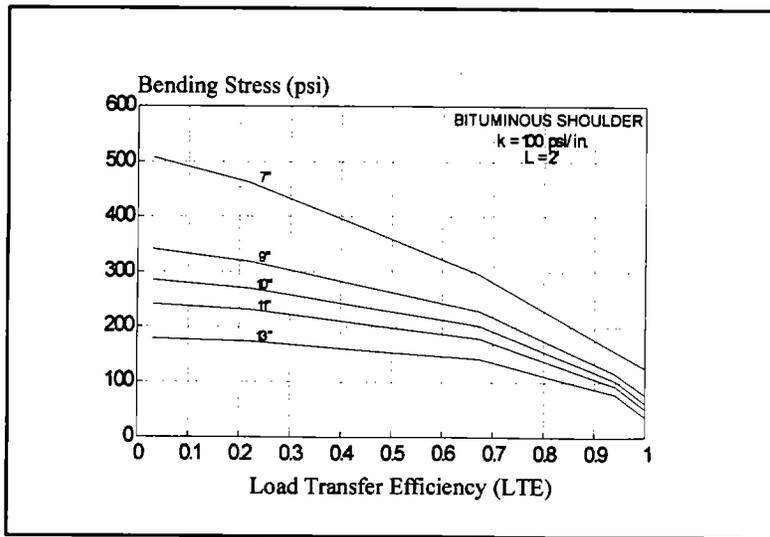
- a =  $\exp(-0.930 + 2.84\{1 + \exp[-(LTE - 96.4)/24.6]\}^{-1})$
- b =  $(0.427 + 9.73 \times 10^{-7} LTE^3)^2$
- L = mean crack spacing (L)
- $\ell$  = radius of relative stiffness (L)
- LTE = load transfer efficiency (%)
- s = dimensionless stress ( $\sigma_{wls}h^2/P$ )
- $\sigma_{wls}$  = wheel load stress ( $FL^{-2}$ )
- h = pavement thickness (L)
- P = wheel load (F)

A load transfer function is necessary to characterize a relationship to incorporate the effects of aggregate wear-out on LTE (via joint stiffness -  $Agg/k\ell$ ). Relating deflection load transfer (LTE) and joint stiffness.<sup>(21, 22)</sup>

$$LTE (\%) = (a + cx + ex^2 + gx^3)/(1 + bx + dx^2 + fx^3) \quad (9)$$

where

$$\begin{aligned}
 x &= \text{Ln}(\text{Agg}/k\ell) \\
 a &= 45.973 \\
 b &= -0.00855 \\
 c &= 19.588 \\
 d &= 0.056 \\
 e &= 2.785 \\
 f &= -1.205 \text{ e-}05 \\
 g &= 0.130
 \end{aligned}$$



0.6 m = 2 ft, 27.1 kPa/mm = 100 psi/in,  
6.89 kPa = 1 psi, 25.4 mm = 1 in

Figure 9. Transverse bending stress: CRC pavement with bituminous shoulder.<sup>(1)</sup>

The relationship between dimensionless shear stress ( $s = \tau P/h^2$  where  $\tau$  is the shear stress on the crack face,  $P$  is the wheel load, and  $h$  is the thickness of the slab) of the transverse crack and the stiffness of the transverse crack as a function of the degree of load transfer offered by a tied concrete shoulder at varying degrees of load transfer is illustrated in figure 10. As the degree of load transfer across the concrete shoulder joint increases, the dimensionless shear stress on the transverse crack decreases, as characterized in the following equation form for a specific crack spacing:

$$s = \frac{a + c \cdot \text{Log} \left( \frac{\text{Agg}}{k\ell} \right)_T}{1 + b \cdot \text{Log} \left( \frac{\text{Agg}}{k\ell} \right)_T}$$

where

$$\begin{aligned}
 a &= a_1 - a_2 \ln (\text{Agg}/k\ell)_s \\
 b &= b_1 + b_2 \exp (-\text{Agg}/k\ell)_s \\
 c &= c_1 - c_2 \ln (\text{Agg}/k\ell)_s
 \end{aligned}$$

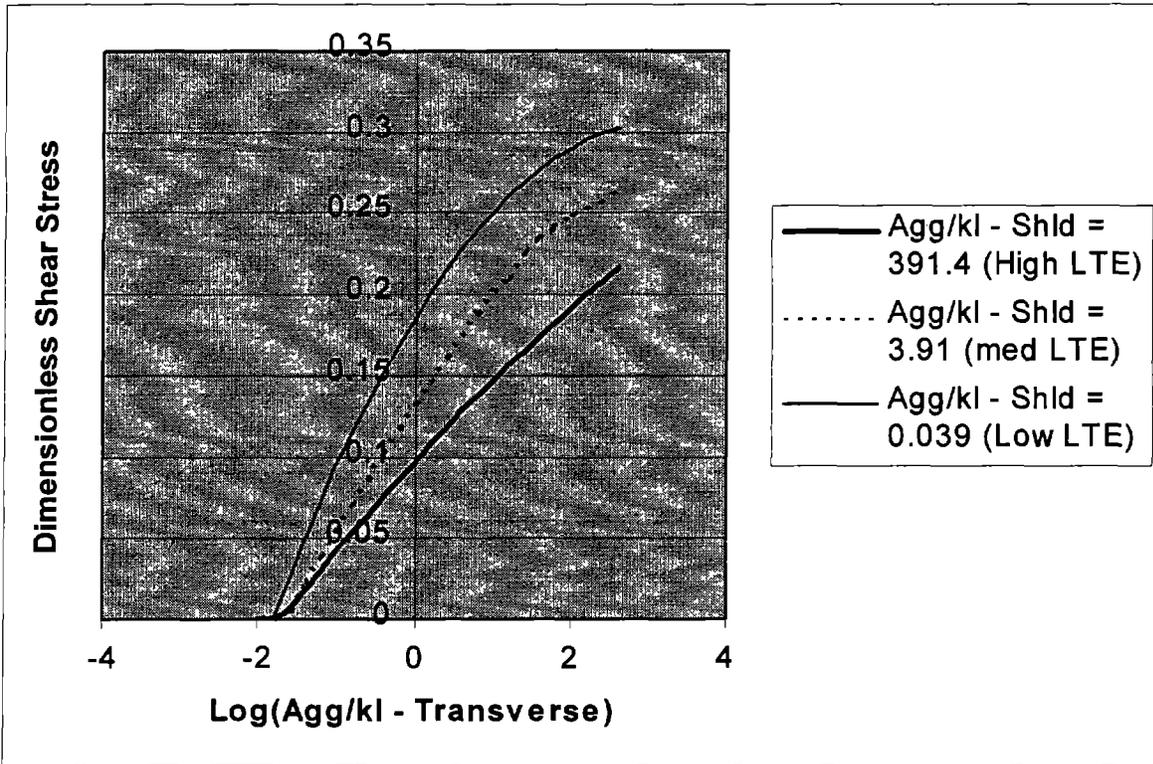


Figure 10. Shear stress as a function of load transfer efficiency provided by a concrete shoulder<sup>(3)</sup>

Note: T: load transfer on the transverse crack; S: load transfer on the longitudinal joint. It is recommended that the coefficients be determined for a 0.61-m (2-ft) cracking interval.

Shear stress also depends on the distance between cracks and decreases as the crack spacing increases. Figure 11 depicts dimensionless shear stress as a ratio ( $s_x/s$ ) of the shear stress for a crack spacing of 0.61 m (2 ft). Therefore, the dimensionless shear stress ( $s_x$ ) can be determined for a wide range of crack spacings in CRC pavement in terms of the dimensionless shear stress ( $s$ ) for a 0.61-m (2-ft) cracking interval in the form of:

$$\frac{s_x}{s} = a_1 + a_2 \cdot \ln\left(\frac{Agg}{kl}\right) + \frac{a_3}{\left(\frac{Agg}{kl}\right)^{1/2}}$$

where

$$a_i = b_1 + \frac{b_2}{\ln L} + \frac{b_3}{L^{3/2}} + \frac{b_4}{e^L}$$

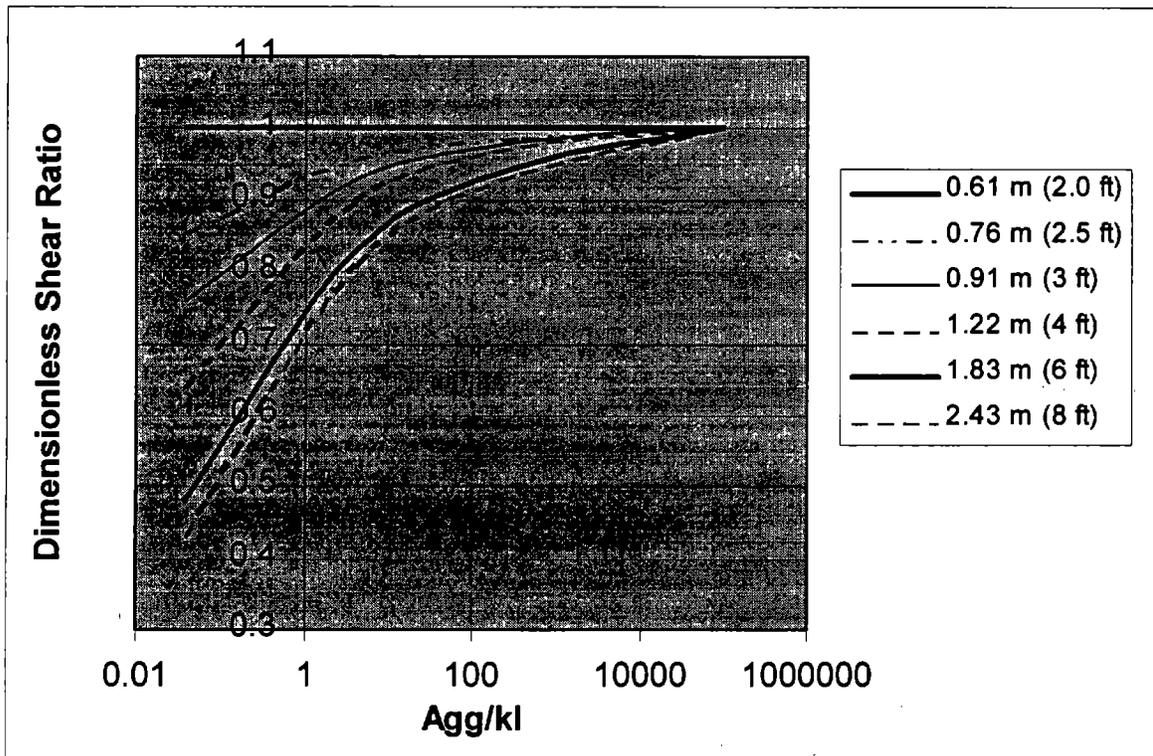


Figure 11. Adjustment of dimensionless shear stress for crack spacing.<sup>(3)</sup>

and

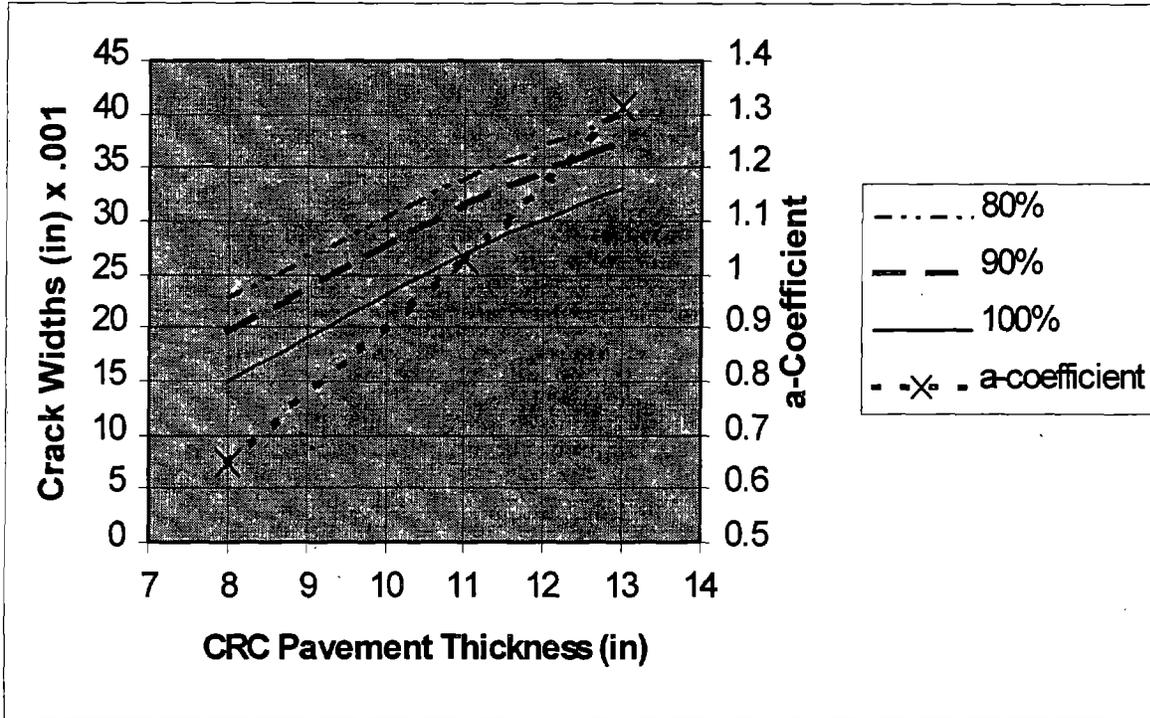
- L = crack spacing
- b = coefficients based on a specific cracking interval (0.61 m is recommended)

It should also be noted that shear capacity is a function of the width of the transverse crack, as characterized in the following form.<sup>(21)</sup>

$$S_{\text{capacity}} = \tau h^2 / P = a e^{-0.039 cw} \quad (10)$$

where  $cw$  = crack width. The value of 'a' ranges from 0.55 to 1.3 as a function of thickness, shown in figure 12. This figure demonstrates crack width requirements relative to slab thickness and load transfer requirements. It should be noted that the limits shown in figure 12 fall between those recommended by Permanent International Association of Roads Congresses (PIARC) (0.5mm - 20 mils)<sup>(23)</sup> and those recommended by AASHTO (1mm - 40 mils).<sup>(24)</sup> Figure 12 suggests that the PIARC requirements are too conservative for typical CRC pavement thicknesses.

The loss of shear capacity ( $\Delta s$ ) due to wheel load applications is also characterized in terms of the width of the transverse crack based on a function derived from analysis of



25.4 mm = 1 in

Figure 12. Maximum crack width limits based on shear capacity requirements.<sup>(1)</sup>

load transfer test data developed by Portland Cement Association (PCA).<sup>(1, 25)</sup> Such a function is important with respect to accounting for the effect of aggregate wear-out in the prediction of performance and remaining life of CRC pavement systems:

$$\Delta S = \sum_i \sum_j \left[ 0.069 - 0.418e^{-cw/h} \right] \cdot \frac{N_i}{10^6} \cdot \frac{\tau_{stress}}{\tau_{ref}} \quad (11)$$

where  $N$  is the accumulated traffic coverages,  $\tau_{stress}$  is the shear stress on the transverse crack (figures 10 and 11), and  $\tau_{ref}$  is a reference shear stress derived from the PCA test result.<sup>(25)</sup> Previous research<sup>(1)</sup> has indicated that non-uniform support conditions can result in an increase in shear stress by a factor of two—which contributes to accelerated aggregate wear-out. Shear stresses are calculated as:

$$\tau_{stress} = sP_i/h^2$$

and

$$\tau_{ref} = s_{pca}(111.1)$$

$$Ln(s_{PCA}) = \frac{a + c \cdot Ln\left(\frac{AGG}{kl}\right)}{1 + b \cdot Ln\left(\frac{AGG}{kl}\right)}$$

where the dimensionless shear is denoted as 's' and  $a = -2.60$ ,  $b = 0.14$ , and  $c = -0.085$ . Equation 11 demonstrates how shear capacity can diminish over time. This expression constitutes the wear-out function, which allows for the deterioration of the aggregate interlock once the LTE drops below 90 percent to be considered in the performance estimate of CRC pavement systems. The coefficients of this function may vary for different aggregate types but preliminary test results<sup>(21)</sup> indicate little differences in the shear wear-out behavior of mixes made with different coarse aggregate types. Further research should be conducted to verify this finding. However, all the expressions introduced above combine together to characterize how the LTE (and consequently, the fatigue stress) can change throughout the performance period of a CRC pavement system. Load transfer is also affected by the opening of the transverse cracks due to thermal affects caused by a change in temperature of the concrete pavement.

Equation 12, shown below, demonstrates how the opening of the crack width can be calculated relative to the temperature of concrete pavement:

$$\overline{cw} = L (z + \alpha_c t_{max}) + \frac{f_t}{E_c} \left( L - \frac{f_t d_b}{4up} \right) \quad (12)$$

where

- $\alpha_c$  = thermal coefficient of expansion
- $t_{max}$  = maximum drop in pavement temperature
- $d_b$  = reinforcing steel bar diameter
- $z$  = concrete drying shrinkage
- $f_t$  = concrete tensile strength
- $E_c$  = concrete modulus of elasticity
- $L$  = crack spacing
- $p$  = percentage of steel
- $u$  = bond strength between the steel and concrete

Subbase friction effects, although not directly included in equation 12, are reflected in the percentage of steel (p) requirements.

The basic outline of design process of CRC pavement is summarized below. The focus of the analysis is the determination of an average crack spacing and crack width and accounting for its effect on the LTE of the transverse cracks within the pavement system.

Use of equations 9, 10, and related expressions are key to this determination. A component of the determination of the LTE is the determination of the shear capacity of the crack relative to the pavement thickness. Using seasonal temperature variations, the crack width and associated load transfer can be varied accordingly. Equation 11 can be used to determine maximum crack widths where no aggregate wear-out occurs beyond which loss of load transverse would accelerate development of punchout distress. The evaluation process follows as:

- 1) Determine the mean crack width - equation 12.
- 2) Determine the shear capacity of the CRC crack pattern - equation 10 and determine the associated mean stiffness of the transverse crack pattern.
- 3) Determine the associated level of LTE - equation 9.
- 4) Determine the associated wheel load stress and level of fatigue damage based on current traffic increment - equation 8.
- 5) Determine level of loss in shear capacity due to load and support conditions for same traffic increment - equation 11 and figures 10 and 11. (new  $S_{\text{capacity}} = \text{old } S_{\text{capacity}} - \Delta S$ )
- 6) Repeat steps 3) through 6) using average LTE and wheel-load stress for the given increment of traffic to determine a new level of fatigue damage.
- 7) Assess the level of cracking - equation 7.

The basic premise of this design concept is based on good pavement support and that load transfer in CRC pavement is a function of the crack width and the intensity of the shear loading. It is well recognized that non-uniform support conditions increase shear stresses significantly and accelerate the loss of shear capacity and development of punchout distress. The design should be adjusted to account for erosion if it is anticipated over the performance period. Data on the thermal coefficient of thermal expansion of the concrete should also be incorporated if they are available.

### *Environmentally Induced Stresses*

Environmental stresses due to curling and warping behavior added to the wheel load stresses contribute as part of the total stress to accumulated fatigue in the concrete slab. Curl and warping stress distribution in the transverse direction across the traveled lane is a function of the weight of the slab, the associated climatic gradient, and the stiffness of the pavement system. These stresses are dependent on the foundation support modulus ( $k$ ), pavement thickness ( $h$ ), and the temperature or moisture gradient ( $G$ ) as a function of the transverse position on the slab. The curl stress in the transverse direction is also a function of the lane width and the shoulder type (i.e., 10-ft tied concrete shoulder, 2-ft extended shoulder, etc.). Curling stress can be derived from daily temperature cycles and warping stress from seasonal moisture variations.

Curling stress for a pavement can be calculated using the well known Westergaard equation for slab stresses under thermal gradients. A similar approach can be taken for slab

stresses caused by moisture gradients. The maximum curling stress ( $\sigma_0$ ) in a concrete slab based on Westergaard's analysis under certain slab edge boundary condition is:

$$\sigma_0 = \sigma^t = \frac{E_c \alpha t}{2(1-\nu)} = \frac{E_c}{2(1-\nu)} \epsilon^t$$

where  $\alpha$  = thermal coefficient of expansion ( $1/^\circ\text{C}$ ),  $t$  = temperature change or drop,  $\epsilon^t = \alpha t$ . Bradbury<sup>(26)</sup> developed coefficients based on the Westergaard solution as applied to slabs of practical dimensions. Warping stresses can be found using the expression noted above for curling stresses by substituting shrinkage strain for the temperature strain  $\epsilon^t$ .

Temperature and moisture gradients in the pavement (which cycle both daily and seasonally) are useful in finding the environmentally induced stresses as a function of time and season. Normally, it is expected that environmentally induced stresses should be broken down on a monthly, daytime, and nighttime basis to coincide with characteristic patterns in truck traffic. Although the discussion here is based on linear temperature and moisture gradients, the framework presented can be adopted to non-linear gradients using the approach suggested by Mohamed and Hansen.<sup>(27)</sup>

### **Construction Methods to Improve CRC Pavement Crack Patterns**

Construction of CRC pavement is characterized by the presence of longitudinal reinforcing steel placed continuously throughout its length. There are no intentionally placed transverse joints (other than construction joints) in the pavement. However, the continuity of the concrete in the pavement is interrupted by a great number of transverse cracks caused initially by volumetric changes in the concrete due to shrinkage and temperature changes. When a transverse crack occurs, the stress distributions in concrete and the reinforcing steel change greatly from point to point in the pavement. Experience has indicated that CRC pavement performance is significantly linked to the resulting transverse crack pattern or post-cracking behavior of CRC pavement. For example, short crack spacings coupled with pavement locations where poor support conditions exist have shown a strong correlation with a high frequency of punchout distress. On the other hand, long transverse crack spacings can lead to large crack openings that may result in crack spalling, steel corrosion and rupture, and poor LTE. Once load transfer has diminished to a certain extent, punchout distress or faulting may be evident particularly where loss of support exists under the pavement.

Some advantage does exist by controlling the crack pattern, for a given set of conditions, to minimize crack widths, spall development, poor crack patterns, and surface defects thereby extending the pavement life. Therefore, positive control of the crack pattern and the initiation of cracking in CRC pavement by initiating or inducing the transverse crack at a desirable crack spacing and crack pattern may be an efficient way to improve the performance of CRC pavement. Since cracking in CRC pavement is, in practical terms, unavoidable it should be employed or induced to the advantage of the design engineer.

In the 1986 AASHTO Guide for Design of Pavement Structures, a procedure was set forth that considers crack spacing, crack width, and steel stress at a crack in the design of CRC pavement. The design percentage of longitudinal steel is selected in such a way that the results from the analysis satisfy the desired range in crack spacing, allowable steel stress, and crack width. This analysis is a function of predetermined parameters such as concrete tensile strength, thermal coefficients of steel and concrete, rebar diameter, concrete tensile stress due to wheel load, concrete shrinkage, and design temperature drop based on predictive formulas. This design method suggests an appropriate percentage of steel reinforcement to distribute transverse cracks, so that instead of a few wide cracks, there are numerous cracks consisting of small widths.

It is expected that the final crack spacing will fall into the desirable range through the above-mentioned design procedure. Unfortunately, it is difficult to eliminate Y cracks and other defects such as closely spaced transverse cracks by only adjusting the amount of longitudinal steel primarily because of the variability of material properties, construction factors, and environmental conditions. Moreover, the early-aged cracking behavior of CRC pavement is not only affected by the previously noted design parameters but also by the vertical location of the longitudinal and transverse steel reinforcement, coarse aggregate type, and ambient temperature condition at the time of paving. Hence, current research efforts are addressing the influence of the above factors in field test sections on crack development in CRC pavement and developing models to consider these factors toward providing and advancing new concepts in the technology of CRC pavement construction.



Figure 13. Early-entry saw cutting technology.

### Crack Control

Cracks in CRC pavement systems can be allowed to occur randomly or be controlled by use of an early-entry saw cut method to induce cracks at prearranged locations (figure 13). Crack induction is useful in the construction of CRC pavement systems to eliminate Y cracks, divided cracks, meandering cracks, cluster cracking patterns, and other undesirable crack pattern features that may increase the variability of cracking or the variability of performance. Crack induction can be best achieved by the use of swallow saw cut notches (as

produced by the early-entry saw cut method) in the surface of the pavement. Field studies of CRC pavement construction have indicated that several transverse cracks are unintentionally controlled by the transverse reinforcement that is typically placed as a part of the normal pavement reinforcement to support the longitudinal reinforcement in position.

CRC pavements placed with skewed transverse steel have approximately a 50 percent reduction in crack initiation by the transverse steel. In this sense, the performance of the longitudinal reinforcement in CRC pavement can be improved so that the resulting crack spacings and widths are limited to the ranges noted in figure 12 using the early-entry saw cut method. Another objective of the longitudinal reinforcement in the pavement structure is to develop uniform crack patterns, but current CRC pavement construction methods allow the crack pattern to develop randomly, which too infrequently results in an unacceptable amount of clustering. Some of the clustering may be due to the previously noted propensity of early-aged cracks to initiate at the location of transverse steel, particularly when placement is done under summer or hot weather conditions.

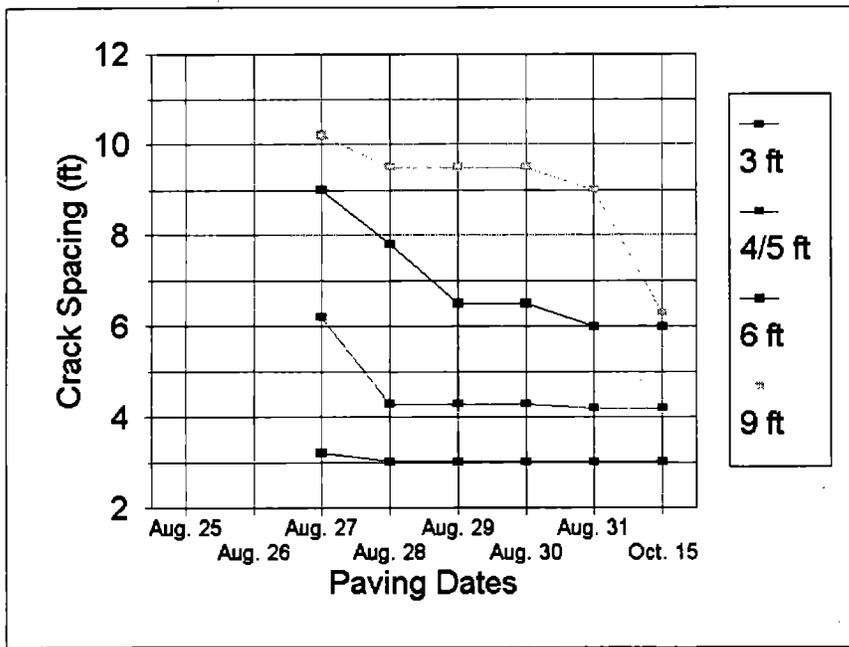


Figure 14. Cracking development at saw cut locations. (0.3048 m = 1 ft).

Experience in early-entry saw cutting practice has indicated that notches should be made between initial and final setting of the concrete. Timing is a very important factor to achieve the goal of artificial crack induction particularly at swallow notch depths. Results from crack surveys conducted on special field test sections have indicated that surface notches placed early (shortly after initial set has occurred) show very positive results and that cracking can be

largely controlled by such notches. Notches made in these special sections were placed about 4 h after placement at 0.9 m (3 ft) and 1.2/1.5 m (4/5 ft) combinations, 1.8 m (6 ft), and 2.7-m (9-ft) intervals. Comparisons of the crack development are shown in figure 14, where it is noted that nearly 100 percent cracking occurred in the notches spaced at 0.9 m (3 ft) and at 1.2/1.5 m (4/5 ft) notch combinations approximately 3 days after paving. However, in the 1.8-m (6-ft) and 2.7-m (9-ft) saw cut interval sections, it took 6 days after placement to reach 100 percent cracking at the notches. As noted in figure 14, secondary cracking occurred (after day 20) in the 2.7-m (9-ft) saw cut interval sections. This may indicate that either the designed length of the saw cut interval or the design percentage of steel reinforcement should be reduced as long as the desired crack widths are maintained (a

10 percent reduction in steel content offsets the cost of the early-entry saw cutting). Comparing similar uncontrolled cracking sections, it was found that it took several months to reach an average crack spacing of 1.8 m (6 ft), or longer to reach an average crack spacing of 0.9 m (3 ft).

### Suggested Guidelines for CRC Pavement Construction and Crack Control

Appropriate CRC pavement design and construction procedures need to recognize and consider the effects of coarse aggregate type selection, curing practice, and weather conditions on performance. Coarse aggregate type selection can be determined in terms of physical properties such as the thermal expansion and the bonding characteristics of the aggregate. The bonding strength can be characterized in terms of fracture toughness (using a modified ASTM C496) of the concrete mixture determined at an early-age of 1 day and the chemical makeup of the aggregate. Although coarse aggregate type may affect drying shrinkage to some extent, this factor is considered insignificant in aggregate type selection.

Coarse aggregate type selection should be made in terms of categories of the thermal characteristics of the concrete mixture or the aggregate itself and on the engineering and chemical properties of the coarse aggregates used in the mix. The proposed categories are as follows:

- Category #1)** Coarse aggregate coefficient of thermal expansion  $< 4.0 \mu\epsilon$  and mixture fracture toughness at 1 day of age  $> 31.3 \text{MPa} \cdot \text{mm}^{1/2}$  (900 psi in<sup>1/2</sup>).<sup>(28)</sup>
- Category #2)** Coarse aggregate coefficient of thermal expansion  $> 4.0$  but  $< 6.0 \mu\epsilon$  and mixture fracture toughness at 1 day of age  $< 31.3$  but  $> 24.3 \text{MPa} \cdot \text{mm}^{1/2}$  (700 psi in<sup>1/2</sup>).
- Category #3)** Coarse aggregate coefficient of thermal expansion  $> 6.0$  but  $> 8.0 \mu\epsilon$  and mixture fracture toughness at 1 day of age  $< 24.3$  but  $> 17.4 \text{MPa} \cdot \text{mm}^{1/2}$  (500 psi in<sup>1/2</sup>).
- Category #4)** Coarse aggregate coefficient of thermal expansion  $> 8.0 \mu\epsilon$  and mixture fracture toughness at 1 day of age  $< 17.4 \text{MPa} \cdot \text{mm}^{1/2}$  (500 psi in<sup>1/2</sup>).

Aggregates in a concrete mixture may be blended to improve the engineering properties of the mixture. Blending can also be considered to improve workability, strength, fracture toughness, and thermal behavior characteristics. Drying shrinkage is largely controlled by the quality of curing.

*Summer vs Winter (or nighttime) Placement:*

*Summertime Placement (air temperature < 32.2° C)*

- Enhanced random crack control by skewing the transverse reinforcement (at a 60° angle) to minimize the incidence of transverse cracking at the location of the transverse bar for sections using *category #1 and #2* coarse aggregate mixtures.
- Positively control the crack spacing and reduce the potential for spalling in pavement sections consisting of *categories #3 and #4* coarse aggregate mixtures with swallow, transverse saw cut notches (made with the early-entry saw cut method) placed at specified intervals in the pavement surface. Also, use the transverse steel (in an un-skewed configuration) to supplement induction of the crack at the surface notches. The percentage of steel should reflect the percentages established by suitable analysis that specify the percentage of steel reinforcement according to the mixture category. *Category #3 and #4* mixtures require less reinforcement to achieve the desired crack spacing and should be designed according to the coarse aggregate properties. The crack pattern can also be satisfactorily controlled with the use of positive control measures on alternating crack locations.
- Use as a minimum a combination of any two of the following curing methods.
  - a. One coat of Type I curing compound,
  - b. One coat of Type II curing compound, or
  - c. Polyethylene sheeting.

*Summertime Placement (air temperature > 32.2° C)*

- Same as above, but use two coats of Type II compound for placements made with *category #1 and #2* mixtures and polyethylene sheeting (with a coat of Type I compound) for placements made with *category #3 and #4* mixtures.

*Winter Placements*

- Use a combination of one coat of Type I and Type II curing compounds and adjust the percent of fly ash to prevent long delays in initial set times.
- Use early-entry transverse saw cutting to minimize the incidence of delamination in *category #4* mixture placements in combination with mid-depth crack inducers (i.e., alignment of double layer transverse steel with the saw cut notches). *Category #4* mixture placements should use inducers placed at mid-depth since crack initiation is much greater at this location in the slab under winter placing conditions.

Concrete strength is not directly considered in these guidelines since its effect is reflected in the aggregate/paste bond strength at an early concrete age.

Control of the crack pattern that develops in CRC pavement can be affected by several factors other than the technique of crack induction. Good mix design, reinforcement steel design, and construction practice will ensure that crack intervals will develop as expected. In traditional design analysis of CRC pavement, the average crack spacing and crack width are derived as a result of the longitudinal steel design, the tensile strength of the concrete, and the design temperature drop. This approach assumes that, when the stress induced by a drop in temperature and drying shrinkage exceeds the tensile strength, a crack forms in concrete pavement. Naturally, a great degree of variation is expected (and does occur as surface defects) in the actual crack patterns, which, if significantly reduced, will result in more economical CRC pavement designs providing greater performance lives.

### **Evaluation of CRC Pavement Performance**

This section focuses on the analysis and the process of the evaluation of CRC pavement behavior and support and its application to design. An approach to the evaluation of existing CRC pavement should take into account two factors relative to performance: (1) the crack pattern and (2) the pavement support system. Characterization of the crack pattern can be accomplished from analysis of crack spacing data. Poor crack pattern characteristics such as Y-cracks, divided cracks, close crack intervals, etc., can be included in parameters derived from analysis of the cracking data. Characteristics noted above can increase the potential for punchout distress if poor support conditions develop or wide cracks occur. Therefore, the effect of the crack pattern should be included as part of the evaluation process of CRC pavement. An approach to the structural evaluation of CRC pavement should take into consideration the development of pavement distress in terms of both fatigue cracking and pavement support.

Uniformly distributed pavement support has been recognized for several years as the key to long-term performance of CRC pavements (particularly for CRC overlays). However, the consequence of lack of uniform support appears to have been only indirectly considered in the design of CRC pavements in terms of the erodibility of the subbase surface. Recent experience in Pennsylvania,<sup>(29)</sup> Wisconsin,<sup>(30)</sup> and Arkansas has indicated a need to consider non-uniformly supported condition for CRC pavements, especially those placed as overlays on jointed concrete systems. Concentrated shear stresses (which can be very intensive) that result in punchout distress are difficult, if not impossible, to account for in design and are generally caused by unsupported subbase conditions. The characterization and analysis of the support under a CRC pavement (or overlay) are based on initial stiffness (at the transverse cracks) of the pavement system and can be described in terms of non-destructive testing (NDT) results. It is noted that these results may depend on several factors that are affected by the degree of pavement support and are therefore useful in characterizing subbase support.

### **Crack Pattern Evaluation**

The evaluation of the crack pattern can be broken down into three areas described below: (1) crack condition, (2) randomness of the crack pattern, and (3) cluster cracking.

### *Crack Condition*

A key factor in the evaluation of the crack pattern is the condition of the transverse cracks. Table 1 is provided to assist in the visual evaluation of the transverse cracking condition. The crack classification categories listed in table 1 are broken down into four groups (as modified from AASHTO Road Test - Report 5). C-1 and C-2 cracks are typically considered to be cracks in good condition and exhibit a high degree of stiffness. The crack width categories are based on widths at the surface of the pavement but typically are much narrower within a short distance below the pavement surface. C-3 and C-4 cracks are typically associated with punchouts in the later stages of development and exhibit low stiffness characteristics. Consequently, it is possible to generally associate the crack classifications with NDT evaluation of different transverse cracks.

Table 1. CRC pavement classification system.<sup>(1)</sup>

C-1:	Fine crack not visible under dry surface conditions at a distance of 4.5 m (15 ft). (Tight)
C-2:	A crack that can be seen at 4.5 m (15 ft), but exhibits only minor spalling. The opening at the surface is 0.8 mm (30 mls) or less. (Open)
C-3:	The crack is opened at the surface 0.8 mm (30 mls) or more for any portion of the crack length. The crack exhibits low to medium spalling. Amount of faulting is noted.
C-4:	The crack is either very wide >1.6 mm (60 mls) or sealed and exhibits medium to severe spalling. Amount of faulting is noted.

### *Randomness of Cracks*

Cracks in CRC pavements can have various shapes. Some cracks might be straight and some curve or meander in shape. Cracks that meander increase the probability of secondary cracks, which result in punchouts. The randomness of the crack can be found by rating the individual crack.<sup>(31)</sup> The Randomness Rating (RR) concept was developed by McCullough et al.<sup>(31)</sup> RR is the mean of the individual randomness ratings. An individual randomness rating is a subjective rating of the randomness of a specific crack by an individual rater, similar to the scale associated with the Present Serviceability Rating.<sup>(31)</sup>

- 5.0: Very Good (almost straight crack)
- 4.0: Good
- 3.0: Fair
- 2.0: Poor
- 1.0: Very Poor (very meandering)

A mathematical model of the randomness index (RI) is derived by correlating the RR with objectively measured values taken from the corresponding cracks. The RI model can be used to obtain an estimate of the RR for any crack without the need for any further

rating. The RR of a crack is represented by  $RR = RI + \epsilon$ , where  $\epsilon$  = the residual not explained by the mathematical model.<sup>(31)</sup>

To determine the randomness, the curve length of the crack (L), the lane width (W), and the number of concrete blocks (N) that are associated with the crack and enclosed by secondary cracks are measured. These parameters are chosen because they are simple to measure and because the effects of secondary cracks which form Y-cracks or punchouts are reflected by the number of separated concrete blocks (N). Randomness (R) is represented by the following equation:<sup>(31)</sup>  $R = 100 \times (L-W)/W$ , where R = randomness; L = curve length of the crack; W = lane width.

Using the general linear model procedure, a mathematical model for the RI, which is a function of R and N, was developed.<sup>(31)</sup>

$$RI = \frac{5.463}{(R + 1)^{0.259} (N + 1)^{0.510}}$$

### *Cluster Cracking*

A very short cracking interval (which may occur in clusters) has been recognized as an undesirable feature, especially in combination with poor support conditions. It is of interest to characterize the occurrence of cluster-cracking in a CRC pavement system in terms of the percentage frequency of cracks occurring in clusters. The crack spacing frequency distribution can provide an indication of the level of cluster cracking. Cluster cracking is a type of distress in CRC pavements. Consequently, cluster cracks typically will act as a locus for punchout development under repeated application of traffic loads. Shear stress may also be higher in these groups of cracks possibly leading to excessive wear out of the aggregate interlock and contributing to a greater rate of punchout distress at these locations. Generally speaking, cluster cracks occur within a distance of 0.3, 0.6, or 0.9 m (1, 2, or 3 ft) intervals. The probability of two, three, or four consecutive cracks occurring within a range of distances can be chosen as a basis to evaluate the evidence of cluster cracking within a particular pavement segment.

Cluster cracking is found from crack spacing distribution data with respect to the probability that a specified number (say two) of consecutive cracks occurring within less than a 0.3-m (1-ft) distance, a 0.6-m (2-ft) distance, etc. can occur. A simple algorithm to calculate the probability of cluster cracking was developed and presented in volume 6 of this report.

### Characterization of Pavement Support Conditions

In the characterization of pavement support conditions, the following types of field information regarding the existing pavement structure are found to be necessary: (1) foundation modulus or k-value; (2) thickness and elastic modulus of each layer, and (3)

FWD data (tests normally conducted in the morning hours) at cracks, joints, and other locations in the existing CRC pavement.

#### *Foundation Modulus or Subgrade k-Value*

The k-value of a soil or its modulus of subgrade reaction is indicative of the support provided by the subgrade and is important, along with its associated variability or coefficient of variation (cv), in a thickness design process. The conventional method of using the plate-bearing test may be used to determine the modulus of subgrade reaction, but may not be practical in some instances, particularly in the case of the construction of a pavement overlay. Various strength tests performed on the subgrade soils in which correlations to k-values for the soil are available may be used to characterize the subgrade k-value. NDT may also be used to determine a subgrade k-value at the center of a slab location by back-calculation methods based on Westergaard's-type formulations. This approach can provide acceptable foundation support data. When a subbase or a stabilized subbase is used under the pavement, the k-value determined from NDT data can serve as a representative value of the entire foundation support unless individual layers are taken directly into account in the analysis process.

#### *Thickness and Modulus of Each Layer*

The type, thickness, and modulus of each layer in the existing pavement section must be known so that effective modeling of the system layer configuration is possible. From the as built plans and profiles of the existing subgrade, details of the thickness of each layer in the pavement section are typically available. The type and thickness of the subbase material may also be noted. The elastic modulus of the existing concrete may be determined from the testing of available cores taken from the original slab or other means and used as an input to the NDT analysis. The modulus of the subbase is also determined using compression tests on field or laboratory samples depending on the type of stabilizer used.

#### *Falling Weight Deflectometer (FWD) Data*

*Load Transfer Efficiency.* Joints have been recognized as a major focal point for pavement distress in jointed concrete pavements and, consequently, transverse cracks are in many instances the source of problems that develop in CRC pavement (and overlays placed on jointed pavement systems). Data related to slab deflections and LTE are obtained during testing and are a primary way of characterizing support conditions under the original pavement. Results of the FWD testing may be described in part with respect to the plate deflection ( $D_o$ ) and the LTE. LTE can be defined as the deflection on the unloaded side of the crack divided by the deflection on the loaded side of the crack:  $LTE (\%) = 100 \times (\Delta L / \Delta A)$ , where  $\Delta L$  = unloaded deflection;  $\Delta A$  = loaded deflection. The objective of a perfectly efficient system for transferring load is to minimize tensile stresses and, in the case of CRC pavements and overlays, the deflections in the pavement that result when loads are applied at (or between) transverse cracks in the pavement.

*Basin Area.* When any type of load is placed on a rigid pavement slab, the slab will deflect to form a basin. The deflected shape of that basin is a function of several variables, including the thickness and stiffness of the slab, the stiffness of the underlying materials, and the magnitude of the load. This may be depicted by the shapes of the basin area created by different strengths or types of subgrade material or different slab configurations. Basin area gives an indication of the deflection profiles measured using FWD, and may be calculated from sensor deflections as:

$$\text{Area} = 12/(2 \cdot D_o) [D_o + 2\{D_1 + D_2 + \dots + D_{n-1}\} + D_n]$$

where area = basin area, in;  $D_i$  = measured sensor deflection;  $n$  = number of sensors (at 0.3-m (12-in) spacing) on one side of load plate minus one.

This area concept combines all measured deflections in the basin into a single parameter. The area being determined is essentially one-half of the cross-sectional area of the deflection basin taken through the center of the load. Each deflection reading is normalized with respect to the maximum deflection  $D_o$ . Thus the basin area has the units of length and is a function of the number and location of the sensors.

With respect to the evaluation process, all the measured basin areas are averaged to determine a mean basin area and the coefficient of variation. This information is used subsequently in the assessment of the variability of the remaining pavement life. For any given sensor arrangement, a relationship between the basin area and the radius of relative stiffness ( $\ell$ ) exists. This forms the basis of the representation of different load transfer conditions in the existing slab as explained later.

*Radius of Relative Stiffness.* In the analysis of rigid pavements, one of the stress inducing factors is the continuity of the subgrade support as affected by permanent deformations of the subgrade or loss of support. A concrete pavement slab deforms under load depending on the position, magnitude, and area of contact of the load on the pavement surface. The resistance to deformation depends on the stiffness of the supporting medium, as well as the flexural stiffness of the slab. This parameter, referred to above, is called the radius of relative stiffness ( $\ell$ ), and depends on the properties of both the slab and the foundation. This relative stiffness is defined as follows:

$$\ell = \left( \frac{E h^3}{12 (1 - \nu^2) k} \right)^{\frac{1}{4}}$$

where  $E$  = concrete modulus of elasticity (psi);  $h$  = thickness (in);  $\nu$  = Poisson's ratio;  $k$  = foundation modulus (psi/in).

### Characterization of Pavement Stiffness Conditions

Application of theoretically sound, mechanistic concepts to the structural evaluation of an existing CRC pavement has been prompted by the development of commercially

available devices for NDT such as the FWD. This can be achieved by matching the theoretically predicted response of the system, typically in the form of a deflection basin, with corresponding behavior observed in situ through the selection of appropriate system parameters such as layer thicknesses and moduli.<sup>(32)</sup>

Failure modes relating to punchout distress have been proposed as fundamental thickness design mechanisms for CRC pavements and CRC overlays. The analysis of these failure modes is based a priori on uniform support conditions.<sup>(1)</sup> Hence, it is important to incorporate a structural model that will allow matching of the deflection basin as measured in the field. A closed-form solution has been suggested by Ioannides et al.<sup>(32)</sup> for back-calculation purposes. A slab with a joint/crack is characterized to represent field conditions with respect to support conditions and load transfer conditions of the transverse cracks. The purpose of this is to back-calculate either an effective layer modulus or a composite k-value as determined by the collected field data. This information is used later in the determination of an “effective” stiffness of the transverse crack.

The slab may either be bonded to the base or it may be unbonded. In either case, it is most appropriate to consider the base or the subbase as a part of the pavement system rather than part of the pavement support. For modeling an unbonded condition, a two-layer analysis may be used where the existing pavement is modeled atop a stabilized base (if one exists) and the subgrade. This approach can provide a back-calculated k-value or an effective layer modulus.<sup>(32)</sup> In a bonded slab, the ILLI-SLAB finite element program treats two layers as one equivalent layer with a composite layer thickness. If the existing slab has no stabilized base, two-layer analysis is most appropriate.

A back-calculated k-value (whether the pavement is bonded or unbonded) as approximated from Westergaard analysis for an interior load condition is:

$$k = \frac{P}{8D_o(\ell_m)^2} \left[ 1 - \left( \frac{a}{\ell_m} \right)^2 (0.217 - 0.367 \log \left( \frac{a}{\ell_m} \right)) \right] \quad (13)$$

The effective elastic modulus ( $E_{eff}$ ), if of interest, may also be back-calculated as:

$$E_{eff} = \ell_m \cdot 12 (1 - \nu^2) \frac{k}{h^3}$$

where P = load applied in lb;  $D_o$  = maximum deflection under the load;  $\ell_m$  = radius of relative stiffness corresponding to the measured basin area;  $\nu$  = Poisson’s ratio; k = back-calculated subgrade modulus.

This approach of back-calculation can also be used to characterize the pavement behavior in terms of the structural parameters of the original pavement system. It may be

shown that an “effective” pavement stiffness ( $E_c h_c^3$ ) can be defined in terms of the existing pavement system and an unbonded overlay as:<sup>(32)</sup>

$$E_c h_c^3 = E_1 h_1^3 + E_2 h_2^3$$

where,  $E_c h_c^3$  = effective pavement stiffness;  $E_i$  = flexural moduli of the pavement layers (1 = overlay, 2 = existing slab); and  $h_i$  = thicknesses of the pavement layers. A similar approach may be applied to the case of a bonded overlay.

It should be noted that an effective pavement stiffness ( $E_c h_c^3$ ) may also be determined on the basis of a field-measured  $\ell$ -value ( $\ell_m$ ) as:

$$E_c h_c^3 = \ell_m^4 12 (1-\nu^2) k$$

and,

$$\ell_m = a_1 + a_2 \cdot \text{Area} + a_3 \cdot \text{Area}^2 \quad (14)$$

where  $a_1 = 74.32$ ,  $a_2 = -4.185$ , and  $a_3 = 0.003163$  are regression constants with  $r^2 = 0.9949$  and the SEE = 0.7548. This expression is applicable and dependant on the configuration of the pavement system that exists at the time of the FWD testing, whether it be a single- or two-layer system. This value is used in equation 14 to calculate the composite pavement stiffness with an overlay.

Therefore, a composite stiffness ( $E_c h_c^3$ ) may be determined, for design purposes, for unbonded as well as bonded layer conditions relative to the basin area and to the radius of relative stiffness of the existing pavement system.  $E_1$  and  $h_1$  are the elastic modulus and the thickness of the pavement surface (or overlay), and  $E_2$  and  $h_2$  are those of the lower pavement layer and may be considered to be effective values since they may include the effective of the transverse cracks and joints.

### *Evaluation Process*

With the important parameters relevant to the evaluation of CRC pavements highlighted, the evaluation process can be elaborated. The evaluation approach discussed in volume 6 of this report is intentionally configured to conform to the design process. A direct comparison between the design life and the performance life of the pavement system is provided.

Using NDT data obtained from the use of an FWD, the radius of relative stiffness (RRS) is determined from basin area calculation using equation 14. The RRS and  $D_0$  values are also used to determine back-calculated  $k$  values using equation 13. The RRS,  $k$  values, and the measured LTEs are then used to find the crack stiffness (AGG) for each tested transverse crack. The present level of shear capacity ( $\tau h^2/P$ ) is next found from known values of  $k$ ,  $\ell$ , and AGG, which allows for the determination of the effective crack width of each tested transverse crack. At this point, all of the necessary inputs are available to assess

the remaining life of the CRC pavement system. For any level of expected future traffic, the loss in shear capacity can be assessed, allowing for the determination of a new crack stiffness value ( $AGG/K\ell$  - at a lower level), which corresponds to a reduced LTE. It is evident that a reduction in the randomness of the cracking interval will lead to an improved performance level of the CRC pavement system since a higher level of performance can be achieved by improving the characteristics of the crack pattern.

### *Application to Overlay Design*

The above evaluation process can be utilized in the determination of overlay thickness as a function of the pavement condition from a structural stiffness perspective. It is important to consider whether the overlay is to be bonded or unbonded to the original pavement surface in the design process and whether or not the existing pavement layer is CRC or jointed concrete. A bonded overlay may be modeled by treating the overlay and the existing slab as one layer in terms of an effective thickness by the use of the parallel axes theorem. This composite layer can be modeled atop the base or the subgrade using a back-calculated k-value for the slab support. In the case of an unbonded overlay, a back-calculated “effective” elastic modulus (for the existing pavement) and k-value at the top of the base may be used.

### **Maintenance and Repair of CRCP**

The cost-effectiveness of rehabilitation depends on performing the appropriate and adequate rehabilitation at the appropriate time in the life of the pavement. The following questions must be answered to determine the cost-effectiveness:

1. How does repair plus a concrete or an asphalt concrete overlay compare in cost effectiveness with repair only?
2. What quantity of pre-overlay repair is most cost effective?
3. What is the appropriate time to perform rehabilitation to maximize cost-effectiveness?

Over the last 20 years, Illinois and Texas DOTs have conducted studies to address the questions listed above. These studies indicate that, once punchout distress begins to show up, the rate of distress development increases with time/traffic application. Most agencies repair punchouts on an as-needed basis, generally a few locations at a time. Once a certain number of punchouts/km have been repaired, it is time to consider a more permanent rehabilitation of the pavement. Typically, the rehabilitation has involved use of an asphalt concrete overlay, although bonded concrete overlays and unbonded concrete overlays have also been used. Resurfacing the existing distressed CRC pavement at the right time can significantly extend the service life of the pavement at an overall lower cost. The generally recommended optimal time for resurfacing is when the number of failures exceed 9 to 12 per km (15 to 20 per mi) or the amount of repaired area exceeds about 2 percent of the total pavement area.

Highway agencies need to know when in the pavement's life rehabilitation will be most cost effective and, if adequate funding is not available at that time, the consequences of delaying rehabilitation. The answers to these questions will likely be different for each project. Delaying the overlay is mostly offset by the costs of keeping up with full-depth repairs, with the result that smaller savings are achieved in subsequent years. Performance data that illustrate the rate of deterioration accelerates rapidly beyond the point at which the pavement's fatigue life is exceeded. Also, expenditures for full-depth repairs accelerate as the pavement continues to deteriorate. Delaying structural improvements significantly beyond the end of a pavement's structural life is very seldom cost-effective.

CRC pavements are a very unforgiving but potentially a zero-maintenance pavement design. Any mistakes in the initial design, construction, or repair will, within the lifetime of the pavement, cause a costly distress to repair. The most expensive rigid pavement design for repair and maintenance is CRC pavement (CRCP). The patched area must be carefully demolished, removed, and replaced. The reinforcing steel must be reconnected to provide steel continuity. All these factors lead to the increased cost of repairing CRC pavement. Key points relative to the maintenance and repair of CRC pavement systems follow:

1. Of the many distress types that affect CRCP, pumping and punchouts are the most common and severe. The mechanisms of the distresses must be known and recognizable to the maintenance engineer. If the mechanisms causing the distresses are not also addressed through the repair procedure, the repairs may also fail by the same mechanisms.
2. Preventive maintenance, often overlooked, is cost effective and efficient in stopping the deterioration of CRCP. Joint sealing and underdrains will limit the amount of water in the pavement structure thus reducing those distresses in which water is a major role player in their formation (pumping, punchouts, voids). Where there have been voids, pumping, and increased deflections, undersealing may be used to reestablish a fully supported condition to the pavement.
3. The smaller the repair area the less costly it is to patch the pavement. The restriction on the smallest patch area was once 3.0 m (10 ft) wide and a full lane width. The dimensions of patch areas have now been reduced to as small as 4 ½ ft (1.5 m) wide and half a lane width. The patch area MUST contain all of the deteriorated pavement and subbase, otherwise the patch or adjacent pavement will again become distressed and require maintenance.
4. The continuity of the reinforcement must be continued through the patch. The movement of the adjacent CRCP slabs will cause an untied patch to become distressed and deteriorate the adjacent concrete. The uses of welded splices and mechanical couplers are viable alternatives to the use of tied splices.

5. Patching should be done in the spring and fall when extreme daily temperatures are at a minimum. Patching should be done after noon, especially during the summer months, to avoid possible crushing of the patch due to the expansion of the CRCP free ends. The effective stiffness of all patches should be checked to see if any further restoration is needed.
6. The overall cost of full-depth patching can be reduced in many ways. The removal of the deteriorated concrete can be quickly done with the lift out method. The end sections' length can be reduced with the use of welded splices or mechanical couplers. The curing time of the concrete patch can be decreased with the use of an insulation layer and the patch can be opened to traffic when the modulus of rupture is 300 psi (2.07 MPa).
7. Partial depth patching provides a cost effective alternative to full-depth patching, if it is used on certain types of concrete surface distresses.

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