DESIGN PROCEDURE FOR CRCP BASED ON LABORATORY AND FIELD OBSERVATIONS

B. Frank McCullough
The University of Texas
Austin, Texas

SUMMARY

In this paper a CRCP design procedure is presented along with the necessary limiting criteria. The procedure is derived from theoretical and empirical models developed in connection with numerous field and laboratory studies. The first basic concept is to provide adequate longitudinal steel considering the specific conditions of a project to achieve a desirable crack width, crack spacing, and steel stress. These limiting criteria are shown in the report. A check is also made to limit spalling and other distress manifestations from occurring. The AASTO Guide for Design of Pavement Structures is recommended for pavement thickness design.

INTRODUCTION

In connection with NCHRP Research Project 1-15 "Design of Continuously Reinforced Concrete pavements for Highways," a design procedure for CRCP was developed. In this paper, the results of that study are used to derive a comprehensive rational step-by-step manual design procedure that does not require the use of a computer.

First, limiting design criteria are given for crack width, crack spacing, and steel stress. Next, a design procedure is given for determining the percent longitudinal steel considering the limiting criteria. The pavement thickness is determined using the AASTO procedure. A check is next made to insure that excessive spalling and other distress manifestations are not developed.

CRITERIA

In this section, limiting criteria for use in design and construction of CRCP Design Criteria are presented for crack width, crack spacing, material strength, and other related areas based on studies on in-service pavements.

Crack Width

Data from both laboratory and field studies provide an insight as to the limiting crack width from both a water flow and spalling standpoint. Reference 1 shows that the water flow through a crack commenced to increase at a width of 0.010-inch and that a crack width of 0.023-inch, a drastic increase of water flow was experienced. Although information is not available to give a quantitative measure as to what is good and bad with respect to corrosion, we may qualitatively assume that the gross increase in flow beyond 0.023-inch is an order of magnitude worse than the lower rate between 0.010 and 0.020-inch. Thus, a value of 0.023-inch may be used as a limiting crack width from the standpoint of water flow.

Crack Spacing

An ideal crack pattern would be one with a uniform spacing. The crack frequency diagrams presented in Refs 1 and 4 show this is seldom achieved in the field. Furthermore, an analysis of the data shows that most of the slab segments are less than 3 feet in length. Many times distressed areas in a section are located where the crack spacing is one to two feet. Thus, too small a crack spacing is an undesirable feature. A recent study by the Illinois Department of Transportation indicated that a crack spacing of 4 to 5 feet minimized steel rusting (Ref 2). McCullough (Ref 5) in a deflection investigation on in-service CRCP found that the optimum load transfer characteristics (minimum deflection) occurred with a crack spacing range of 5 to 8 feet. Abou-Ayyash et al (Ref 6) found that the minimum load stresses were achieved in this range also. A recent stress analysis by Sathaport (Ref 7) showed a substantial increase in deflection occurred with crack spacings of less than 30 inches. Thus, an overstressed condition may occur in the lower layers, resulting in a secondary failure in the pavement layer.

From these data, it is recommended that designers try to achieve a crack spacing of 5 to 8 feet with most conventional aggregates. Crack spacing greater than 8 feet result in wide cracks, and thus, a loss in load transfer and then spalling. With special cements or low thermal coefficient aggregates, this value may be increased. For less than the lower range, it is hypothesized that higher deflections may also occur, due to a loss of concrete bond along the steel bar. Thus, the reinforcing bar will not act as an integral unit.

Material Strength

Generally for design of CRCP, it is recommended that the allowable stress values for concrete and steel be as outlined in the AASHO Interim Guide for Rigid Pavements (Ref 8). An additional criterion is to have a minimum concrete tensile strength of 400 psi or greater to reduce surface spalling of the concrete.

CONSTRUCTION

Condition surveys of in-service pavements indicated a low incidence of failures associated with honeycombed concrete and transverse construction joint problems (Ref 1). Obviously, the practices instituted in the early 60's, after numerous of these type failures were observed, have been successful in preventing this type of distress. Thus, good concrete vibration practices should be retained as a minimum.

Comparison of steel placement types in Ref 1 provides information for specifications and construction procedures. Realistic tolerances can be initiated using that information as guidelines. Furthermore, the data show several types of steel placement techniques can give satisfactory results, but each one has problems of placing too high or too low in the slab. Since performance studies have revealed the steel position has a profound influence on crack spacing, frequent checks should be made during construction.

Bond breakers can be very effective in minimizing reflection cracking (longitudinal), but do not influence transverse spacing. Caution should be used in obtaining the final texture of subbase since a smooth condition may result in excessive laterial movements.

The time from construction to opening to traffic has very little effect on future performance once the minimum specification time limits are met.

DESIGN PROCEDURES

In this section, a design procedure for CRCP is outlined, based on the development and findings of various studies. Initial procedures for design of longitudinal steel were simple expressions derived on the basis of gross assumptions in an attempt to model a

very complex phenomenon. In this section, we have attempted to utilize information from recent research studies to produce a more reliable design procedure.

In the past, design procedures have always separated the reinforcement steel and pavement thickness design into independent functions. The basic assumption has always been that the cracks are held tightly closed, the slab will behave as a unit in a continuous manner. Thus, the prime influence of the reinforcement was to hold the cracks tightly closed so that this condition would exist. Previous studies have indicated that there is a reduction in the pavement deflection with an increase in reinforcing steel, but these data are not quantitative and cannot be used in a design procedure. Thus, the design procedure proposed in the following section still adheres to the basic principle separating the reinforcing steel and pavement thickness design, but as a result of recent studies, better design criteria are available and more sophisticated techniques for accomplishing this design are presented.

In the following sections, a step-by-step procedure is outlined for longitudinal steel reinforcement design and a reliable method for predicting pavement thickness is indicated. It is proposed that the subbase design criteria proposed by the NCHRP Report 128 (Ref 9) and the AASHO Interim Design Guide for Rigid Pavements (Ref 8) be used.

LONGITUDINAL STEEL REINFORCEMENT DESIGN

Figure 1 is a flow chart outlining the proposed design procedure for reinforcement steel. In each of the procedural boxes, the step number of the procedure is indicated. Note that the procedure is an iterative process requiring many trial and error steps. Computations of average crack width, average crack spacing, the state of stress, and the distress manifestations are made and checked against the "limit criteria" presented earlier.

The input data may be considered in three basic categories, i.e., environmental variables, fixed variables, and designer options. The values for the first two categories may be considered as fixed parameters, and hence the designer has no tolerance for adjustment in design. The flexibility comes with the designer options, such as percent longitudinal steel, steel type, and steel spacing. Concrete shrinkage may also be considered as a designer option since the revision in the curing procedure, cement factor, etc. may alter the shrinkage history, thus affecting the crack spacing, crack width, and state of stress in steel and concrete.

Step One

This step consists of predicting the average crack width, average crack spacing, and the state of stress using Figs 2, 3, and 4, respectively.

Step Two

This step consists of using the ouput from Step One to check against the limited criteria as shown by the following equations:

$$\hat{\Delta X} \leq 0.023 \text{ inch} \quad \text{(Eq. 1)}$$

$$5 \text{ ft} < \hat{\bar{X}} < 8 \text{ ft}$$

where

$\hat{\Delta X}$ and $\hat{\bar{X}}$ are as defined previously (Eq. 2)

and

$$\sigma_p \leq \sigma_s \quad \text{(Eq. 3)}$$

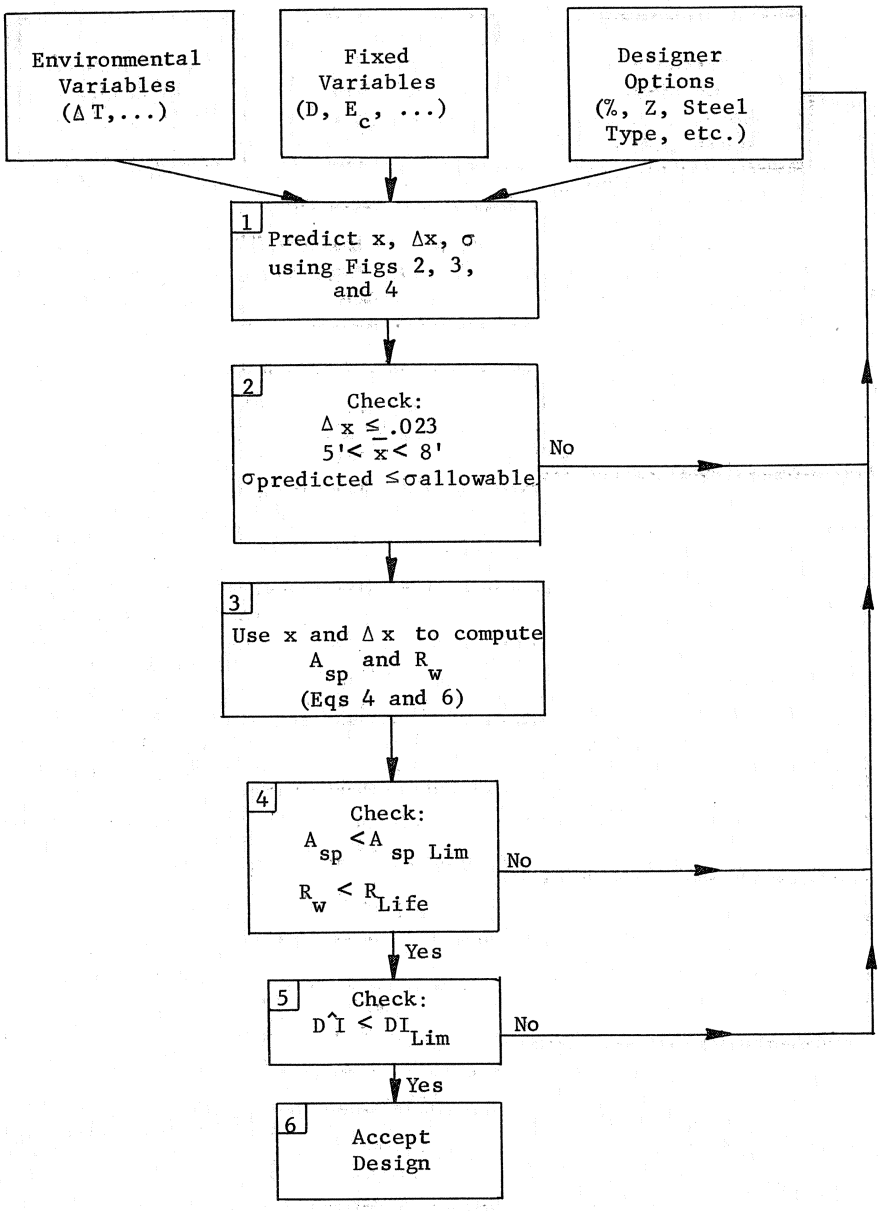
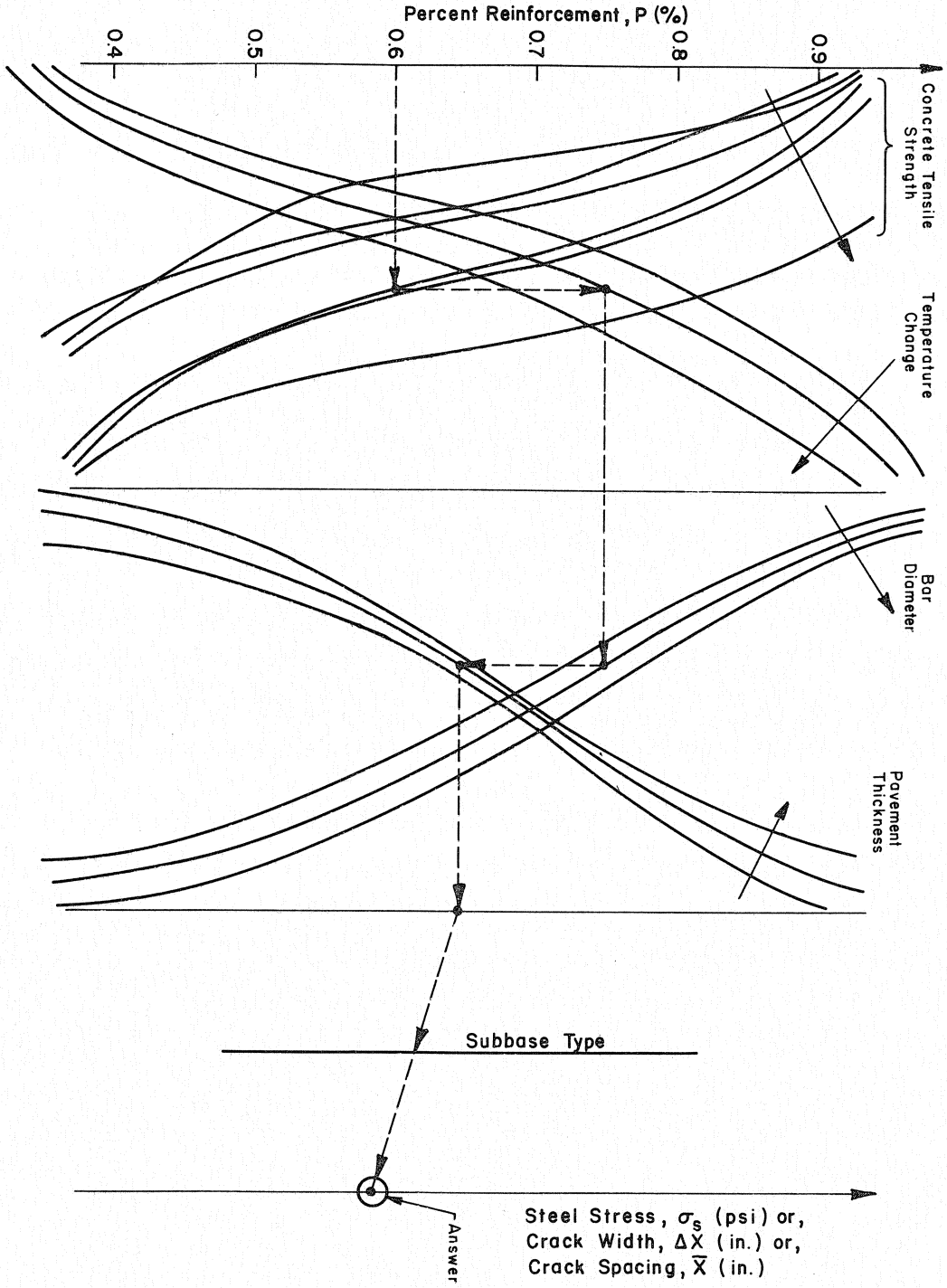


Fig 1. Steel reinforcement design procedure for CRCP.



Figs (2,3)&4. Prediction of steel stress, or carck width or crack spacing.

where

σ_p = predicted stress in the longitudinal reinforcing steel,

σ_s = allowable working stress in the steel.

Limiting criteria of crack width and crack spacing are based on the information presented previously. Note that a limiting criterion is not applied to concrete stresses since it is not an independent function. The magnitude of the concrete stress influences the crack spacing and crack width, and, hence, it is considered indirectly by the procedure.

If the predicted values of crack spacing, crack width, and steel stress do not satisfy the limiting criteria, the designer must return Step One and change the inputs in the designer option category.

Step Three

This step consists of predicting the distress manifestations of spalling and loss of load transfer. The spalling is computed using Eq 7. This is an areal phenomenon that must be converted to crack length.

The loss in load transfer is derived using the computed deflection as a function of load application. Computations of the deflection are made for the first load application and for the number of cumulative 18-kip equivalent axle loads expected at the end of the design period. These values are substituted into the following equation to obtain the percent loss in load tranfer in the life of the facility:

$$R_w = \frac{\hat{W}_f - \hat{W}_o}{\hat{W}_o} \quad \text{. . . . . (Eq. 4)}$$

where

R_w = ratio of increased deflection during the life of the facility,

$\hat{W}_f$ = predicted deflection at the end of the design period, inches,

$\hat{W}_o$ = predicted deflection at the start of the design period, inches.

Step Four

This step consists of checking the output of Step 3 against the limiting criteria for spalling and the increase in deflection. Limiting criteria for spalling are not available, and thus the designer must develop his own input. If several in-service CRC pavements are available for observation by the designer, it is recommended that the designer observe them and rate each as to whether it is acceptable or unacceptable from the spalling standpoint.

The limiting criteria for increased deflection may be developed by establishing a tolerance to the predicted loss of facility service life due to increasing deflection with load applications. Using the measured edge deflection and a predicted number of load applications from the AASHO Road Test, an expression can be derived to predict the increased loss of life in terms of 18-kip single-axle load applications. This is quantified by the following expression:

$$R_{Life} = \frac{W_{2.5} - W_{2.5'}}{W_{2.5}} R_w^{3.15} \quad \text{. . . . . (Eq. 5)}$$

where

R_{Life} = ratio of reduced 18-kip axle load applications due to the increase in deflection,

$W_{2.5}$ = cumulative number of equivalent 18-kip single-axle load applications to a psi of 2.5 for W_o, and

$W_{2.5'}$ = cumulative number of equivalent 18-kip single-axle load applications to a psi of 2.5 for W_f.

The designer should establish a practical tolerance level of reduced load applications for a facility since a thicker pavement will be required to carry the traffic if an excessive loss of load transfer is permitted. The above equation gives a severe design condition, since the life is computed with the deflection at the end of the service life, whereas the loss in load transfer is a progressive phenomenon occurring over the life of the facility. Note that the actual reduction factor should be somewhat lower, but since this is a design procedure, a more conservative approach is used.

If the predicted values of spalling and loss of load transfer do not satisfy the limiting criteria, the designer must return to Step One and change the input values for the designer option category.

Step Five

This step consists of checking a predicted distress index from Eq 6 against a limiting value. As was the case for the spalling index, the designer should use in-service pavements to compute this limit.

If the predicted value is greater than the limiting criterion, the designer must return to Step One.

Step Six

If the requirements of Step Four are satisfied, then the design may be considered as an acceptable one. As steel is placed at positions other than mid-depth, the predicted crack width may be empirically corrected using the field data presented in Ref 1 or a ratio of the expression from Eq 6.

PAVEMENT THICKNESS

The diagnostic study presented in this report indicated that the revised formula presented in the AASHO Interim Guide for the design of rigid pavements (Ref 8) may be used with a high degree of reliability to predict the required thickness of the pavement. Although the types of distress are slightly different, the equation does a fair job of predicting service life. This statement is based on the fundamental assumption that adequate load transfer is provided. The problems due to poor construction procedures, i.e., inadequate vibration, improperly placed steel, faulty concrete, etc., are assumed to be non-existent.

PREDICTION OF DISTRESS MANIFESTATIONS

The performance of continuously reinforced concrete pavements is influenced by a significant number of design and construction variables. In Ref 1, pavement performance was characterized by the following models:

(1) model to predict distress index from field data,

(2) model to predict percent of slab area spalled, from laboratory data.

In the following sections, a brief description of these models is presented.

Distress Index

As discussed thoroughly in Ref 1 of this report, a weighted distress index was developed from the various distress manifestations observed on in-service CRCP. A regression model is available from Ref 1 to predict the distress index as a function of 29 independent variables. The results of this regression study yielded the following function:

$$\text{Distress Index} = f(\text{concrete strength, pavement thickness, coarse aggregate type, curing, etc.}) \quad \text{(Eq. 6)}$$

Spalling

The following is a conceptual regression equation established from test slabs reported in Ref 1.

$$A_{sp} = f(\text{percent steel, depth of steel, number of cycles, coarse aggregate type}). \quad \text{(Eq. 7)}$$

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