the tensile strength that corresponds to the modulus of elasticity of PCC determined from the fatigue analysis falls within the bounds determined from the low-temperature and shrinkage analysis. The lower bound on the tensile strength selected from the low-temperature and shrinkage analysis is greater than the minimum tensile strength indicated by the examination of spalling information. Thus, the set of material properties listed above may be considered an optimal combination of material properties for a premium CRCP designed with the pavement components that are described in the section of this paper that deals with specific inputs to distress models.

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Limiting Criteria for the Design of Continuously Reinforced Concrete Pavements

B. FRANK McCULLOUGH, J. C. M. MA, AND C. S. NOBLE

The primary factors to consider in the thickness and reinforcement design for continuously reinforced concrete pavements (CRCPs) are the structural response variables—crack spacing, crack width, and maximum steel stress. They perform an important role in the outcome of the pavement's performance and can be related to the major distresses common to CRCPs. This paper describes the design-limiting criteria for these structural responses. Previous investigations of the design criteria are reviewed, and the most recently developed analytical models are studied. The basic procedures used to establish design criteria include an examination of the major distresses, such as punchout, spalling, and steel rupture, and a study of correlations between these distresses and the corresponding structural responses at appropriate levels. The procedure for use of the limiting criteria in CRCP design is outlined.

The design concept for continuously reinforced concrete pavement (CRCP) is to force cracks to form at relatively close intervals, thus controlling the tightness of the crack to provide good load transfer and prevent excessive water percolation. The frequency of cracks and the final crack width depend on a complex interaction of environmental variables, material properties, and magnitudes of applied loads. Initial cracks in the CRCP are primarily caused by critical stresses induced by the initial temperature drop and drying shrinkage of the concrete. Additional cracks may develop during application of an external load when the combined stresses of the internal and external forces exceed the concrete tensile strength. Close to 90 percent of the transverse cracks occur within one month after construction. The crack pattern will eventually reach a stabilized condition when the pavement has experienced the minimum temperature during the cold season and when most of the drying shrinkage in the concrete has occurred.

The CRCP-2 computer model (1) was designed to fully simulate the mechanistic behavior of the CRCP with respect to time and load. The model predicts the structural responses of the CRCP to environmental load and static external load from the time that initial cracks form to the time when the volumetric changes of the CRCP have stabilized. The final crack spacing, crack width, and steel stress appear to strongly influence the performance of the

CRCP because major distresses common to CRCP are highly correlated with the types of responses noted here.

In a report by Noble, McCullough, and Ma (2), relations between the significant input variables and the structural responses predicted by the CRCP-2 model are quantified by using regression techniques and are expressed as a set of nomographs. This set of design charts allows a graphic prediction of the final responses--crack spacing, crack width, and steel stress--and greatly reduces computation time and effort.

The first objective of this study is to investigate correlations between mechanisms of major distress and structural responses as predicted in works by Noble, McCullough, and Ma (2,3). Design criteria for each of the responses are then established to control and restrain distress that would otherwise adversely affect the performance of the continuous pavement.

MECHANISTIC BEHAVIOR OF CRCP

Much information concerning major distress in CRCPs can be found in studies conducted by Darter and Barenberg (4) and McCullough and others (5). The following table lists the predominant distress types found in CRCP and summarizes the frequency of occurrence and severity of the distress types in 12 projects (3,4):

Distress Type	Total Projects	Distressed	Maintained
Surface depression	12	7	0
Crack spalling	12	6	2
Punchout	12	4	4
Interconnecting cracks	12	4	2
Longitudinal cracking	12	2	0
Steel rupture	12	2	2

The information presented above was obtained from pavements that had survived for 20 years. Maintenance was applied only to the specific distress noted in the above table. Results from statewide condition surveys, along with the collected experience of prominent researchers, were used in establishing the significant distress types and rank order.

The following table shows the resulting priority ranking of distress types for CRCP in decreasing order of the significance of their effect on pavement performance:

Rank	Major Distress Type
1	Punchout
2	Crack spacing
3	Fatigue cracking
4	Low-temperature cracking
5	Shrinkage cracking
6	Steel rupture

Fatigue cracking, low-temperature cracking, and shrinkage cracking are secondary distress types that define the spacing of transverse cracks in the continuous pavement. Secondary distresses are responsible for the development of the primary distress that leads to reduction of serviceability in the pavement. Punchout, for instance, is a primary distress type that occurs between closely spaced transverse cracks that are subsequently connected by longitudinal cracks. Steel rupture is ranked last and does not usually occur in the southern United States.

Criteria for CRCP Structural Responses

The primary factors to consider in the design of CRCPs are the structural responses--crack spacing, crack width, maximum steel stress, and maximum concrete stress. They play an important role in the outcome of the pavement's performance and can be related to the major distresses discussed previously. These factors are also interrelated with each other. A design that forces cracks to form in either a narrow or wide space will affect the accumulated drag forces due to frictional restraint from the subbase and subsequently will alter the level of response of crack width, maximum steel stress, and maximum concrete stress.

Model Description

The computer program CRCP-2 (1) models the one-dimensional changes in concrete stress, steel stress, crack width, and crack spacing that occur in a CRCP and that are caused by drying shrinkage of the concrete, temperature variation, and wheel loads.

The difference in the thermal coefficients of the steel and the concrete, together with the drying shrinkage of the concrete, enables us to determine the internal stress in the reinforced slab. By using the friction-movement characteristic of the slab and the soil, as determined in laboratory experiments, the degree of restraint due to the soil frictional resistance can be estimated (1). By assuming equilibrium in the system, the stress of one material can be computed in terms of the stress of the adjacent materials. Finally, an incremental approach can be adopted to predict the formation of transverse cracks as a function of time by comparing the historical changes of the concrete stress with the strength of the concrete. A complete list of the assumptions made during the development of the model appears in Ma, McCullough, and Noble (3).

Structural Responses of CRCP

The transverse cracking in a continuous pavement is the result of the restraint of the pavement slab induced by internal environmental forces and external wheel-load forces. Most transverse cracks occur at an early age of the pavement when most of the moisture evaporation takes place. Additional cracks may later develop if the stress, which has been increased by the wheel-load application, exceeds the fatigue strength of the concrete.

Spacing of transverse cracks that occur in CRCPs is perhaps the most important variable directly affecting the behavior of the pavement. Relatively large distances between cracks result in a higher accumulated drag force due to frictional resistance from the subgrade, thus producing high steel stress at the crack and large crack width. Closer crack spacing reduces the frictional restraint and, thus, the steel stress and crack width. It is clear that the crack spacing is directly related to other responses such as steel stress and crack width. Control of one will immediately affect the behavior of the others. In general, assuming adequate foundation support, closely spaced cracks in CRCP are desirable because the steel stress and the crack width will be small. However, it is commonly known that the major distress observed on in-service CRCP is punchout, which can be associated with the combination of closely spaced transverse and longitudinal cracking. An optimum design, therefore, calls for a balance in all of the structural responses in the continuous pavement.

Failures in CRCP are usually manifested as isolated areas of premature distress in different forms (according to environment) such as steel rupture, excessive spalling at the crack, edge pumping, and punchout. Among the distresses, some can be associated with poor subbase and drainage (these are outside of the scope of this report), although others can be linked directly to the above pavement responses. As stated earlier, punchout is associated with transverse crack spacing in the continuous pavement. Narrow crack spacing, when combined with crack deterioration, will force the beam action of the continuous pavement to act transversely instead of longitudinally. Transverse beam action will, in turn, cause longitudinal cracks to appear and eventual deterioration into punchout failure. Punchout, therefore, can be alleviated by controlling the crack spacing of the continuous pavement while maintaining adequate foundation support. Similarly, other failures, including spalling and steel rupture, can be controlled by tracing the origin of the distress mechanism and by assigning design criteria to the corresponding pavement responses.

Previous Design Criteria

Contemporary procedures for the design of CRCP are summarized in the AASHTO Interim Guide for the Design of Pavement Structures (6) and in the Texas State Department of Highways and Public Transportation Operations and Procedures Manual (7). These procedures are based on early developments in the modeling of CRCP behavior, and, as such, they restrict steel stress to values below yield. However, they do not consider other variables that have a significant effect on performance, such as crack width and spacing. More recent work (1,5) established newer design criteria for use with the computer program design approach. It is the purpose of this report to outline criteria for use in conjunction with the nomograph (regression equation) design techniques outlined in Noble, McCullough, and Ma (2).

DESIGN CRITERIA FOR CRACK SPACING

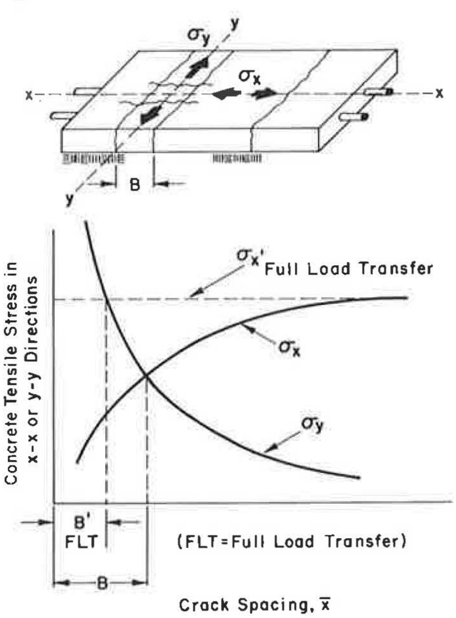
A CRCP can be simulated as a series of continuous beams resting on an elastic foundation. Transverse cracks develop as a result of frictional restraint of the slab against changes caused by shrinkage and temperature drop. Additional cracks due to bending in the longitudinal direction may develop when traffic loads are applied. As transverse crack spacing becomes relatively narrow and when load transfer at the crack deteriorates, the pavement structure no longer responds as a longitudinal beam. Rather, it responds as a transverse beam with stress in the transverse direction higher than that in the longitudinal direction. With further increase of fatigue loadings, longitudinal cracks crossing the transverse cracks will develop and eventually will deteriorate into punchout failure. One critical crack spacing, therefore, is the spacing at which the stress in the transverse direction becomes dominant.

Effect of Crack Spacing on Transverse and Longitudinal Stresses

The relation between crack spacing and stresses in the X-X and Y-Y directions is illustrated in Figure 1. Solid lines in the figure represent the relation for the condition of zero load transfer at the crack. For a crack spacing greater than B, the pavement slab acts as a longitudinal beam, and the stress in the X-X direction is more critical because it becomes larger than that in the Y-Y direction. The reverse is true for a crack spacing less than B, because the slab acts as a transverse beam. The spacing between cracks in the continuous slab can be thought of as the span length of a rectangular plate on an elastic foundation. Increase in crack spacing or span length will result in higher σ_x and lower σ_y. The increase in bending stress will gradually diminish as movement farther away from the midspan occurs--where the load was applied. The stress in the X-X direction remains constant after reaching the maximum level. The crack spacing B at the intersection of the σ_x and the σ_y curve is, therefore, the minimum allowable crack spacing for zero load transfer at the crack if σ_x is to control.

For full load transfer conditions, the pavement can be viewed as a continuous slab with no cracks. The σ'_x at the crack spacing under the full load transfer conditions, therefore, should be equal to the σ_x for an infinitely long slab. The horizontal dashed line in Figure 1 represents the stress in the X-X direction σ'_x for an infinitely long slab or one with full load transfer conditions at the cracks. It is obtained by drawing a line tangent to the point of maximum stress, which occurs when the slab length no longer influences the stress. The length B' is derived from the intersection of the σ'_x line and σ_y curve, and it represents the minimum allowable crack width for full load transfer. Thus, B' is the minimum crack spacing for full load transfer if σ_x is to control, and B is the minimum for zero load transfer as discussed above. In-service CRCP has a condition between these two extremes because it is closer to full transfer after construction and decreases with repeated load application.

Figure 1. Illustration of critical stress location as affected by crack spacing for a given set of conditions.



Effect of Stiffness Reduction at the Crack

Load transfer at the crack is possible through moment transfer, granular interlock, and dowel action of the steel reinforcement, assuming adequate foundation support. In field conditions, neither full nor zero load transfer at the crack are likely to be found. Theoretically, if the granular interlock and dowel action of the reinforcing bars are 100 percent efficient, half the applied load will be transferred across the crack to the adjacent slab. This is true only if the same amount of deflection occurs on both slabs and each assumes half of the applied load. However, considering a certain amount of debonding of the steel and looseness that develops in the aggregates under repeated loads, a further reduction in load transfer of between 5 and 10 percent can be assumed (8). Thus, the design load transfer due to aggregate interlock and dowel action of the steel should be 45 percent of the design load.

Under vertical load, deflection of the slab at the crack will cause the crack width to decrease. Moment transfer occurs only when the slab segments at both sides of the crack are in contact. The amount of reduction in handling stiffness at the crack depends on a combination of design variables. Abou-Ayyash and Hudson (9) studied the effect of transverse cracks on the bending rigidity of continuous pavement. Figure 2 (9) shows the result of the investigation in which the percentage reduction in bending stiffness at the crack is related to the concrete compressive strength and to the percentage of longitudinal reinforcement for a given set of environmental conditions.

Crack Effect on Allowable Crack Spacing

Assuming that a linear relation exists between the structural response of the slab as affected by the load transfer at the crack and the spacing between cracks, allowable crack spacing for cracks with various degrees of load transfer capacities can be predicted. For example, if

Figure 2. Variation of the percentage reduction in bending stiffness at crack loaction with longitudinal percentage reinforcement.

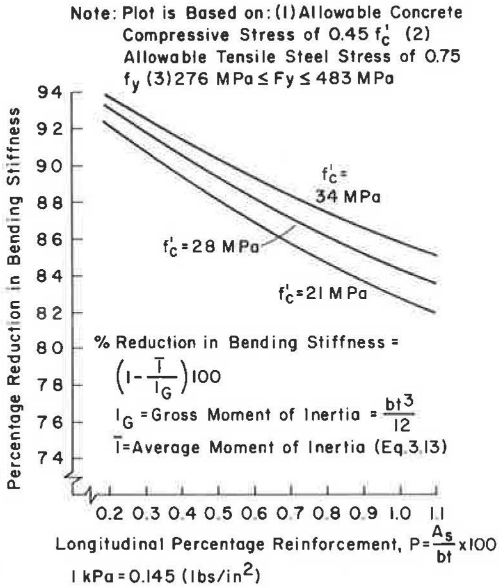
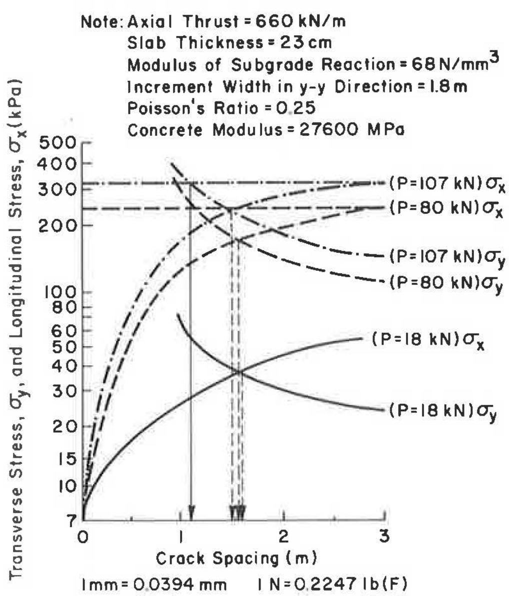


Figure 3. Variation of transverse and longitudinal concrete stresses with crack spacing for various axle loads.



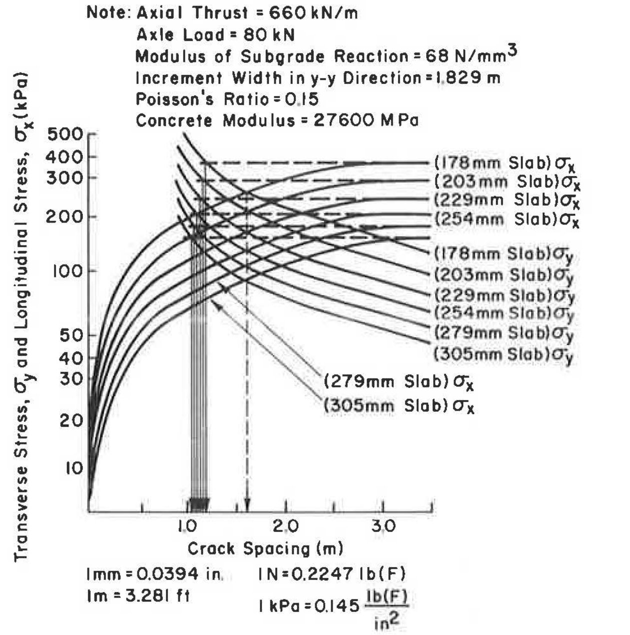
cracks in the slab can provide half of the structural integrity of the uncracked section, the critical crack spacing will be at the midpoint between B and B' in Figure 1.

Prediction of Allowable Crack-Spacing Range

Concrete Tensile Stress Condition

The SLAB-49 program (10,11) provides an excellent analysis tool for studying the effect of crack spacing on continuity and provides a basis for choosing a minimum crack spacing. The logic and procedures used herein are documented in the appendix to the work by Ma, McCullough, and Noble (3). In order to obtain a limiting (minimum) value for crack spacing, a factorial of representative values of the design

Figure 4. Variation in transverse and longitudinal concrete stresses with crack spacing for various slab thicknesses.



variables (3) was evaluated by using the SLAB-49 program. The magnitudes of the variables cover a broad range of slab thicknesses, axle loads, and crack spacings. Both longitudinal and transverse stresses with respect to crack spacing were computed (3) and plotted for various axle loads and slab thicknesses (Figures 3 and 4). The minimum allowable crack spacing B' is determined at the intersection of the σ'_x line and the σ_y curve. Allowance for reduced bending stiffness is made as indicated above (Figure 2).

Criterion of Bond Development Length

As discussed in Ma (1), the required length for full development of the bond between the reinforcing steel and the concrete in CRCP must be kept below a value equal to one-half the crack spacing. This bond development length, however, can be calculated in terms of the change in steel stress between the crack location and midspan (3), as movement occurs longitudinally down the concrete slab--that is,

$$\bar{x} \geq 2b \tag{1a}$$

and

$$b = (\Phi^2/38\sqrt{f'_c})(\sigma_{sc} - \sigma_{sm}) \tag{1b}$$

where

b = required bond development length,
Φ = bar diameter,
f'_c = concrete compressive strength,
σ_{sc} = steel stress at the crack, and
σ_{sm} = steel stress at midspan (3).

The maximum required value of b likely to be encountered in practice, then, is that for a low-strength concrete, a large bar diameter, zero steel stress at midspan, and a steel stress at the crack of just less than yield. If $\Phi \leq 19.1$ mm (0.75 in), $f'_c \geq 17\,240$ kPa (2500 psi), $\sigma_{sc} \leq 413\,700$ kPa (60 000 psi), and $\sigma_{sm} > 0$, then $b \leq 602$ mm (23.7 in) and $\bar{x} \geq 1.22$ m (4 ft).

Thus, because the maximum required length for full bond development is less than 0.61 m (2 ft), the minimum

Table 1. Effect of limited crack spacing on fraction of spalled cracks.

Maximum Allowable Crack Spacing (m)	Spalled Cracks (%)	Probability That < p Percent of Cracks Will Spall (%)
3.048	50	92
	40	86
	30	78
	20	58
2.743	50	93
	40	89
	30	79
	20	58
2.438	50	94
	40	90
	30	78
	20	61
2.134	50	94
	40	90
	30	80
	20	62
1.829	50	98
	40	96
	30	84
	20	66

Note: 1 m = 3.281 ft.

allowable crack spacing in this case is 1.22 m (4 ft). In general, this value may be used as a lower bound on crack spacing for all CRCP designs unless excessively large reinforcing bars--that is, 19.1 m (0.75 in)--are used in combination with very low-strength concrete [$f'_c \leq 17$ MPa (2500 psi)], which, of course, is very unlikely. However, in practice, the designer should calculate the lower bound on $\bar{x}$ peculiar to the design situation by using the procedure detailed above. In most cases, this value will be on the order of 0.9 m (3 ft).

Spalling (Condition Survey) Criterion

A scattergram of percent spalled cracks against crack spacing was plotted (3) by using data from the 1978 Texas CRCP condition survey (12). Based on this large sample of 212 observations taken from sections of CRCP all over the state, recommendations as to an upper bound on crack spacing can be safely made (Table 1). No allowance has been made here for regional or local variation because it is thought that separate estimates of the reliabilities for each district would not differ significantly from those listed in Table 1. From Table 1, it is clear that, if a designer wished to restrict the fraction of spalled cracks to less than 40 percent, this could be done with 90 percent confidence of restricting crack spacing to no more than 2.4 m (8 ft). However, if a designer wished to restrict this fraction further (for example, to less than 30 percent), the reliability of the design drops to 84 percent, even if the crack spacing were limited to no more than 1.8 m (6 ft).

Allowable Range

An allowable range of crack spacing can be obtained by any CRCP designer by choosing a maximum allowable value from the spalling criterion, along with a minimum allowable value from the criteria for concrete tensile stress and bond development length.

DESIGN CRITERIA FOR CRACK WIDTH

Design criteria for crack width are established from the standpoints of controlling both water flow and spalling. In considering the water flow problem, the design criteria are developed by limiting the permanent crack width for the continuous pavement. Because permanent crack width is related to the deformation of reinforcing steel at the crack, it will be discussed in both this section and the section on design criteria for steel stress.

Crack Width Criteria Based on Spalling Measurements

Spalling in CRCP

Spalling (i.e., minor or deflection spalling) is one of the distresses in CRCP. The primary causes for spalling are believed to be

1. Entrapment of road debris in cracks, which causes stress concentration when the cracks close as temperature increases;
2. Combined shear and tensile stress at joints or cracks due to horizontal temperature loading and vertical traffic loading; and
3. Poor material at surface due to overworking concrete during finishing.

Laboratory studies conducted by McCullough and others (5) indicated that spalling for CRCP caused by road-debris entrapment is relatively insignificant but that the combined horizontal and vertical forces produced by repeated loading seem to be the major contributors to spalling. Darter and Barenberg's study (4) on the ranking of major distresses in rigid pavements appears to corroborate McCullough's conclusions that combined horizontal and vertical forces are among the major contributors for jointed concrete pavement (JCP) and jointed reinforced concrete pavement (JRCP) as well as CRCP. Spalling occurred in 9 of 18 and 6 of 12 pavements surveyed. Because the reinforcement in both JRCP and CRCP exerts horizontal forces while resisting thermal or shrinkage volume change, higher concrete stresses generally occur in these pavements than in JCP. This action may contribute to stress concentrations that cause the spalling in these two types of pavements to be much more pronounced.

Correlation Between Crack Width and Spalling

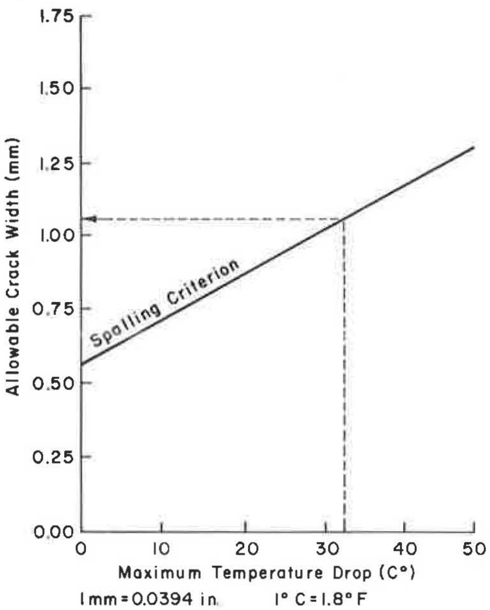
Horizontal stresses developed in CRCP can be correlated with design parameters, such as percent reinforcement, slab thickness, concrete modulus of elasticity, concrete strength, base friction, and thermal and shrinkage coefficients. A good indicator for the amount of horizontal stress in CRCP is the crack width. In general, crack widths are directly proportional to the magnitude of horizontal stresses.

The primary spalling mechanism identified by McCullough and others (5) was the combination of horizontal and environmental stresses and stresses resulting from vertical traffic loads. Because both crack width and degree of spalling correlate highly with the horizontal stress, they should theoretically also correlate with each other. In the diagnostic study based on condition surveys of CRCP in Texas (5), crack widths were measured in the field for a temperature range of 27°-32°C (80°-90°F). The results were plotted with respect to the general condition of spalling, and it was shown that spalling increases with increased measured crack width. The mean crack width was reported to be 0.538 mm (0.0212 in) for the spalled sections and 0.447 mm (0.0176 in) for the nonspalled sections. Spalling of cracks with less than 0.51-mm (0.02-in) widths was not observed. Similar results were obtained from a set of measurements taken in Illinois (5).

Maximum Allowable Crack Width

Only 5 percent of the pavements surveyed (4,5) experienced spalling at crack widths of less than 0.61 mm (0.024 in). The 0.61-mm level is, therefore, used as the basis in the determination of the design criteria for crack width based on spalling discussed in this paper. This was confirmed when a similar value was obtained following analysis of the Illinois data (5). Note that the crack widths measured in the field surveys are temperature dependent, although the spalling occurred over a long period of time during which the pavement temperature varied widely. Accordingly, the crack width varied over a wide range of values during this

Figure 5. Variation of allowable crack width with temperature.



period. The curve labeled spalling criterion in Figure 5 characterizes this variation for the range of temperatures applicable to the surveyed pavements. Hence, Figure 5 must be used in the design process described below to determine the allowable crack width for minimum temperature. First, we need to calculate the value of temperature drop in the pavement when the crack width of 0.61 mm was measured; then, by using back calculation, the critical crack width for spalling under maximum temperature drop can be found. A section of CRCP under environmental conditions similar to those of the pavements surveyed, with a crack width equal to 0.61 mm, has been used to back-calculate the critical crack widths for various temperature drops. Ma (1) and Dhamrait, Jacobsen, and Schwartz (13) describe the theoretical approach used for the calculation of the allowable crack widths for various temperature drops for the surveyed sections, with the maximum drop approximated by a mean of 33°C (60°F). Thus, the limiting (maximum) value of crack width in the CRCP recommended is 1.07 mm (0.042 in), as indicated in Figure 5. This value would then be compared with a value based on other limiting criteria and the more conservative value used in the design.

Crack Width Criteria Based on Steel Corrosion and Subgrade Erosion (Permeability) Restrictions

Corrosion in CRCP

As recognized, the purpose of steel reinforcement is to limit the crack width to a level that will (a) provide adequate load transfer, (b) control spalling, and (c) avoid excessive water percolation and, subsequently, prevent subgrade erosion and steel corrosion. The design criterion for the crack width discussed previously has already put limits on the width of the pavements' cracks in line with objectives (a) and (b).

In considering the problem of water percolation, refer to the study conducted by McCullough and others (5), which also presents the results of the research described in this paper. The relation between various crack widths and time required for water to reach different depths in the crack was plotted. Assuming a water depth of 19.1 mm (0.75 in) and a 3.7-m (12-ft) wide pavement section, the time required for water to flow across a CRCP section (for various cross slopes) can be calculated by using Manning's formula for open channel flow. Then,

$$V = (1.49/N)(R^{2/3} S^{1/2}) \quad (2)$$

where

V = mean velocity,
n = approximately 0.016 for rough concrete,
R = hydraulic radius, and
S = slope of channel bed.

The time required for water to percolate to various depths and the cross-pavement flow times were superimposed on the above plots; it was shown that a crack width of less than 0.25 mm (0.01 in) can prevent water from reaching the subgrade. In that same study (5), it was found that, for a crack width of less than 0.25 mm, virtually no rusting of the steel developed. Similarly, a study conducted by the Illinois Department of Transportation (13) also supports this observation. It was found that a crack width equal to or greater than 0.20 mm (0.008 in) has a greater potential for the occurrence of significant rusting of the reinforcing steel. Ideally, based on these studies, the steel reinforcement would be designed to control the crack width to a level of less than 0.25 mm under the most critical situation (i.e., when the temperature is lowest and the pavement is flooded). However, to design for such a criterion is highly impractical, because to keep crack width at such a level will require an exorbitant amount of steel and will cause excessive cracking. Also, such a restriction is unnecessarily conservative, because this most critical situation occurs for only a small fraction of each year of pavement life. Consequently, by using the procedure discussed in the following paragraph, the designer should be able to choose the maximum crack width that is within a sensible range of values and yet keep the steel corrosion caused by any water that may reach the steel down to an acceptable level.

Maximum Allowable Crack Width

McCullough and others (5) relate the quantity of flow of water into the crack (permeability in gallons per minute per inch of crack as determined by measured headloss in ponded water) to crack width and to degree of steel corrosion and subgrade erosion. The permeability of cracks below the 0.25-mm level is really quite small (resulting in minor corrosion only), but the permeability of cracks and associated corrosion between the 0.25-mm and the 0.64-mm (0.025-in) levels are only slightly larger. However, above the 0.64-mm level, the cracks are extremely permeable, with substantial quantities of water flowing into the pavement and subsequent heavy corrosion and subgrade erosion occurring. Accordingly, for design purposes, if the pavement were to be continuously flooded and kept constantly at a temperature just above freezing, crack width would have to be kept below the 0.64-mm level.

Yet, because neither of these two extreme conditions is likely to occur constantly throughout the entire life of the pavement, the value of 0.64 mm should be adjusted accordingly. By examining the distribution of maximum daily temperature drops from curing temperature in any one year, the value of temperature drop from curing that will not be exceeded a chosen fraction of the time (usually 95 percent) may be calculated. The technique for calculation of change in crack width with change in temperature, which was discussed for the spalling restriction, should be applied. This would involve preparing a chart similar to Figure 5. It is important to note that the designer should obtain climatological data, a temperature distribution, and a crack width-temperature plot appropriate to the environment of the particular pavement being designed.

Maximum Crack Width for Design

The lower of the two maximum allowable crack widths, as recommended by the spalling and permeability restrictions, should be chosen as the design maximum crack width.

DESIGN CRITERIA FOR STEEL STRESS

Two criteria are used to define the allowable steel stress in CRCP. First, the steel stress must be lower than its ultimate tensile strength divided by a safety factor. This criterion is to safeguard against rupturing of the steel under high tension. Second, if the steel stress is to be greater than yield, permanent crack width associated with the permanent deformation of steel at the crack must be less than the allowable amount to avoid excessive water percolation.

Criteria for Steel Rupture

To guard against rupturing of the steel, the allowable stress in steel is set to be less than ultimate strength times a safety factor of 0.75. Table 2 (6) shows the ultimate strength for various types of deformed bars and their allowable stress against rupture.

Criteria for Permanent Deformation

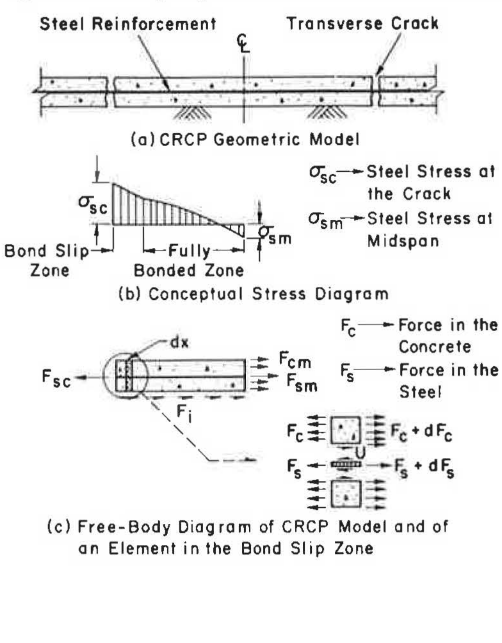
Conventional design criteria for steel stress generally require that the stress be less than the yield strength times a safety factor. Such criteria prevent the steel from undergoing plastic deformation. Based on our experience, however, we know that many kilometers of CRCPs have been performing adequately although their steel stresses are predicted to be higher than yield. This prompts us to consider the adequacy of such steel stress limits as criteria by evaluating the response of steel reinforcement in the CRCP, when stressed beyond the elastic range. Less-conservative criteria can then be obtained by evaluating the maximum stress in the steel in terms of its permanent deformation, which is equal to crack width at the point of maximum stress. The maximum allowable steel stress is thus calculated by keeping the crack width below some suitable value.

Table 2. Maximum allowable steel stress to prevent rupture in CRCP.

Steel and Grade Type	Minimum Yield Strength, f_y (MPa)	Ultimate Strength, f_u (MPa)	Allowable Stress, f_s (MPa)
Billet			
Grade 40	276	414	362
Grade 60	345	621	465
Grade 75	690	690	517
Rail			
Grade 50	345	552	414
Grade 60	414	621	465

Note: 1 MPa = 145 psi.

Figure 6. Free-body diagram and stress distribution in CRCP model.



Evaluation of Permanent Deformation of Steel

Plastic deformation of steel in CRCP can be determined by multiplying the plastic strain at the crack by a defined gauge length. The gauge length at which the steel undergoes plastic deformation can be approximated as the length of the region in the bond slip zone where steel stress is above yield. To estimate the gauge length, it is necessary to review the basic CRCP model.

Figure 6 shows a steel stress distribution diagram for a CRCP section under the effect of volumetric change. At the crack, because concrete provides no resistance, the steel tension is at maximum. Moving away from the crack, a decreasing amount of tension force will be carried by the concrete, thus reducing the tensile stress in the steel. The rate of change in stress or the slope of the stress diagram at the bond slip zone depends on the bond strength between the steel and the concrete. The rate of change in steel stress can be determined by summing the forces acting on the steel bar. From Figure 6c at the bond slip zone, $F_S + dF_S = F_S + Udx$; by combining terms and solving for U, $U = dF_S/dx$, where U = average bond force per unit length of bar. The average bond force may also be expressed as

$$U = UE_o \tag{3}$$

where

u = shear strength = $9.5\sqrt{f'_c}/\Phi$,
f'_c = concrete compressive strength,
E_o = bar perimeter, and
Φ = bar diameter.

By equating the expression that solves for U and Equation 3 for average bond force and converting to stress, the following are obtained:

$$\Delta_s d\sigma_s/dx = uE_o \tag{4a}$$

$$(\pi\Phi^2/4)\cdot(d\sigma_s/dx) = u\pi\Phi \tag{4b}$$

$$d\sigma_s/dx = 4u/\Phi \tag{4c}$$

Knowing the slope of the steel stress diagram at the bond slip zone, the gauge length ℓ can be estimated as

$$\ell = 2\cdot[(\sigma_{sc} - \sigma_{yield} \times SF)/d\sigma_{s/dx}] \tag{5a}$$

$$\ell = 2[(\Phi/4u)(\sigma_{sc} - \sigma_{yield} \times SF)] \tag{5b}$$

$$\ell = (\Phi^2/19\sqrt{f'_c}) \times (\sigma_{sc} - \sigma_{yield} \times 0.75) \tag{5c}$$

By approximating the plastic strain E_p to be σ_{yield}/E_s, the amount of permanent deformation Δx in the steel becomes

$$\Delta x = 2\ell(\sigma_{yield}/E_s) = (\Phi^2/19\sqrt{f'_c})(\sigma_{sc} - \sigma_{yield} \times 0.75)(\sigma_{yield}/E_s) \tag{6}$$

Prediction of Allowable Steel Stress

For permanent deformation of less than 0.25 mm (0.01 in), the maximum allowable steel stress at the crack can be obtained by setting Δx in Equation 7 equal to 0.25 mm so that

$$0.25 = (\Phi^2/19\sqrt{f'_c})(\sigma_{max} - \sigma_{yield} \times 0.75)\cdot(\sigma_{yield}/E_s) \tag{7a}$$

thus,

$$\sigma_{max} = (19 E_s \sqrt{f'_c}/\Phi^2 \sigma_y) + (\sigma_y \times 0.75) \quad (7b)$$

where σ_{max} = allowable steel stress and σ_y = steel yield stress.

Table 3 summarizes the maximum allowable steel stress for various bar diameters and steel yield strengths for low-strength [$f'_c < 24$ MPa (3500 psi)] and regular-strength [24 MPa (3500 psi) $\leq f'_c$] concretes.

Maximum Steel Stress for Design

The limiting value on steel stress to be used in design should be chosen as the lower of the maximum allowable steel stresses recommended in Tables 2 and 3. That is, the maximum recommendation from the steel rupture and permanent deformation criteria should be used.

USE OF LIMITING CRITERIA

The results from this study are presented in this paper and in Ma, McCullough, and Noble (3), which are to be used in conjunction with Nomographs for the Design of CRCP Steel Reinforcement (2). In that report, the relationship between the significant input variables and the structural responses predicted by the CRCP-2 model is quantified by using regression techniques and is expressed as a set of nomographs. This set of design charts permits graphic prediction of the final responses of the pavement to the total load. Crack spacing, crack width, and steel stress are predicted. It should be noted that the CRCP-2 model only simulates the loading conditions of environmental force and bending stress under application of a single wheel load. It should also be noted that fatigue cracking caused by the combination of repetitive wheel loads and reduction of tensile strength due to fatiguing of the concrete material was not considered in Noble, McCullough, and Ma (2). However, it is proposed to treat fatigue in the design process by following the procedure for CRCP thickness and reinforcement design outlined in that report and summarized here. The major steps in this design process are as follows:

1. Determine the design slab thickness on the basis of fatigue analysis alone (3, section on Guidelines for Selection of Design Input Variables);

Table 3. Maximum allowable steel stress for control of permanent deformation.

Steel Yield Strength, f_y (MPa)	Steel Bar Diameter, ϕ (mm)	Maximum Allowable Steel Stress (MPa) and Concrete Compressive Strength, f'_c (MPa): Low[a] (f'_c < 24 MPa)	Regular[b] ($f'_c \geq 24$ MPa)
276	13	415	447
	16	340	378
	19	299	326
	22	275	294
345	13	425	474
	16	365	396
	19	332	354
	22	313	329
414	13	449	489
	16	399	425
	19	371	390
	22	356	369
517	13	498	531
	16	459	479
	19	402	452
	22	390	434

Note: 1 MPa = 145 psi; 1 mm = 0.0394 in.

[a]Low = ≥4.5 percent air content or ≤4 cement sacks/yd^3 concrete.
[b]Regular = ≤4.5 percent air content and >4 cement sacks/yd^3 concrete.

2. By using this slab thickness and chosen values of the other trial input variables, predict the final crack spacing, crack width, and steel stress by means of the nomographs in Noble, McCullough, and Ma (2);
3. Check the predicted responses by means of the design criteria established in this paper and in Chapters 3, 4, and 5 of Ma, McCullough, and Noble (3);
4. If the predicted responses exceed the allowable criteria, lower or raise the level of design variables according to the general behavior of the CRCP as discussed in this paper and in Chapter 2 of Ma, McCullough, and Noble (3); and
5. If changes in input variables involve a change in slab thickness or concrete flexural strength, repeat step 1.

CONCLUSIONS AND RECOMMENDATIONS

By establishing values for input variables by means of the procedures discussed above, the design of steel reinforcement for a CRCP can be performed by following the procedure that is outlined in Chapter 5 of Noble, McCullough, and Ma (2). Limiting criteria are also to be used in this process as described above and in Chapters 3, 4, and 5 of Ma, McCullough, and Noble (3).

The limiting criteria on crack spacing, crack width, and steel stress discussed in this report represent part of the only national, comprehensive, and easy-to-use procedure for the design of CRCP available at this time. It is strongly recommended, therefore, that the entire procedure summarized in this paper and detailed elsewhere (2,3) be incorporated into appropriate CRCP design manuals as soon as possible.

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Nomographs for the Design of Steel Reinforcement in Continuously Reinforced Concrete Pavement

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This study sought to develop graphic procedures (nomographs) for the design of continuously reinforced concrete pavement (CRCP) by the Texas State Department of Highways and Public Transportation for a range of specified local conditions. This set of nomographs, when used as a supplementary design tool with the CRCP-2 computer program model, will facilitate CRCP design. This will substantially reduce both the time and the cost involved in the design process, while at the same time taking into account the effect of regional and local environments. First, regression equations were developed for the prediction of three design parameters (crack spacing, crack width, and steel stress), and then principles of nomography were applied to these mathematical relations to prepare three corresponding nomographs. The choice of equations was made following multiple linear and nonlinear least-squares fits to a fractional factorial of simulated observations that were output from the CRCP-2 computer program. Theoretical models, developed at the Center for Highway Research in Austin, Texas, and variations of the three design parameters with each of the relevant input variables over the range of the simulated data were considered in deciding on the form of the regression equations. Standard-error-of-residuals and R^2 (proportion of variance explained by the regression equation) statistics were considered in the final choice of coefficients for the regression equations. Confidence prediction limits were determined by using multiple linear-regression techniques for application to nomograph predictions. A recommended procedure for the use of the nomographs with appropriate limiting criteria is outlined and an example given.

Continuously reinforced concrete pavement (CRCP) is considered a relatively new pavement type by many engineers, although it has been in use since 1921, when it was first introduced by the Bureau of Public Roads on the Columbia Pike near Arlington, Virginia. The next reported use of CRCP was in 1938, when Indiana, in cooperation with the Bureau of Public Roads, constructed an experimental pavement that involved several test sections.

The state highway departments of Indiana, Illinois, Texas, California, Mississippi, New Jersey, Michigan, Maryland, and Pennsylvania have laid other pavements of this type that have provided good service for a number of years. The oldest of these is approximately 30 years of age.

After there were several successful experiences with CRCP on experimental projects, the use of CRCP increased substantially, especially during the 1960s. Several research studies in rigid pavement design led to the development of the design procedures currently used for CRCP (1-5).

In 1972, a study under the auspices of the National Cooperative Highway Research Program (NCHRP) was conducted at the University of Texas at Austin. It comprised a review of design and construction variables, theoretical studies, field surveys, and laboratory investigations. The fundamental philosophy of this review was that, through a combination of field observations and laboratory studies, reliable procedures could be achieved to develop mathematical models that simulate CRCP field performance. Based on these mathematical models, the CRCP-1 computer program was developed to calculate the stresses in concrete and steel, crack width, and crack spacing that result from concrete volume changes due to temperature and shrinkage (6).

Generally, the engineer is encouraged to design each pavement for the soil conditions, traffic, materials, and so forth at the given site and to be wary of inappropriate boundary values and practices. However, in order to cover such a wide variety of input variables, the engineer needs a large-scale experiment to anticipate the effects of the individual variations of the variables and the variations in groups. Thus, a sensitivity analysis of the behavior of CRCP that used the CRCP-1 model (7) was conducted for the Texas State Department of Highways and Public Transportation (SDHPT). From the results of this study, the relative importance of about 15 input variables was determined in order to investigate the effect of changes in values of these variables on CRCP behavior. The list of the input variables includes steel properties, concrete properties, friction-movement relations, and temperature variations. In addition to establishing the relative importance of such variables, the study revealed several inconsistencies in the initial model at extreme boundary conditions that resulted in modification of the computer program.

The next step was to include the effect of wheel-load stresses on crack-spacing history. The NCHRP study found that heavy volumes of 18-kip (80-kN) single-axle loads resulted in reduced crack spacings (6). The study of the effect of wheel-load stress on pavement behavior and its interaction with the other input variables is discussed in Ma and McCullough (8), which describes the development of the CRCP-2 model. This development process is outlined in flowchart form in the upper part of Figure 1.