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16. Abstract

There is a strong correlation between the structural responses and the frequency of distress in CRCP. Hence, the primary factors to be considered in the design of continuously reinforced concrete pavement are the structural responses: crack spacing, crack width, and steel stress.

The structural responses in CRCP are the outcome of the interactions among material characteristics, environmental conditions, and traffic loading. Since at least three-quarters of the volume of concrete is occupied by aggregate, concrete properties, such as thermal coefficient, tensile strength, modulus of elasticity, and drying shrinkage, are different for concrete made with different types of coarse aggregates. Hence, even under identical environmental and traffic loading conditions, diverse structural responses will result from using different coarse aggregates. In this study, the CRCP Design Standards were evaluated for concretes made with various coarse aggregates.

Two types of steel reinforcement are used in continuously reinforced concrete pavement construction: deformed bar and deformed wire fabric. Different structural responses are expected due to different structural mechanisms between the two reinforcement types. The Design Standards for two reinforcement types were separately evaluated.

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EVALUATION OF PROPOSED TEXAS SDHPT DESIGN STANDARDS FOR CRCP

by

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Research Report 472-1

Research Project 3-8-84-472
Rigid Pavement Data Base

conducted for the

Texas State Department of Highways
and Public Transportation

in cooperation with the

U.S. Department of Transportation
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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report presents an evaluation of the proposed Texas State Department of Highways and Public Transportation (SDHPT) CRCP Design Standards. The theoretical model used is the CRCP computer program. The behavior of CRCP with various coarse aggregate types was also studied. The study was carried out as a part of Research Project 3-8-84-472, "Rigid Pavement Data Base," which is sponsored by the Texas State Department of Highways and Public Transportation.

Our thanks are extended to Mr. James L. Brown and Mr. Jerry Daleiden, of the SDHPT, for their valuable advice. We express our appreciation to the staff of the Center for Transportation Research of the University of Texas at Austin.

Mooncheol Won
B. Frank McCullough
W.R. Ronald Hudson

LIST OF REPORTS

Report No. 472-1, "Evaluation of Proposed Texas SDHPT Design Standards for CRCP," by Mooncheol Won, B. Frank McCullough, and W. R. Hudson, presents the results of evaluation of proposed CRCP Design Standard for various coarse aggregates used, describes the theoretical models used in the study, and discusses several important design parameters in CRCP. April 1988.

ABSTRACT

There is a strong correlation between the structural responses and the frequency of the distress in CRCP. Hence, the primary factors to be considered in the design of continuously reinforced concrete pavement are the structural responses: crack spacing, crack width, and steel stress.

The structural responses in CRCP are the outcome of the interactions among material characteristics, environmental conditions, and traffic loading. Since at least three-quarters of the volume of concrete is occupied by aggregate, concrete properties, such as thermal coefficient, tensile strength, modulus of elasticity, and drying shrinkage, are different for concretes made with different types of coarse aggregates. Hence, even under identical environmental and traffic loading conditions, diverse structural responses will result from using different coarse aggregates. In this study, the CRCP Design Standards were evaluated for concretes made with various coarse aggregates.

Two types of steel reinforcement are used in continuously reinforced concrete pavement construction: deformed bar and deformed wire fabric. Different structural responses are expected due to different structural mechanisms between the two reinforcement types. The Design Standards for two reinforcement types were separately evaluated.

KEY WORDS: Continuously Reinforced Concrete Pavement(CRCP), Coarse Aggregates, CRCP-2, Deformed Bar, Deformed Wire Fabric, Bond Development Length, Structural Responses, Crack Spacing, Crack Width, Steel Stress, Pavement Behavior, Pavement Distress, Design Criteria, Performance

SUMMARY

In this study, Texas SDHPT CRCP Design Standards for deformed bar and deformed wire fabric reinforcement were evaluated using the CRCP-2 computer program. The CRCP-2 computer program is a state-of-the-art technology for analyzing the complex behavior of continuously reinforced concrete pavement. A sensitivity analysis was made using the CRCP-2 computer program. The solutions were reasonable — steel reinforcement and concrete properties along with environmental factors were significant variables.

Generally, the Design Standards for deformed bar and deformed wire fabric reinforcement are satisfactory.

Since the properties of coarse aggregate have a considerable effect on the performance of concrete, the effects of various coarse aggregates on the CRCP behavior were studied. The effects were significant, and the development of different design standards for various coarse aggregate types is desirable.

One of the findings in this study is that the ratio of bond area to concrete volume is a more significant variable than the percentage of steel. The importance of the ratio of bond area to concrete volume was studied. In 13, 14, and 15-inch thicknesses with one-layer reinforcement, the Q values should be increased.

A new aspect of the Standard for deformed bar reinforcement is that the contractors have the option of placing two layers of steel for pavements move than 10 inches thick. Two-layer reinforcement is more desirable than one-layer reinforcement for thicker slabs.

For deformed bar reinforcement, there is a linear relationship between crack spacing and crack width regardless of bar size and coarse aggregate type. Crack width is not directly influenced by bar size. Crack width should be controlled by crack spacing, through a proper combination of design variables.

There is not a definite relationship between crack width and steel stress, and the latter was significantly influenced by bar size.

For deformed wire fabric reinforcement, transverse wire spacings of 18 inches or greater are desirable, regardless of the coarse aggregate types used.

IMPLEMENTATION STATEMENT

The proposed SDHPT CRCP Design Standards for deformed bar and deformed wire fabric reinforcement were evaluated using computer program CRCP-2.

Generally, the Design Standard for deformed bar reinforcement is satisfactory. However, minor changes are needed.

For deformed wire fabric reinforcement, wire spacings of 18 inches or greater are recommended.

The study, using various coarse aggregates, shows that the structural responses are different, depending on the coarse aggregate type used. In order to provide equal performance regardless of coarse aggregate type, specific details of the Standards need to be developed for different coarse aggregate types.

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CHAPTER 1. INTRODUCTION

BACKGROUND

In the 1960's, as traffic volumes continued to increase, it became apparent that user delays due to maintenance work or rehabilitation of pavements were significant and should be an input in the design of pavement systems.

The use of continuously reinforced concrete pavement (CRCP), which costs more to build than asphalt concrete pavement or other types of portland cement concrete pavement but needs minimum maintenance, was considered desirable for high-volume and, thus, high-user-cost roads. Partly for that reason, Texas built an extensive system of CRCP in the 1950's, 60's, and 70's, eventually using both deformed bars and deformed wire fabric for longitudinal reinforcement, the latter of which was developed to permit quicker steel placement.

More recently, as axle loads have continued to increase, a thicker and more durable pavement was needed, and the Texas SDHPT developed a standard for thicker pavements. Until now the specifications and standards have not differentiated between coarse aggregate types and, therefore, any coarse aggregate could be used as long as the minimum strength requirement was met.

However, recent performance studies have shown significant variations between pavements using different coarse aggregates (Refs 1 and 2). Project 3-8-86-422, "Evaluation of Pavement Concrete Using Texas Coarse Aggregates," developed extensive background material on the concrete properties for various coarse aggregates used in Texas, and reports by that project (Refs 3 and 4) show a difference in material properties of concretes containing siliceous river gravel and crushed limestone: varying material properties caused variations in performance between pavements with different coarse aggregates. This warrants the necessity for developing design standards to minimize the variations in performance between pavements using different coarse aggregates.

OBJECTIVE OF THE STUDY

In Research Report 177-17 (Ref 5), the limiting design criteria for each of the structural responses were established to control and restrain distress. The major distresses common to CRC pavements are highly correlated with the structural responses. If the structural responses of CRC pavements are known, it is possible to predict occurrence of the major distresses in CRC pavements. The Texas SDHPT has developed CRCP Design Standards based on its experience. The primary objective of this report is to evaluate CRCP Design Standards for concretes containing siliceous river gravel and limestone coarse aggregates, using the computer program CRCP-2 and design criteria developed in Research Report 177-17 (Ref 5), recommending improved guidelines when appropriate.

SCOPE OF THE STUDY

In this study, Texas SDHPT CRCP Design Standard (B)-85 is evaluated using the theoretical model computer program CRCP-2.

Chapter 2 describes the CRCP-2 program, lists the assumptions made in the development of the model, gives a brief explanation of the model, together with input and output parameters of the program, and reports the results of the sensitivity analysis done on the output from this program.

In Chapter 3, a description of the proposed CRCP Design Standard for deformed bar reinforcement is presented. It was evaluated for concretes with siliceous river gravel and limestone aggregate. The properties of concrete made with different coarse aggregates are discussed. The three structural responses and the limiting criteria are discussed. The results of the evaluation are presented.

Chapter 4 discusses the results of the evaluation of the CRCP Design Standard for deformed wire fabric reinforcement.

Conclusions and recommendations are in Chapter 5.

CHAPTER 2. THE COMPUTER PROGRAM

Using mechanistic analyses of pavement systems helps pavement engineers to develop rational pavement designs. A recent AASHTO Design Guide (Ref 6) supports this and includes a mechanistic analysis study. In this chapter, theoretical mechanistic models and a computer program developed for the analysis of CRCP are described. The computer program was evaluated using a sensitivity analysis, and the result is presented.

HISTORY OF CRCP MODEL DEVELOPMENT

Continuously reinforced concrete pavement is pavement in which the continuity of the reinforcement is not interrupted except at abutting structures or at the ends of a project. It has no transverse joints other than expansion joints at bridges and other structures and construction joints used at the end of a day's pour. Instead of having joints, the pavement develops numerous transverse hairline cracks whose movement is reduced by the steel reinforcement.

This pavement type was developed to reduce maintenance problems encountered, principally at joints, in concrete pavements. Elimination of the transverse joints in continuously reinforced concrete pavement gives it several distinct advantages over other pavement types. Although the initial construction cost is higher than that of conventional concrete pavements, the low maintenance cost, long pavement life, and smooth riding quality give CRCP a definite advantage.

However, these advantages are achieved only when CRCP is properly designed and constructed. Thus, the development of a rational design is essential for successful performance of this pavement type. This need for a rational design procedure led to an extensive and comprehensive study initiated in 1972 at the Center for Highway Research of The University of Texas at Austin under NCHRP Project 1-15. The project consisted of a review of design and construction variables, theoretical studies, field surveys, and laboratory investigations. The fundamental philosophy of that review was that, through a combination of field observations and laboratory studies, reliable procedures could be achieved for developing mathematical models that simulate field performance of CRC pavements. Based on these mathematical models, the CRCP-1 computer program was developed to calculate the stresses in concrete and steel, the crack spacing, and the crack width resulting from concrete volume changes due to temperature change and shrinkage (Ref 2).

The program provides detailed information on the structural responses of CRC pavements as a function of time. Based on that information, a rational design can be developed for CRC pavements. An incremental approach to predict the formation of transverse cracks as a function of time was adopted. From the predicted crack pattern, theoretical models were developed to compute the other structural responses, such as crack width and stresses in the steel and concrete. The method for solving these theoretical models is too complex for hand calculations, thus a computer program was developed which utilizes the derived theoretical equations and the various nonlinearities encountered in the CRCP problem. This computer program was designated as CRCP-1.

CRCP-2 is an extension and revision of the CRCP-1 computer program for the analysis of continuously reinforced concrete pavement. The major changes incorporated into the CRCP-2 program are three-fold: (1) the inclusion of wheel load stress, (2) the modification of the mathematical model to cover the case where the bond development length exceeds half the crack spacing, and (3) the inclusion of the concrete strength gain as a function of time to cover a wide range of age, i.e., greater than 28 days (Ref 7).

THEORETICAL MODELS

The major component in analyzing CRC pavement is the determination of the stresses in concrete and in steel as a function of time and space. There are two forces in CRC pavements: internal forces developed from restrained pavement volume changes, and external forces developed from the traffic loads. In the following sections, first the basic concepts used in modeling the stress history of a CRCP are presented. Next, the basic assumptions made in the development of the models are outlined.

Modeling Concepts

Since concrete properties and the environmental factors inducing internal forces, i.e., temperature and moisture change, are changing with time, an incremental approach was adopted to predict the structural responses. The basic concept, shown in Fig 2.1, is summarized as follows:

(1) At any time t_1, determine the tensile strength of concrete from the strength-time relationship [Fig 2.1(a)].

(2) Compute the drying shrinkage Z_1 and the temperature drop DT_1 corresponding to the time t_1 [Fig 2.1(b)].

(3) With the mathematical models, calculate the maximum concrete tensile stress. If the pavement is open to traffic before time t_1, the maximum concrete tensile stress is the sum of the concrete tensile stress due to shrinkage, the temperature drop at time t_1, and the wheel load stress. If not, it is the sum of only the concrete tensile stress due to drying shrinkage and the temperature drop [Fig 2.1(c)].

(4) Compare the concrete strength with concrete stress [Fig 2.1(d)]. If the strength is higher than the stress, cracking does not occur.

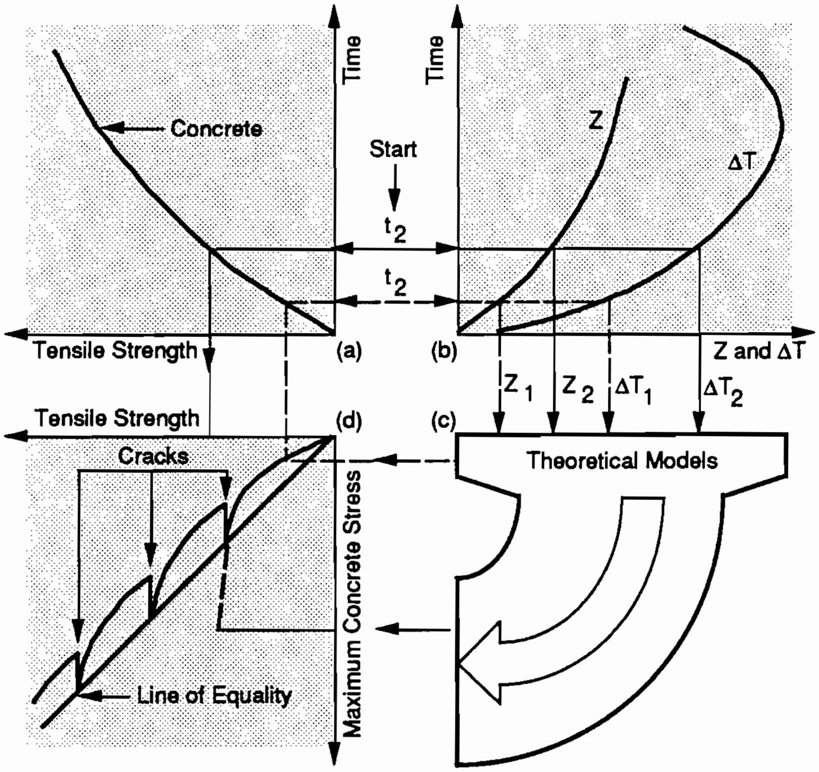


Fig 2.1. Incremental approach as applied to the CRCP system (Ref 2).

(5) Increase the time to t_2 and repeat steps (1) through (4). If the stress is higher than the strength, as shown in Fig 2.1(d), a crack occurs between times t_1 and t_2.

(6) Solve for the time (somewhere between t_1 and t_2) and the corresponding state of stress at which cracking occurred.

(7) Increase time and search for additional cracks as they develop.

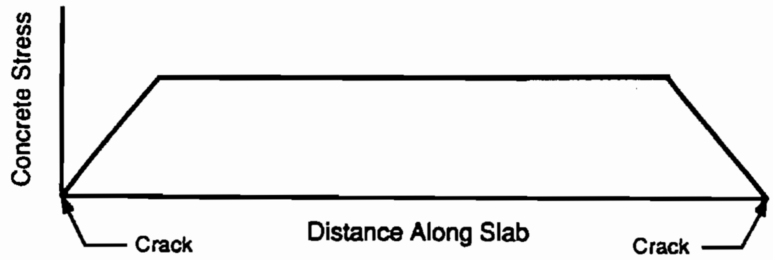
If there is no steel in the slab and no friction between the slab and the subbase material, the slab will move freely, and no concrete stress will develop. It is the restraint due to the steel reinforcement and the subbase resistance to the concrete volume change which causes the concrete stress to develop. After the concrete placement, stress begins to build up in the concrete as the temperature changes and the concrete loses moisture. Figure 2.2 shows the concrete stress distribution along the slab length. The concrete stress distribution in CRCP when there is no subbase resistance is illustrated in Fig 2.2 (a). At the crack, considerable stress develops in the steel. This high steel stress is transferred to the surrounding concrete, resulting in the change in the concrete and steel stresses along the slab length. The change in the steel stress causes the bond stress to develop between the steel and the concrete. The bond stress near the crack usually exceeds the bond strength, resulting in bond slippage, or, in other words, the relative movement between the steel and the concrete. However, in the fully bonded zone, if the effect of subbase friction is ignored, there is no stress variation in the concrete and the steel, and there is no bond stress between them. Figure 2.2(b) illustrates the effect of subbase friction on the concrete stress distribution. The maximum concrete stress, whether it is tensile or compressive, always occurs at the middle of the slab. Figure 2.3 explains why the concrete stress due to the subbase friction is the maximum at the middle of the slab length. The resulting concrete stress distribution in the CRCP slab segment is shown in Fig 2.2(c). If the maximum concrete stress at the middle of the slab between the cracks exceeds the concrete tensile strength, the crack will occur at that location. The concrete and steel stress distributions depend on many variables, such as the amount of drying shrinkage, temperature change, thermal coefficient of concrete and steel slab length, steel reinforcement, and subbase friction characteristics. Reference 4 provides detailed information on the development of the models which calculate the concrete and steel stresses for a given drying shrinkage and temperature change.

Figure 2.4 explains how the program determines the crack spacing as a function of time. With a given drying shrinkage and temperature change data, the maximum concrete stress is determined for a given slab length. At time A, the maximum concrete stress exceeds the concrete tensile strength, and hence cracking occurs at the middle of the slab length. The new crack spacing is half of the previous one. The cumulative frictional resistance for the shorter slab is lessened, which in turn lowers the concrete stress. However, further drying shrinkage and/or temperature drop causes the concrete stress to exceed the concrete tensile strength at times B, C, and D, resulting in cracking at each time. When wheel load is applied, if the sum of the concrete stresses due to the environment and the external wheel load exceeds the concrete tensile strength, a crack will develop. After crack spacing and steel stress are determined, crack width is determined by subtracting the concrete elastic elongation due to the developed concrete tensile stress from the free concrete slab movement. With this method, the history of the three structural responses is predicted.

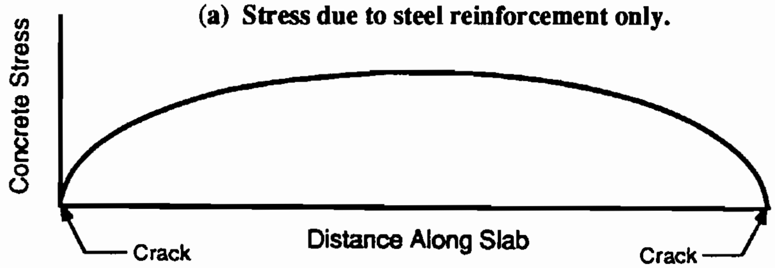
Assumptions

In the development of the model, the following assumptions were made:

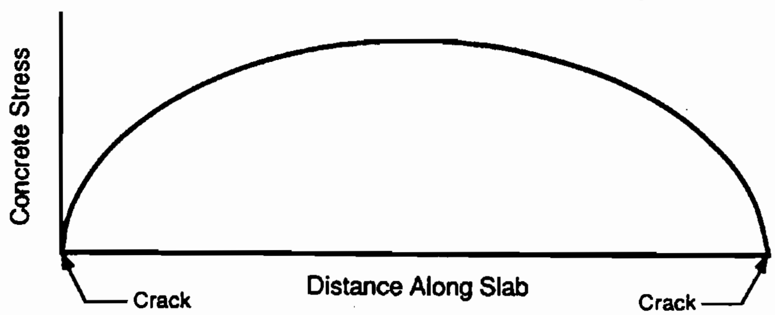
(1) A crack occurs when the concrete tensile stress exceeds the concrete tensile strength andwhen the



(a) Stress due to steel reinforcement only.



(b) Stress due to subbase friction only.



(c) Stress due to steel reinforcement and subbase friction.

Fig 2.2. Concrete stress distribution along the slab in CRCP.

concrete stress at the location of the crack is zero after cracking.

(2) Concrete and steel properties are linearly elastic.

(3) In the fully bonded sections of the concrete slab, there is no relative movement between the steel and the concrete.

(4) The force displacement curve which characterizes the frictional resistance between the concrete slab and the underlying subbase is elastic.

(5) Temperature variations and shrinkage due to drying are uniformly distributed throughout the slab and, hence, a uni-axial structural model is adopted for the analysis of the problem.

(6) Material properties are homogeneous and isotropic.

(7) The effects of the creep of the concrete and slab warping are neglected.

THE INPUT PARAMETERS

The input parameters for the CRCP-2 program consist of (1) steel properties, (2) concrete properties, (3) environmental inputs, (4) external wheel load inputs, and (5) the slab-base friction relationship.

Steel Properties

The primary purpose of the longitudinal reinforcing steel is to force the concrete to develop numerous transverse hairline cracks and to keep the cracks tightly closed. The optimum reinforcement causes sufficient stress-relieving transverse cracks to occur and then holds these cracks tightly closed under service to prevent the passage of water from the surface to the subbase and to insure sufficient aggregate interlock.

Information on this input includes the type of reinforcement (deformed bar or deformed wire fabric), percent steel reinforcement, bar diameter, yield stress, modulus of elasticity, thermal coefficient, and transverse wire spacing in the case of deformed wire fabric reinforcement.

Concrete Properties

The major difficulty facing a pavement engineer is controlling the inherent concrete volume changes due to temperature change and moisture loss. An accurate estimation of the concrete properties is extremely important for obtaining an analysis that simulates field conditions.

For this program, the slab thickness, thermal coefficient, total drying shrinkage, unit weight, and age-tensile strength relationship are needed. If the age-tensile strength relationship is not available, the 28-day compressive strength can be used, and the tensile strength data are generated inside the program.

Environmental Inputs

The environmental inputs consist of two parts. The first deals with the analysis period directly after concrete placement, where the average curing temperature and the minimum daily temperature for the desired number of days are input.

The second part deals with the analysis period after the concrete achieves, for all practical purposes, its full strength. The minimum temperature the pavement is expected to experience in its life and the age after construction at which this temperature occurs are the inputs for this part.

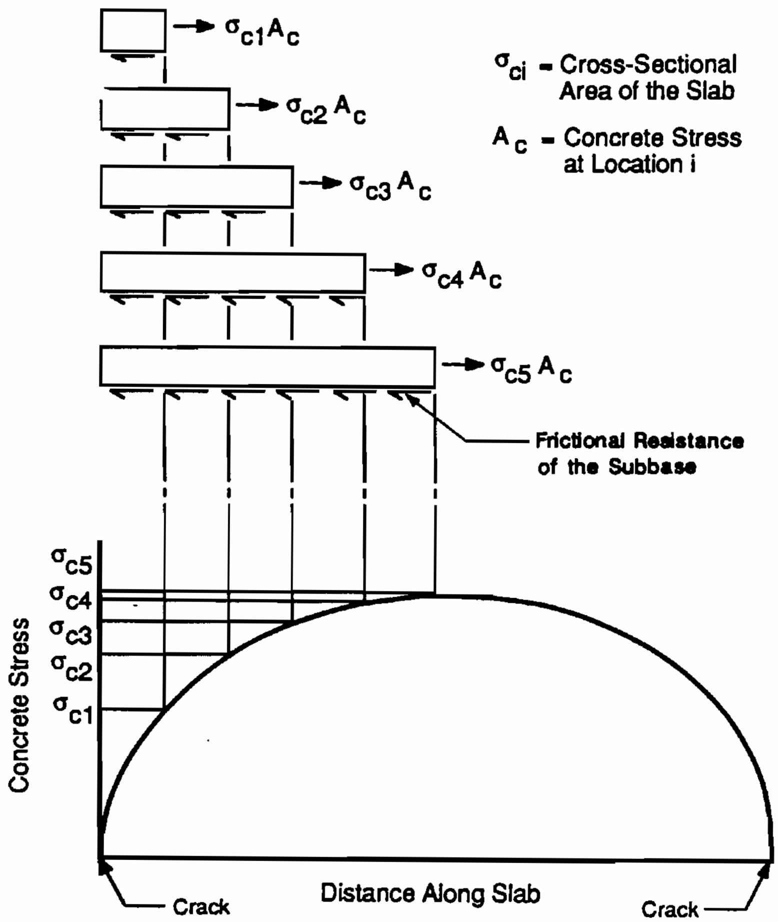


Fig 2.3. Concrete stress distribution in a slab due to friction when there is no reinforcement.

External Load Inputs

Wheel load, wheel-base radius, the number of days after concrete placement before the wheel load is applied, and the modulus of subgrade reaction are required. As an option, the user can input the stress due to wheel load, in which case the above variables will not be required.

Slab-Base Friction Relationship

The resistance to movement during contraction is a result of shearing resistance in the subbase, plus sliding friction. A contracting slab will move more at its free end than in the center, and, therefore, frictional resistance varies along the slab with the maximum at the edge and the minimum at the center. The frictional force-movement relationship, and the type of curve (straight line, parabola, or multi-linear) are needed.

THE OUTPUT PARAMETERS

The output of the CRCP-2 program consists of

(1) an echo print of the input values,

(2) detailed information on theconcrete properties and structural responses as a function of time up to 50 days,

(3) structural responses after the crack spacing has been stabilized, and

(4) detailed information on the slab segment, such as concrete movement, frictional force, concrete stress, and steel stress as a function of space.

An example output is shown in Appendix A.

In the echo print, all the input values are printed so that the user can check to see whether or not the input format was followed correctly.

In the second part of the output, detailed information, such as temperature drop, shrinkage, concrete tensile strength, crack spacing, crack width, and stresses in concrete and steel as a function of time, is printed. From this information, pavement behavior at early ages can be fully studied. The concrete stress here is the value at the mid-point of the slab between adjacent transverse cracks, and the steel stress is the value at the crack at that time.

In the third part of the output, the structural responses after the crack spacing is stabilized are printed. This crack spacing is the mean crack spacing after the pavement experiences the lowest temperature. Since concrete gains its strength as time elapses, this crack spacing is considered as the final spacing until the cracking due to fatigue occurs. The maximum concrete stress is the maximum the pavement ever experiences and is not necessarily the value when the temperature is the lowest. The maximum steel stress and concrete tensile strength printed here are the values when the temperature is the lowest in the winter after construction.

The fourth part of the output presents detailed information on slab movement, frictional force, and the stresses in the concrete and steel as a function of space along the slab segment between cracks.

SENSITIVITY ANALYSIS

When a theoretical model for the analysis of a highly complex system such as CRC pavement is developed or a major modification is made to that model, it is necessary to perform a sensitivity analysis to establish the reasonableness of the solutions compared to field observations and the relative importance of the input variables in the model.

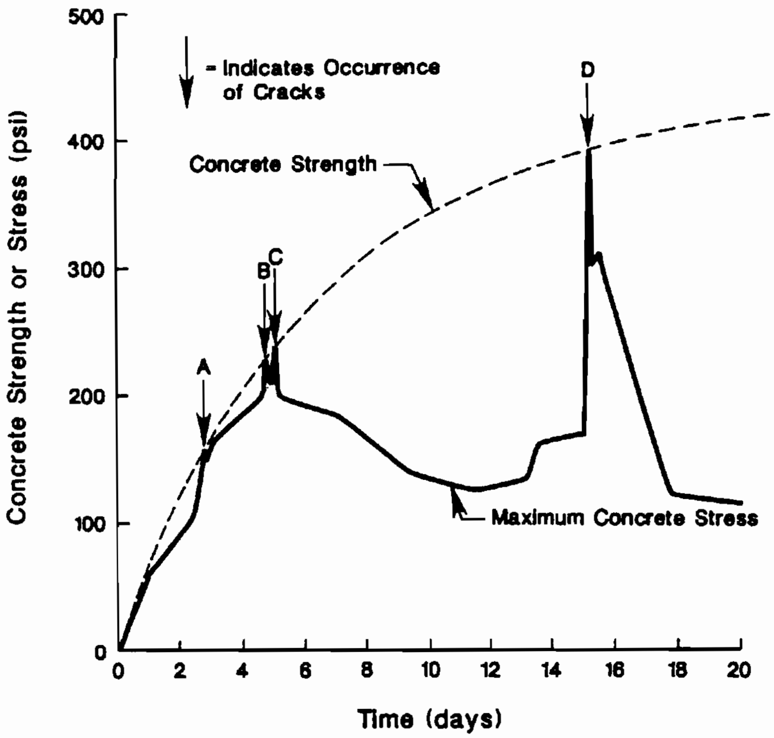


Fig 2.4. Variation of concrete strength and stress, and crack history (Ref 4).

A sensitivity analysis was done on CRCP-1 in Ref 8, and, in that study, it was found that the solutions from the program were reasonable. As described earlier, some modifications were made to CRCP-1 to simulate the field conditions more accurately, and it was necessary to perform the sensitivity analysis on the revised computer program, CRCP-2.

In this study, a traditional one-factor-at-a-time analysis was done for three levels of design values.

Selection of Levels

In order to get reasonable results from the analysis, it is important to use a reasonable range of values for the input variables. For this study, a three-level experiment was established, and each input variable was given low, medium, and high values, based on the ranges which are found in the field.

A medium level is a value under average design conditions; a low level is a practical value at the lower extreme with respect to the medium level; and a high level is a practical value at the upper extreme. Table 2.1 shows the values selected for this analysis. In this one-factor-at-a-time experiment, all the design variables except one were kept at a medium level, and the response values for the three levels of the selected variable were taken. Another variable was then chosen and this process was continued until all variables had been considered. In this method, the effect of an independent variable is determined from the difference in responses corresponding to different levels of that variable.

Presentation of the Results

Table 2.2 presents the results of this analysis. In this analysis, the steel reinforcement has the most significant effect on the predicted structural responses. The difference in predicted responses for low and high level input values of steel reinforcement was considered to be unity, and, consequently, the structural response values for each variable in Table 2.2 show the relative ratio of the response of that variable to that of the steel reinforcement.

Comparing the results from CRCP-1 (Ref 8) and CRCP-2 shows the inclusion of the wheel load doesn't change the relative importance of the variables. Generally, concrete properties have more significant effects on the structural responses than steel properties, except steel reinforcement.

SUMMARY

A mechanistic model for analyzing structural behavior of CRCP has been developed. It predicts crack spacing, crack width, and steel stress history for CRCP. In a comprehensive study of CRCP, structural responses of CRCP were significantly affected by longitudinal percent steel reinforcement, slab thickness, and concrete modulus of rupture (Ref 2). A sensitivity analysis shows the model is reasonable. This model can be used for design of or modeling a specific pavement.

TABLE 2.1. THE INPUT VALUES FOR SINGLE FACTORIAL EXPERIMENT

Variables	Low	Medium	High
Reinforcement (percent)	0.4	0.6	0.7
Bar size	#5	#6	#7
Elastic modulus of steel ($\times 10^6$ psi)	27.0	29.0	30.0
Thermal coefficient of steel ($\times 10^{-6}$/°F)	3.0	5.0	7.0
Slab thickness (inches)	6.0	8.0	10.0
Thermal coefficient of concrete ($\times 10^{-6}$/°F)	3.0	5.0	7.0
Drying shrinkage ($\times 10^{-4}$ inch/inch)	3.0	5.0	6.5
Elastic modulus of concrete ($\times 10^6$ psi)	3.0	4.0	5.0
Concrete tensile stength (psi)	400.0	450.0	500.0
External load (kips)	8.0	9.0	10.0
Modulus of subgrade reaction (psi/inches)	400.0	600.0	800.0
Curing temperature (°F) *	70.0	85.0	95.0
Maximum subbase frictional resistance (psi) *	1.0	2.0	6.0

* The minimum temperature for 28 days after setting concrete is 50°F, and the minimum temperature this slab experiences occurs 165 days after setting concrete, and is 10°F.

** The movement at sliding is kept at 0.06 inch.

TABLE 2.2. THE RELATIVE IMPORTANCE OF INPUT VARIABLES

Variables	Crack Spacing	Crack Width	Steel Stress	Sum	Ranking
Reinforcement	1.00	1.00	1.00	3.00	1
Bar diameter	0.27	0.27	0.01	0.55	6
Elastic modulus of steel	0.09	0.08	0.01	0.18	9
Thermal coefficient of steel	0.00	0.00	0.11	0.11	12
Slab thickness	0.50	0.49	0.62	1.61	3
Thermal coefficient of concrete	0.34	0.08	0.32	0.74	5
Shrinkage	0.42	0.08	0.36	0.86	4
Elastic modulus of concrete	0.09	0.05	0.08	0.22	8
Concrete tensile strength	0.78	0.74	0.92	2.44	2
Wheel load	0.14	0.13	0.16	0.43	7
Modulus of subgrade reaction	0.05	0.05	0.06	0.16	10
Curing temperature	0.13	0.01	0.02	0.16	10
Slab-base friction	0.002	0.000	0.003	0.005	13

CHAPTER 3. EVALUATION OF CRCP DESIGN STANDARD FOR DEFORMED BAR REINFORCEMENT

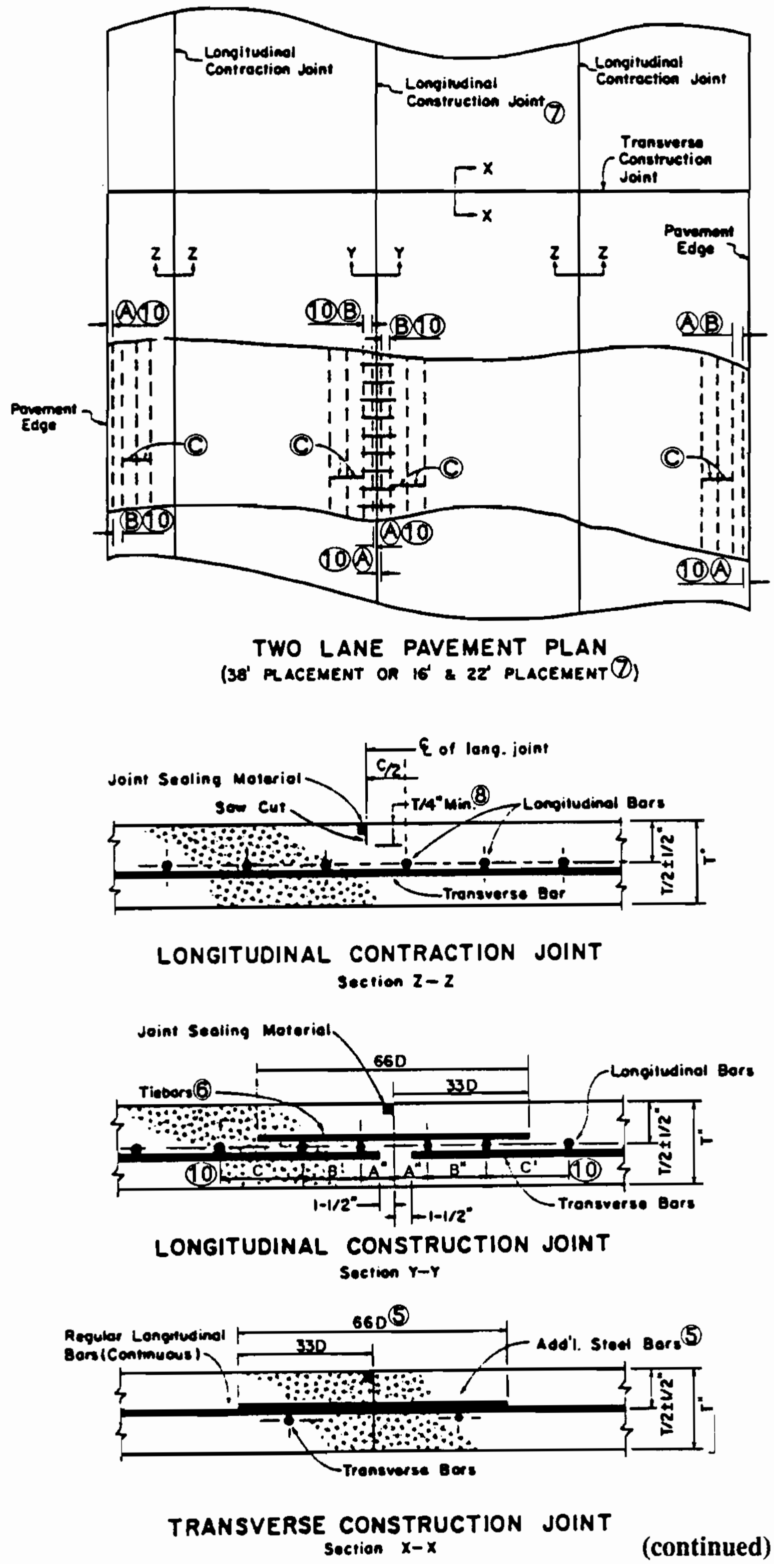
In this chapter, the CRCP Design Standard for deformed bar reinforcement developed by the Texas SDHPT is described, and the results of its analysis using a computer program are presented. There are significant variations in performance of pavements using different coarse aggregates; the two most widely used in Texas, siliceous river gravel and limestone, were selected for the evaluation of the standard. The ratio of the bond area to the concrete volume is an important parameter in CRCP behavior, and a study using various ratios was conducted and described.

PROPOSED DESIGN STANDARD

The Texas SDHPT first developed a CRCP standard design detail in 1958, and since that time numerous revisions have occurred. Due to increasing pavement thicknesses, the SDHPT developed a CRCP Design Standard for deformed bars [Texas SDHPT CRCP(B)-85], as shown in Fig 3.1. As may be noted, it specifies the longitudinal bar size and spacing for each slab thickness, the transverse bar size, and spacing for various maximum allowable pavement widths.

The Standard also specifies the required number of bars for each bar spacing and typical placement width. The specified number of bars includes two bars placed at each edge and having spacings different from the interior bars. The edge bars permit adjustments and, therefore, uniform practical spacings may be used for the interior bars. Although the design contains 0.5 percent longitudinal steel reinforcement, the actual percentage varies with placement width. The cross-sectional steel reinforcement for various pavement widths and slab thicknesses provided for by the standard is shown in Table 3.1.

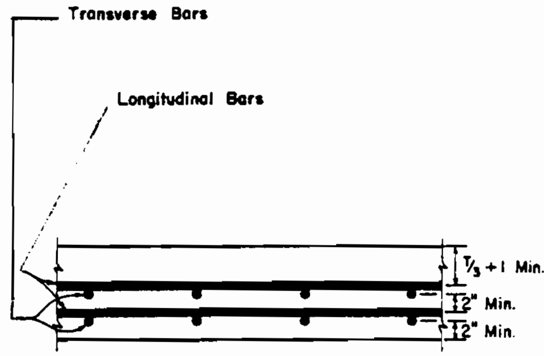
The new aspect of the proposed design standard is that, for pavements greater than 10 inches thick, the contractors has the option of placing two layers of steel. Placing two layers of steel with smaller size bars in thicker pavements provides more bond surface area over one layer of steel reinforcement.



(continued)

Fig 3.1. CRCP Design Standard for deformed bar reinforcement [Texas SDHPT CRCP(B)-85].

Z ①	T (in.)	Longitudinal Bar Size	Spacing C (in.)	Trans. Bar Size	Maximum Allowable ③ Pavement Width (ft.) For Given Transverse Steel Spacings (in.) 12	24	36	B_sW ④ (in.-ft.)
0.6	8	6	9	4	120	60	40	120.0
				5	186	93	62	186.0
				6	264	132	88	264.0
	9	6	8	4	106	53	35	106.7
				5	165	82	55	165.3
				6	234	117	78	234.7
	10	6	7	4	96	48	32	96.0
				5	148	74	49	148.8
				6	211	105	70	211.2
	11	6	7	4	87	43	29	87.3
				5	135	67	45	135.3
		4②	6	6	192	96	64	192.0
	12	6	6	4	80	40	26	80.0
				5	124	62	41	124.0
		5②	9	6	176	88	58	176.0
	13	7	8	4	73	36	24	73.8
				5	114	57	38	114.5
		5②	8	6	162	81	54	162.5
	14	7	7	4	68	34	22	68.6
				5	106	53	35	106.3
		5②	7.5	6	150	75	50	150.9
	15	7	7	4	64	32	21	64.0
				5	99	49	33	99.2
		5②	7	6	140	70	46	140.8



OPTIONAL STEEL PLACEMENT ②

NOTE: ① Longitudinal and Transverse Bars shall be deformed steel conforming to ASTM A-615 or ASTM A-616 (Grade 60) as noted in the standard specifications and therefore the percentage of steel required is higher than that for wire mats. (Grade 70 steel).

② For pavements greater than 11" in thickness, contractors may have the option of placing two layers of steel. The smaller longitudinal bar sizes indicated are only to be used when two layers of steel are placed.
For transverse bars, if all other variables are held constant, the maximum allowable pavement width may be doubled when two layers of steel are used.

When the "Double Strike-Off" procedure is not used chairs will be required to support both layers of steel.

③ Pavement width shall be measured at right angles to the centerline and shall include all mainlanes, connectors, ramps and concrete shoulders that are tied together. Transverse steel requirements and the maximum allowable pavement width were determined using subgrade drag theory (see Appendix F, Section 109 of the Highway Design Division Operations and Procedures Manual) with a coefficient of sliding resistance (F of 1.5, and an allowable steel stress (F_s) of 45.0 ksi.

④ To determine the maximum allowable pavement width (W) for spacing other than those given, divide "B_sW" (for the given bar size) by the desired transverse bar spacing (B_s). Transverse bar spacing shall not be less than 12" nor greater than 36".

⑤ Additional steel at the transverse construction joints shall be bars of equal diameter; and a spacing of double that specified for the longitudinal steel of the given thickness. The length of the bars shall be 66 times the bar diameter ("D").

⑥ Transverse tiebars at the longitudinal construction joints shall be bars of equal diameter and spacing to those specified for the transverse steel of the given thickness. The length of the bars shall be 66 times the bar diameter ("D").

⑦ The longitudinal construction joint can be relocated or may be replaced by a longitudinal contraction joint depending on the placement width.

⑧ If siliceous river gravel is used as a coarse aggregate, a cut of T/3 shall be required.

⑨ When machine-placing of steel reinforcement is used, the use of chairs shall not be required, and the transverse steel may be placed above or below the longitudinal steel.

⑩ The number of bars required for the various placement widths (indicated in the Table) includes 2 bars at "B" spacing on both sides with an overhang "A".

"A" spacing shall be between 3" and 4".
"B" spacing shall be between 3" and 9".

The two spacings combined ("A" and "B"), located at both longitudinal edges of the pour, shall provide for the remaining space and steel location to round out the placement width.

(continued)

Fig 3.1. (continued).

Spacing c (in.)	Number of Bars Required for Various Typical Placement Widths (ft.) (10)						
	12	16	22	24	27	34	38
6	24	32	44	48	54	68	76
7	21	27	37	41	46	58	65
8	18	24	33	36	41	51	57
9	16	22	30	32	36	46	51

GENERAL NOTES

1. NO EXPANSION JOINTS WILL BE USED EXCEPT AT STRUCTURE ENDS OR FIXED OBJECTS AS SHOWN ELSEWHERE IN THE PLANS.

2. FOR FURTHER INFORMATION REGARDING THE PLACEMENT OF CONCRETE AND REINFORCEMENT REFER TO THE GOVERNING SPECIFICATIONS FOR "CONCRETE PAVEMENTS."

3. DETAILS AS TO PAVEMENT WIDTH, PAVEMENT THICKNESS AND THE CROWN CROSS-SLOPE SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.

4. WITHIN ANY AREA BOUNDED BY TWO FEET OF PAVEMENT LENGTH MEASURED PARALLEL TO THE CENTERLINE AND TWELVE FEET OF PAVEMENT WIDTH MEASURED PERPENDICULAR TO THE PAVEMENT CENTERLINE, NOT OVER 33% OF THE REGULAR LONGITUDINAL STEEL SHALL BE SPLICED.

5. THE LONGITUDINAL STEEL SHALL BE PLACED AT THE VERTICAL SLAB CENTER WITH A TOLERANCE OF 1/2 INCH. TRANSVERSE STEEL SHALL BE PLACED DIRECTLY ABOVE OR BELOW THE LONGITUDINAL STEEL.

6. SPLICES SHALL BE A MINIMUM OF 33 TIMES THE NOMINAL STEEL DIAMETER ("D").

7. BARS THAT REQUIRE BENDING SHALL BE GRADE 40 STEEL CONFORMING TO REQUIREMENTS OF ASTM DESIGNATION: A 615. SPACINGS FOR GRADE 40 STEEL SHALL BE 2/3 OF THAT SPECIFIED FOR GRADE 60 STEEL.

8. AT TRANSVERSE CONSTRUCTION JOINTS THE REGULAR LONGITUDINAL STEEL SHALL EXTEND A MINIMUM OF FOUR FEET ON EITHER SIDE OF THE JOINT.

9. VIBRATION WITH HAND-MANIPULATED MECHANICAL VIBRATORS WILL BE REQUIRED ADJACENT TO ALL TRANSVERSE CONSTRUCTION JOINTS.

10. THE CHAIRS USED TO SUPPORT THE STEEL SHALL BE OF SUFFICIENT STRUCTURAL QUALITY AND NUMBER TO HOLD THE STEEL MAT WITHIN THE PLACEMENT HEIGHT TOLERANCES. CHAIRS SHALL BE OF A TYPE APPROVED BY THE ENGINEER.

11. WITH THE APPROVAL OF THE ENGINEER, MULTIPLE PIECE TIEBARS (THREADED COUPLING OR OTHER ADEQUATE DEVICE) MAY BE USED TO FACILITATE CONSTRUCTION PROVIDED THE SYSTEM DEVELOPS A FORCE EQUAL TO 1-1/2 TIMES THE MINIMUM YIELD STRENGTH OF THE TIEBAR SHOWN. THE SPACING FOR THE SYSTEM SHALL BE LESS THAN OR EQUAL TO THAT OF THE TIEBARS SHOWN.

12. JOINT, GROOVE AND SEAL DETAILS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.

13. LONGITUDINAL AND TRANSVERSE STEEL SPACING SHALL NOT VARY MORE THAN ONE-TWELFTH OF THE SPACING SHOWN HEREON.

14. IF WIDTHS OCCUR, OTHER THAN THE TYPICAL WIDTHS SHOWN, INDIVIDUAL BARS (WIRES) OF THE SIZE SPECIFIED HEREON MAY BE ADDED OR REMOVED TO OBTAIN THE APPROPRIATE WIDTH. SPACING REQUIREMENTS SHALL NOT BE EXCEEDED, HOWEVER.

Fig 3.1. (continued).

TABLE 3.1. LONGITUDINAL STEEL CROSS-SECTIONAL PERCENTAGES FOR VARIOUS PLACEMENT WIDTHS AND THICKNESSES PROVIDED FOR IN TEXAS SDHPT CRCP (B)-85 DESIGN STANDARD

Slab Thickness (inches)	Pavement Width (feet)						
	12	16	22	24	27	34	38
8	0.614	0.633	0.628	0.614	0.614	0.623	0.618
9	0.614	0.614	0.614	0.614	0.621	0.614	0.614
10	0.644	0.621	0.619	0.629	0.627	0.628	0.630
11	0.586	0.565	0.563	0.572	0.570	0.571	0.572
	0.595	0.595	0.595	0.595	0.595	0.595	0.595
12	0.614	0.614	0.614	0.614	0.614	0.614	0.614
	0.568	0.586	0.581	0.568	0.568	0.576	0.572
13	0.578	0.578	0.578	0.578	0.585	0.578	0.578
	0.590	0.590	0.590	0.590	0.597	0.590	0.590
14	0.626	0.604	0.602	0.611	0.610	0.611	0.612
	*	*	*	*	*	*	*
15	0.585	0.564	0.562	0.571	0.569	0.570	0.571
	0.597	0.575	0.573	0.585	0.518	0.582	0.583

* Not specified in the Standard.

INPUT VALUES FOR THE ANALYSIS

There are five categories of input variables in the CRCP-2 program, i.e., steel properties, concrete properties, slab-base friction relationships, environmental characteristics, and external load characteristics.

Since the slab thickness, percent steel, and bar size are all specified in the Standard, some reasonable values had to be assumed for the other input variables. Great care was exercised in selecting the values for the input variables which were found to be significant in the sensitivity analysis described earlier. Those variables include concrete tensile strength, thermal coefficient of concrete expansion, drying shrinkage of concrete, modulus of elasticity of concrete, and environmental characteristics. For the other variables, typical values were selected considering the average conditions in Texas. Table 3.2 presents the selected input values for this study.

At the present time, concrete pavements are constructed of various types of coarse aggregates, the choice of which generally considered to be contractor's option. The present procedures do not separate coarse aggregate types into different performance categories. Hence the present assumption is that all will perform the same as long as the specification requirements, such as cement factor, strength, and water cement ratio, are met, ignoring the effect of coarse aggregate on concrete properties, such as thermal expansion, shrinkage, creep, and modulus of elasticity of concrete. The field findings show that there is a significant difference in the structural responses for CRCP constructed with different coarse aggregate types.

Since basically two types of coarse aggregates, i.e., siliceous river gravel and limestone, hereafter referred to as SRG and LS, respectively, are used in the construction of CRC pavements in Texas, the analysis was performed for CRC pavements with those two types of coarse aggregates.

Since at least three-quarters of the volume of concrete is occupied by aggregate, its physical, thermal, and sometimes chemical properties influence the performance of concrete.

SDHPT specifications (Ref 9) state that the flexural strength (or modulus of rupture) of concrete subject to center-point loading be not less than 575 psi at the age of 7 days. A formula from Ref 10 gives the percentage of the 28-day compressive strength for various intermediate ages, as seen in Fig 3.2. The American Concrete Institute (Ref 11) suggests the following relationship between compressive strength and modulus of rupture under third-point loading for normal weight concrete:

$$f_r = 7.5\sqrt{f'_c}$$

TABLE 3.2. INPUT VALUES FOR THE ANALYSIS

Variables	Coarse Aggregate Used: SRG	LS
Reinforcement (percent)	*	*
Bar diameter (inches)	*	*
Steel modulus ($\times 10^6$ psi)	29.0	29.0
Thermal Coefficient of steel ($\times 10^{-6}$ /°F)	5.0	5.0
Slab thickness (inches)	*	*
Thermal coefficient of concrete ($\times 10^{-6}$ /°F)	6.0	3.8
Shrinkage ($\times 10^{-4}$ inch/inch)	4.5	4.0
Concrete modulus ($\times 10^6$ psi)	5.0	4.0
Concrete tensile stength (psi)	400.0	425.0
Wheel load (lbs)	9000.0	9000.0
Modulus of subgrade reaction (psi/inch)	640.0	640.0
Curing temperature (°F) *	85.0	85.0
Minimum temperature for 28 days (°F)	70.0	70.0
Minimum temperature in the first winter (°F)	17.0	17.0
Number of days to minimum temperature (days)	210.0	210.0
Maximum frictional resistance (psi) **	2.0	2.0

* See Tables 3.3 and 3.4.

** The movement at sliding is 0.06 inch.

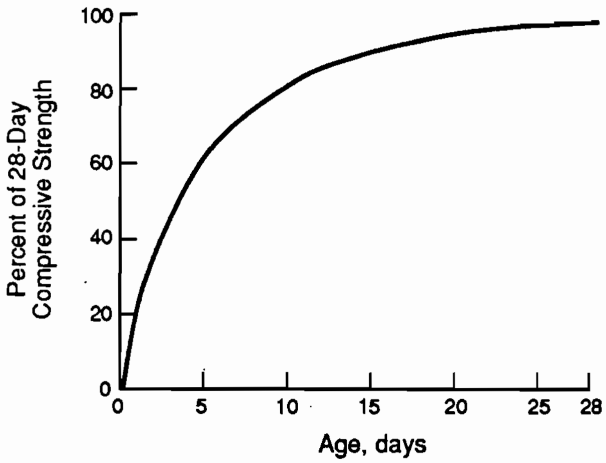


Fig 3.2. Variation of compressive strength of concrete with age (Ref 10).

where

f_r = modulus of rupture in psi and
f'_c = compressive strength in psi.

In order to get the modulus of rupture of concrete using third-point loading at the age of 28 days, it is necessary to find a relationship between modulus of rupture at center-point loading and third-point loading. Center-point loading gives a triangular bending moment distribution so that the maximum stress occurs at one section of the beam where a load is applied. Hence, failure will generally occur only when the strength of the fiber immediately under the load point is exhausted. On the other hand, under third-point loading, one-third of the beam is subjected to the maximum moment and maximum stress, and the critical crack may start at any section not strong enough to resist this stress. Since concrete consists of elements of varying strength, it is expected that third-point loading will yield a lower value of the modulus of rupture than when one point load is applied. According to Ref 12, the modulus of rupture subject to third-point loading is approximately 80 per cent of that to center-point loading. From these relationships, the modulus of rupture at the age of 7 days is 72 percent of the 28-day value. Accordingly, a modulus of rupture at third-point loading of 640 psi was selected for the 28-day strength.

Since transverse cracking due to drying shrinkage and temperature drop results in a tensile failure, the tensile strength provides a more accurate simulation than modulus of rupture. The assumption in the calculation of modulus of rupture is that stress is proportional to the distance from the neutral axis of the beam, while the shape of the actual stress block under loads nearing failure is not triangular, but parabolic. The modulus of rupture thus overestimates the tensile strength of concrete and gives a higher value than would be obtained in a direct or in a splitting tensile test.

It was found that there is a relationship between indirect tensile strength and modulus of rupture as a function of aggregate type – indirect tensile strength is five-eighths of the modulus of rupture for SRG concrete, two-thirds of the modulus of rupture for LS concrete, and three-quarters of the modulus of rupture for light-weight concrete (Ref 13). Hence 400 psi and 425 psi were selected for the indirect tensile strength for SRG and LS concrete, respectively.

The coefficient of thermal expansion for an aggregate influences the value of that coefficient for concrete containing the aggregate: e.g., the higher the coefficient for the aggregate, the higher the coefficient for the concrete. The biggest factor influencing the coefficient of thermal expansion for concrete appears to be the type of coarse aggregate, with gravel, quartz, and slag giving high coefficients of thermal expansion. Limestone and portland stone give low values, and granite gave an intermediate value (Ref 14). Figure 3.3 shows some experimental values of the thermal coefficient for neat cements, mortars, and concretes containing different types of aggregate. For this study, the values of $3.8 \times 10^{-6}/°F$ and $6.0 \times 10^{-6}/°F$ were selected for the SRG and LS concretes, respectively (Ref 10).

Drying shrinkage, an inherent characteristic of hydraulic-cement concrete, occurs with the loss of absorbed water and inter-layer water from calcium silicate hydrate gel formed during hydration of the cement. Upon exposure to drying conditions, moisture slowly diffuses from the interior mass of the concrete to the surface, replacing the moisture loss by surface evaporation. Even though extensive research studies have been carried out on the mechanism of shrinkage and creep and much is known about the various phenomena, the number of variables is so great that the precise behavior in any particular case cannot be forecast. However, the most important influence is exerted by the water-cement ratio and aggregate type used; the latter has influence on the moisture movement and resists the amount of shrinkage that can actually be realized (Refs 14 and 15). Figure 3.4 shows the effect of the mineralogical character of aggregates on shrinkage. The relative humidity of the air surrounding the concrete greatly affects the magnitude of shrinkage, as shown in Fig 3.5. Shrinkage values of 4.5×10^{-4} and 4.0×10^{-4} were selected for SRG and LS concretes, respectively.

Since the deformation produced in the concrete is partly an elastic deformation of the aggregate, it is reasonable to expect the higher values of the modulus of elasticity to be obtained for concrete made of stiffer aggregates. A review of the literature shows a number of formulas developed for the prediction of the elastic modulus of concrete from the elastic moduli and the volume concentrations of the aggregates (Refs 17 and 18). The values of 5 million and 4 million psi were selected for SRG and LS concretes, respectively (Ref 19).

It has been found that the setting temperature of the concrete has an effect on the structural responses of CRC pavements (Ref 20). The setting temperature, more specifically the temperature difference between the setting tem-

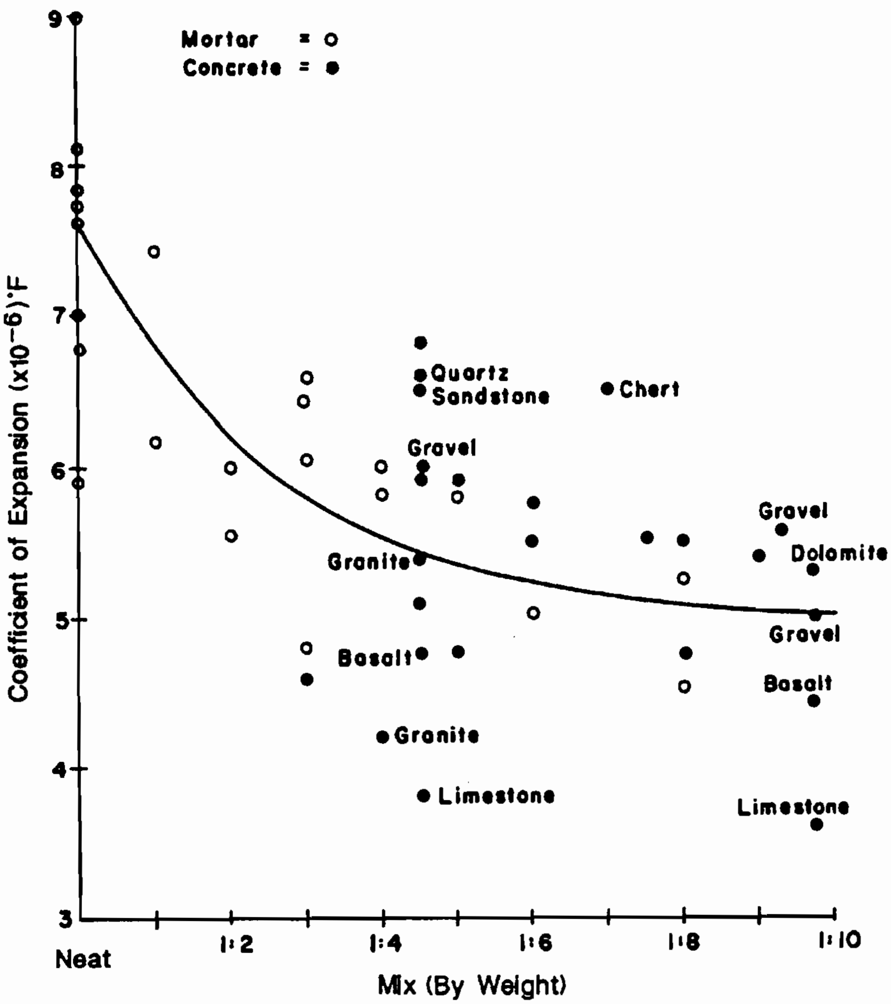


Fig 3.3. Thermal coefficient of expansion for neat cements, mortars, and concretes (Ref 10).

however, the concrete resists a moderate amount of tensile stress; this reduces the tensile force in the steel and creates a variable force in the bar.

Hence, the structural responses of CRC pavements are interrelated, and a change in crack spacing causes changes of the values for crack width and steel stress. However, the relationship among these responses varies, depending on many factors.

The major distress types in CRC pavement which have significant effects on pavement performance are punchouts, crack spacing, fatigue, low-temperature and shrinkage cracking, and steel rupture. These distresses are closely related to the structural responses of the CRC pavement, i.e., crack spacing, crack width, and steel stress. The limiting criteria for those structural responses, from consideration of the relationship between the structural responses and distress occurrence, have been established by McCullough et al (Ref 5). Thus, the distress prediction model has been developed in terms of the structural responses and other factors (Ref 21).

perature and the temperature at any time, varies for the pavement location as well as the time when the pavement is constructed.

Since the crack spacings of the 8-inch-thick CRC pavements with 0.6 percent reinforcement constructed with SRG and LS aggregates in Texas are approximately 4 feet and 6 feet, respectively, the temperature drop values giving those crack spacings were selected for this study.

RESULTS OF THE CRCP DESIGN DETAIL ANALYSIS

The crack spacing at any time is the slab segment length which has survived the environmental conditions and wheel load applications since construction. The crack width is the accumulation of the slab movement over the distance between transverse cracks. The steel stress at the crack is a value such that the equilibrium in a slab segment is achieved. The maximum steel stress occurs at the crack, since no resistance is provided by the concrete. Beyond the crack,

Therefore, this discussion is focused on the three structural responses predicted by the theoretical model. Tables 3.3 and 3.4 show the predicted structural responses for SRG and LS concretes, respectively, for the input values specified in the Standard.

Crack Spacing

There are many important factors influencing transverse cracking; among them are longitudinal reinforcement, the setting temperature of the concrete, air temperature and other environmental conditions during curing, the properties of concrete, and traffic load applications. In CRC pavement, the slab may act as a longitudinal beam or as a transverse beam, depending on the crack spacing. When the crack spacing is narrow, the slab acts as a transverse beam, with the concrete stress in the transverse direction more critical, increasing the chances of longitudinal cracking. This narrow crack spacing with resulting longitudinal cracking causes punchouts. Another problem associated with the narrow

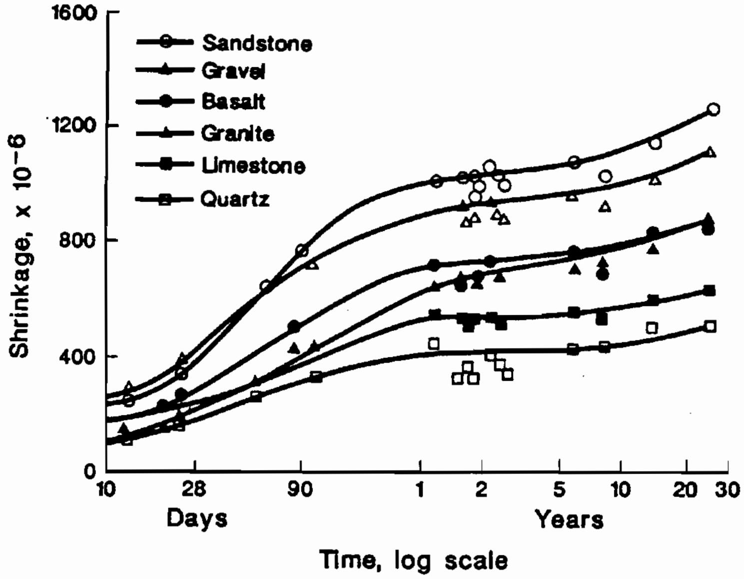


Fig 3.4. Shrinkage of concretes of fixed mix proportions but made with different aggregates and stored in air at 70°F and a relative humidity of 50 percent (Ref 16).

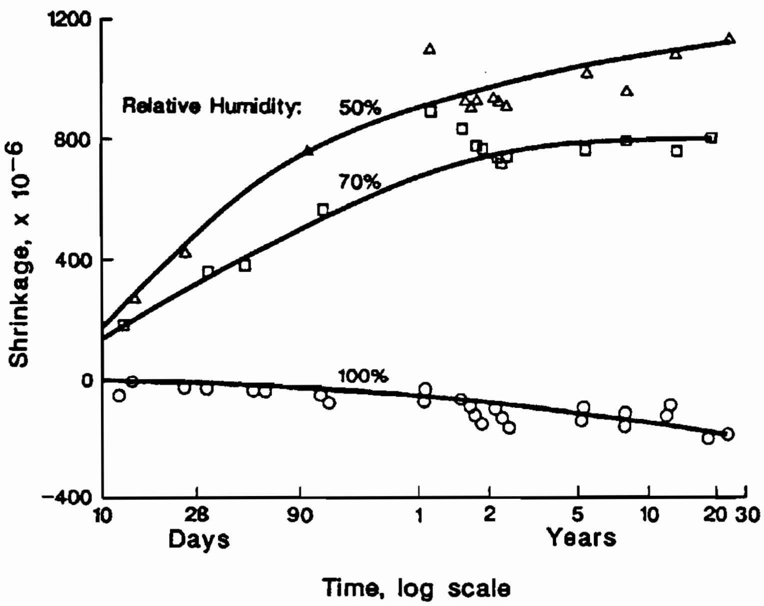


Fig 3.5. Relationship between shrinkage and time for concrete stored at different relative humidities (Ref 16).

crack spacing is the higher probability that the bond development length will exceed one-half of the crack spacing. If that happens, the structural integrity of the pavement is degraded. On the other hand, when the crack spacing is larger than 3.5 feet, the slab acts as a longitudinal beam. Accordingly, the probability of longitudinal cracking is significantly reduced; however, the transverse crack width will be wider, and spalling of transverse cracks results.

Hence, the lower and upper limits of crack spacing can be set by selecting a maximum allowable value from the spalling criterion and a minimum allowable value from punch-outs and bond development length criteria. Approximately 3.5 to 8 feet is the recommended range for crack spacing; it may be adjusted for effects of slab thickness and other considerations (Ref 5).

Figure 3.6 presents the predicted crack spacings for various slab thicknesses. It shows that the proposed Design Standard gives acceptable crack spacings except when LS aggregate is used for 13, 14, and 15-inch-thick slabs with one-layer reinforcement. For those slab thicknesses with one-layer reinforcement, the small values in the ratio of bond area to concrete volume, hereafter referred to as Q, together with low values of drying shrinkage and thermal expansion for LS concrete resulted in high crack spacing.

In Fig 3.6, there may seem to be no consistent relationship between slab thickness and crack spacing; however, this apparent inconsistency is due to the steel reinforcement and bar size combinations used for various slab thicknesses (Tables 3.3 and 3.4). For a fixed reinforcement and bar size, the cracks would be farther apart for a thicker slab. Generally, two points can be made: (1) the pavement with LS aggregate produces larger crack spacing than that with SRG aggregate, which agrees well with the findings in the field (Refs 1 and 4) and (2) there are considerable differences between the predicted crack spacings for one-layer and two-layer reinforcement.

The difference in crack spacings between the pavement with SRG and that with LS aggregate is attributed to the difference in thermal properties, drying shrinkage, and modulus of elasticity of the portland cement concrete made with those aggregates. It is also attributed to the fact that the stress at which the cracks form depends largely on the properties of the coarse aggregate: smooth SRG leads to cracking at lower stresses than rough and angular LS, because the mechanical bond is influenced by the surface properties and, to a certain degree, by the shape of the coarse aggregate (Ref 15).

The difference in crack spacings between the two types of reinforcement cannot be explained by reinforcement alone; for 11, 13, and 15-inch slab thicknesses, in spite of there being no practical difference in reinforcement between them, there is a big difference in predicted crack spacing. For

TABLE 3.3. PREDICTED STRUCTURAL RESPONSES FOR PROPOSED DESIGN STANDARD WITH SRG AGGREGATE

Slab Thickness (inches)	Steel Reinforcement (percent)*	Bar Number	Q	Crack Spacing (feet)	Maximum Crack Width (x 10^{-2} inch)	Maximum Steel Stress (x 10^4 psi)
8	0.614	6	0.0327	3.24	3.17	4.86
9	0.614	6	0.0327	3.82	3.71	5.38
10	0.631	6	0.0337	4.10	3.95	5.61
11	0.574	6	0.0306	5.21	5.00	6.48
	0.595	4	0.0476	3.37	3.23	6.36
12	0.614	6	0.0327	4.88	4.67	6.22
	0.568	5	0.0364	4.75	4.54	6.83
13	0.578	7	0.0264	7.02	6.68	6.96
	0.590	5	0.0378	4.57	4.37	6.68
14	0.614	7	0.0281	6.43	6.12	6.61
	0.584	5	0.0374	4.97	4.73	7.01
15	0.573	7	0.0262	7.40	7.03	7.18
	0.584	5	0.0374	4.97	4.73	7.01

* The value is from the bar spacing specified in the Standard.

TABLE 3.4. PREDICTED STRUCTURAL RESPONSES FOR PROPOSED DESIGN STANDARD WITH LS AGGREGATE

Slab Thickness (inches)	Steel Reinforcement (percent)*	Bar Number	Q	Crack Spacing (feet)	Maximum Crack Width (x 10^{-2} inch)	Maximum Steel Stress (x 10^4 psi)
8	0.614	6	0.0327	5.51	3.87	6.14
9	0.614	6	0.0327	6.50	4.50	6.69
10	0.631	6	0.0337	6.60	4.55	6.74
11	0.574	6	0.0306	8.86	6.02	7.87
	0.595	4	0.0476	5.54	3.77	7.61
12	0.614	6	0.0327	8.28	5.61	7.58
	0.568	5	0.0364	7.92	5.36	8.17
13	0.578	7	0.0264	10.87	7.35	8.07
	0.590	5	0.0378	7.63	5.14	7.99
14	0.614	7	0.0281	10.24	6.90	7.80
	0.584	5	0.0374	8.07	5.43	8.23
15	0.573	7	0.0262	11.51	7.87	8.37
	0.584	5	0.0374	8.12	5.54	8.31

* The value is from the bar spacing specified in the Standard.

12-inch and 14-inch slab thicknesses, the reinforcement in one-layer reinforcement is even higher than that in two-layer reinforcement, but the predicted crack spacings for one-layer reinforcement are larger than those for two-layer reinforcement. It may be relevant to mention that, although there is a general relationship between steel reinforcement and crack spacing, (i.e., the higher the steel reinforcement, the lower the crack spacing due to the greater restraint of the reinforcement on the concrete volume change), that relationship is influenced by other factors. From Tables 3.3 and 3.4, it appears that the crack spacing is more related to the ratio of bond area to concrete volume (Q), than to steel reinforcement alone. A correlation study was done on these data (for both SRG and LS aggregates) to determine which parameter, Q or steel reinforcement, has as stronger association with the structural responses in CRCP. Table 3.5 summarizes the results, which show Q has a stronger association with crack spacing and crack width than steel reinforcement alone. This result agrees with the findings in the field (Refs 18 and 20). The Texas SDHPT built experimental CRCP test sections in 1964 in Houston. On that particular study, Q was held constant and the percent steel was varied from 0.3 to 0.5. After 20 years, it was found that the crack spacings for all percentages of steel were practically the same(Ref 22).

Although most states do not consider bond area in designing longitudinal reinforcement, the 1972 "AASHO Interim Guide" suggests that Q be greater than 0.003 inch^2/inch^3. The Q values for 13, 14, and 15-inch thicknesses with

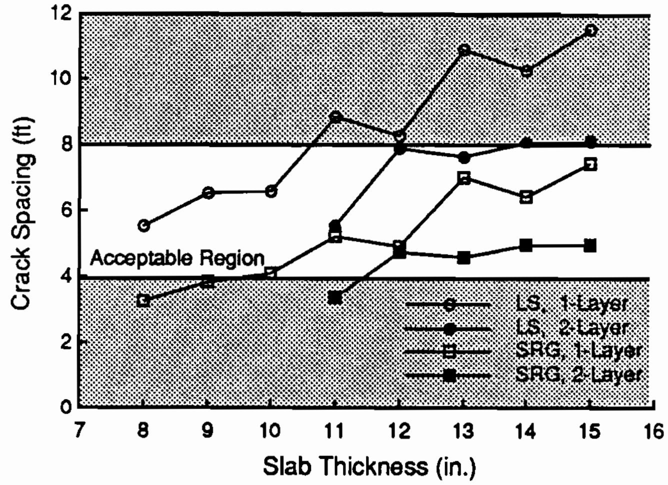


Fig 3.6. Predicted crack spacings for various slab thicknesses using CRCP-2.

TABLE 3.5. THE CORRELATION BETWEEN DESIGN PARAMETERS AND STRUCTURAL RESPONSES

Coarse Aggregate	Structural Responses	Design Parameters	Correlation Coefficient
SRG	Crack spacing	Percent steel	-0.460
		Q	-0.732
	Crack width	Percent steel	-0.455
		Q	-0.742
LS	Crack spacing	Percent steel	-0.478
		Q	-0.737
	Crack width	Percent steel	-0.470
		Q	-0.754

one-layer reinforcement in the SDHPT Standard need to be increased when the AASHO Interim Guide recommendation is considered.

Crack Width

The basic concept behind CRCP is that numerous hairline cracks will develop and the cracks will be kept tightly closed. Tightly closed cracks ensure structural continuity by good load transfer across cracks and by preventing the infiltration of foreign materials, which eliminates the problems transverse joints have caused.

Load transfer is thought to be achieved by moment transfer into the adjacent slab, aggregate interlock, and dowel action of the steel reinforcement. McCullough et al (Ref 23) found, however, that the effect of moment transfer was highly questionable, and that the other two mechanisms provided most of the load transfer. If aggregate interlock is sufficiently effective, the structural continuity of the slab is, to a great extent, maintained and there will be little differential vertical slab movement when a wheel load is applied near a crack. On the other hand, if cracks are not kept closed, the proportion of load transfer carried by aggregate interlock will decrease, and the dowel action of the longitudinal steel becomes more important. However, looseness that develops between steel surfaces and concrete around cracks under the action of repeated loads tends to reduce load transfer. Decrease in load transfer results in an increase in differential vertical slab movement across cracks. This differential vertical movement subjects the longitudinal steel to cyclic shear stress under repeated loading.

Another problem related to a loss of aggregate interlock is the high concrete stress level near a crack due to wheel load. Steel reinforcement in CRCP is not intended to increase the structural capacity of the slab to resist bending, but merely to control crack width. The structural capacity of the slab to resist bending at a crack largely comes from aggregate interlock. When there is not sufficient aggregate interlock and a wheel load is applied close to the edge, a "corner condition" develops near the crack. Thus, high concrete tensile stresses develop at the top of the slab, increasing the chances of a punchout.

Wide crack widths allow the infiltration of water and incompressible foreign materials into the crack. The infiltration of water causes problems such as corrosion of reinforcing steel, erosion of subbase materials, and pumping, creating voids underneath the slab. A slab experiences constant volume changes due to temperature variations. When the temperature increases, the slab expands, and blow-ups result from the incompressible foreign materials in the crack. Wide crack widths also increase the chances of spalling, by increasing the shear stress at the slab surface near a crack when a wheel load is applied.

It was found that crack width is a more sensitive indicator of pavement condition than the mean crack spacing (Ref 24). In this sense, for good performance of the pavement, it is very important to keep cracks tightly closed.

Effort was made to quantify the relationship between crack width and load transfer (Ref 21), however it is not currently feasible to set a design criterion for crack width from a load transfer standpoint. In Research Report 177-17 (Ref 5), the design criteria for crack width were established from the standpoint of controlling spalling, steel corrosion, and subgrade or subbase erosion.

Crack width is an outcome of the interaction of concrete drying shrinkage and creep, and thermal contraction due to temperature drop, along with the restraint due to slab-base friction and the embedded steel.

As may be seen in Fig 3.7, the rate of drying shrinkage decreases rapidly with time:

* 14 to 34 percent of the 20-year shrinkage occurs in 2 weeks,
* 40 to 80 percent of the 20-year shrinkage occurs in 3 months, and
* 66 to 85 percent of the 20-year shrinkage occurs in one year (Ref 18).

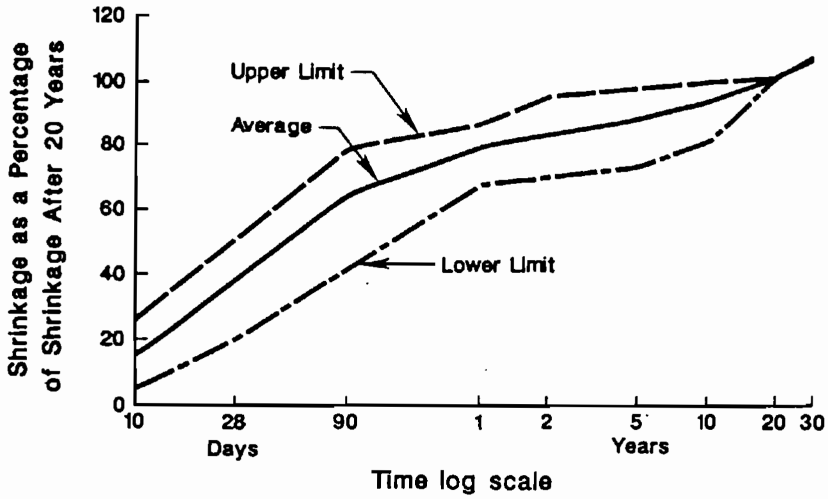


Fig 3.7. Range of shrinkage-time curves for different concrete stored at relative humidities of 50 and 70 percent (Ref 14).

The increase in the rate of drying shrinkage after one year is very small. Consequently, temperature can be considered the only variable that affects the variation in the value of crack width after the crack spacing has been stabilized. However, while the crack width varies according to the temperature change, spalling occurs over a long period of time during which the pavement temperature varies over a large range. Therefore, the maximum crack width allowed in any design should be determined for the lowest temperature to which the pavement will be subjected. From the standpoint of controlling spalling, the allowable crack width (for the maximum temperature drop in this study of 68°F) is 0.047 inch (Ref 5).

The maximum allowable crack width based on the permeability restriction is 0.025 inch, if the pavement is continuously flooded and left constantly at a temperature just above freezing. Neither of these two extreme conditions is likely to occur constantly throughout the pavement life, and the 0.025-inch level should be adjusted according to the climatological characteristics of the design site.

Figure 3.8 presents predicted maximum crack widths for various slab thicknesses. Predicted maximum crack width values for some thicknesses exceed the limiting values. The figure also shows that the crack width for the pavement with LS aggregate is larger than that for the pavement with SRG aggregate. Figure 3.9 presents a relationship between predicted crack spacing and crack width for both aggregates. It shows that crack width is directly proportional to crack spacing in a linear relation, which agrees with the field observation. It also shows that, for the same crack spacing, the crack width in LS concrete is less than that in SRG concrete, which is the result of the lower values of thermal coefficient and drying shrinkage for LS concrete. Since crack width is directly proportional to crack spacing, the ratio of bond area to concrete volume is more significant than percent steel in explaining the variation in the crack width (Table 3.5).

Steel Stress

In order to achieve the primary objective for using steel reinforcement in CRCP, to maintain transverse cracks in a tightly closed condition, it is necessary that steel at the crack be under the elastic range while the slab expands and contracts due to temperature changes. Tables 3.6 and 3.7 show the design criteria established to safeguard against steel rupture and permanent deforma-

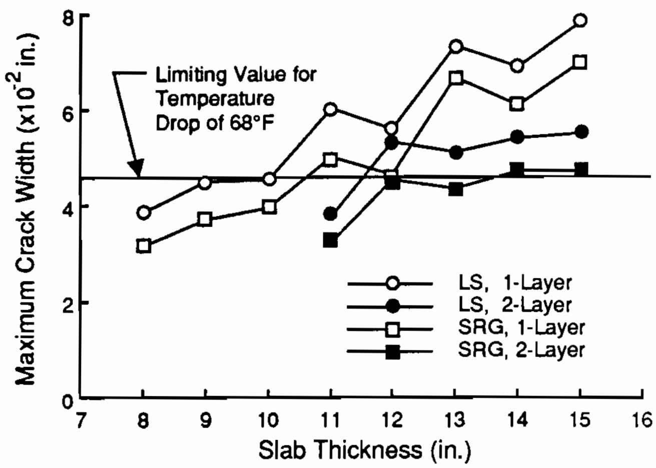


Fig 3.8. Predicted maximum crack widths for various slab thicknesses using CRCP-2.

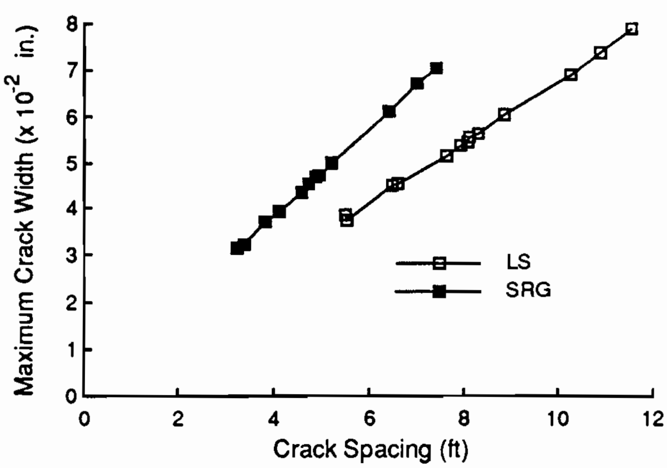


Fig 3.9. The relationship between crack spacing and maximum crack width.

TABLE 3.6. MAXIMUM ALLOWABLE STEEL STRESS TO PREVENT STEEL RUPTURE IN CRCP (REF 5)

Steel Type and Grade	Minimum Yield Strength, f_y (ksi)	Ultimate Strength, f_u (ksi)	Allowable Stress, f_8 (ksi)
Billet Steel			
Grade 40	40	60	52.5
Grade 60	50	90	67.5
Grade 75	100	100	75.0
Rail Steel			
Grade 50	50	80	60.0
Grade 60	60	90	67.5

1 ksi = 6.895 MPa

tion (Ref 5). Since Grade 60 ASTM A615 billet steel is usually used for longitudinal reinforcement, the allowable stress to prevent steel rupture is 67.5 ksi and the maximum allowable steel stress for control of permanent deformation is from 53.5 ksi for #7 bar to 71 ksi for #4 bar.

Figure 3.10 presents predicted maximum steel stress for various slab thicknesses. In SRG concrete pavement, the steel stress in thicknesses of less than 11 inches meets the above criteria, but the steel stress in thicknesses greater than or equal to 11 inches is somewhat higher than the above criteria. In LS concrete pavement, the steel stress is a little higher than the above criteria for all thicknesses. It also shows that the steel stress in LS concrete pavement is higher than that in SRG concrete pavement. Since crack spacing and crack width in LS concrete pavement are larger than those in SRG concrete pavement, the steel stress in LS concrete pavement is also higher than that in SRG concrete pavement.

In a field study, for a given bar size, steel strain at the crack was directly proportional to the crack width (Ref 25). However, Figs 3.8 and 3.10 indicate that steel stress is not directly proportional to crack width. This is explained by differences in bond characteristics due to various bar sizes used in this study. The smaller the bar size, the more effective the stress transfer from the steel to the concrete for a given percent longitudinal steel reinforcement because the greater number of bars provided by the smaller bar size results in a larger total bond area. The effective stress transfer for a smaller bar size results in a decrease of the bond development length. For a given crack width, a decrease in the bond development length results in a higher steel strain. This explains why there is little difference in steel stress between one- and two-layer reinforcements, while there is a big difference in crack width.

TABLE 3.7. MAXIMUM ALLOWABLE STEEL STRESS FOR CONTROL OF PERMANENT DEFORMATION (REF 5)

		Maximum Allowable Steel Stress (psi)	
		Concrete Compressive Strength, f_c'' (psi)	
Steel Yield Strength	Steel Bar Diameter, ϕ (inch)	Low, $f_c'' < 3{,}500$ psi (≥ 4.5 percent Air Content or ≤ 4 cement sacks per cubic yard of concrete)	Regular, $f_c'' \geq 3{,}500$ psi (≤ 4.5 percent content and ≥ 4 cement sacks per cubic yard of concrete)
40,000	0.500	60,200	64,900
	0.625	49,300	54,900
	0.750	43,400	47,300
	0.875	39,900	42,700
50,000	0.500	61,600	68,700
	0.625	53,000	57,500
	0.750	48,200	51,400
	0.875	45,400	47,700
60,000	0.500	65,100	71,000
	0.625	57,900	61,600
	0.750	53,900	56,500
	0.875	51,600	52,500
75,000	0.500	72,300	77,000
	0.625	66,600	69,500
	0.750	63,400	65,500
	0.875	61,500	63,000

1 inch = 25.4 mm
1 psi = 6.895 kPa

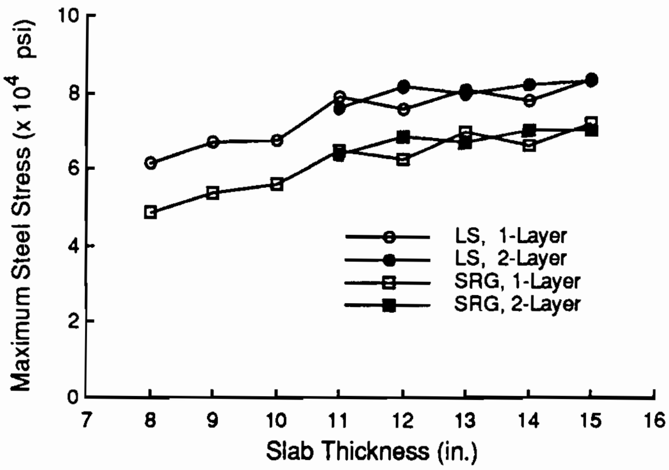


Fig 3.10. Predicted maximum steel stresses at a crack for various slab thicknesses using CRCP-2.

ANALYSIS FOR IMPROVEMENTS OF CRCP DESIGN STANDARD

The results of the previous analysis indicate that the structural responses are better explained by Q values than by steel reinforcement alone. This means the combination of percent steel and bar size, rather than steel reinforcement alone, should be considered in CRCP design. Thus a study of the effect of various Q values on the structural behavior may achieve a better design.

The ratio of bond area to concrete volume can be expressed in terms of the design variables.

For unit length of the slab,

$$Q = \frac{A_b}{V_c} = \frac{\pi \, \phi \, n \, l}{W \, h \, l} = \frac{\pi \, \phi \, n}{W \, h} \tag{3.1}$$

and

$$p = \frac{A_s}{A_c} = \frac{\pi}{4} \phi^2 \frac{n}{W h} = \frac{\pi \, \phi \, n}{W h} \times \frac{\phi}{4} \tag{3.2}$$

From Eqs 3.1 and 3.2, we get

$$p = Q \frac{\phi}{4}$$

i.e.,

$$Q = \frac{4p}{\phi} \tag{3.3}$$

in which A_b and V_c are the bond area and concrete volume, respectively; A_s and A_c are the steel and concrete cross-sectional areas, respectively; n is the number of bars in the cross section; W is the width of the slab; h is the slab thickness; f is the bar diameter; and p is the percent longitudinal steel reinforcement.

As can be seen in Eq 3.3, various Q values are obtained by changing steel reinforcement and/or bar size. For steel reinforcement, 0.6 percent is a well established and widely used value for CRCP. Hence, percent steel was fixed at 0.6, and, in order to obtain various Q values, bar sizes ranging from #4 through #7 were selected. For the other input variables, the values used in the previous analysis were selected. As in the previous analysis, two types of aggregates, SRG and LS, were considered. Since the bar size and slab thickness are the only variables for the given aggregate type, this study shows the effect of the variation in Q and slab thickness on the structural responses. In Appendix B, the results of this analysis are presented. In the following section, the results for crack spacing, crack width, and steel stress are discussed.

Crack Spacing

Figures 3.11 and 3.12 present the predicted crack spacing for various bar sizes and slab thicknesses for SRG and LS concrete pavements, respectively. Since the only difference in the graphs for a given slab thickness is the Q value, these graphs vividly show the effect of Q on crack spacing.

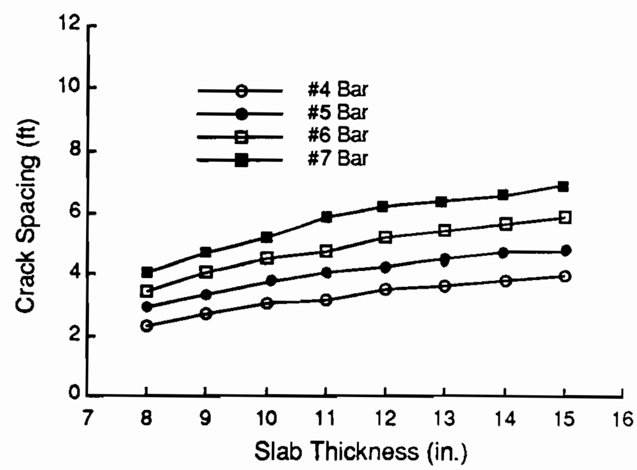


Fig 3.11. Predicted crack spacings for various bar sizes and thicknesses of slabs with SRG aggregate.

From the figures, three observations can be made:

(1) LS concrete gives larger crack spacing than SRG concrete. A literature survey shows that thermal coefficient and drying shrinkage of LS concrete are lower than those of SRG concrete. When concrete has a low value of thermal coefficient and low drying shrinkage, the concrete volume change also becomes less, resulting in large crack spacing. This explains the larger crack spacing for LS concrete.

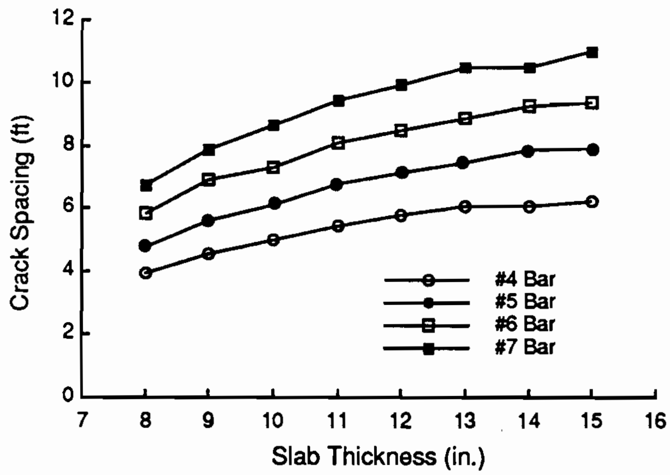


Fig 3.12. Predicted crack spacings for various bar sizes and thicknesses of slabs with LS aggregate.

(2) A larger size bar gives larger crack spacing. The effect of the restraint of steel reinforcement on concrete volume change causes concrete stress to develop, and, when the concrete stress developed exceeds the concrete tensile strength, a crack occurs. The degree of restraint from steel reinforcement largely depends on the availability of steel surface area: the larger the steel surface area, the higher the degree of restraint. The high degree of restraint on concrete volume change causes high concrete stress, resulting in smaller crack spacing. For a given steel reinforcement, a smaller size bar provides a larger steel surface area; and, in turn, a higher degree of restraint on concrete volume change. This explains why a smaller size bar gives smaller crack spacing.

(3) The thicker the pavement slab, the larger the crack spacing. In CRCP, initial cracking is caused mainly by the restraint on concrete volume change due to temperature and moisture change. Additional cracking is developed when the external wheel load is applied. According to Westergaard (Ref 26), the stress due to the wheel load at the bottom of the slab is inversely proportional to the slab thickness. When the sum of the stresses due to concrete volume change and due to the application of wheel load exceeds the concrete tensile strength, a crack will develop. Since the thicker the slab, the less the concrete stress due to the application of wheel load, crack spacing is larger for thicker slabs. Another reason is that as discussed in the next section, the rate of concrete stress increase in the bond slippage zone decreases with slab thickness. Hence, less concrete stress develops for thicker slabs, resulting in larger crack spacing for thicker slabs.

Crack Width

Crack widths for various slab thicknesses are presented in Fig 3.13 for SRG and in Fig 3.14 for LS concrete. Comparison of the figures indicates that the crack width for LS concrete is larger than that for SRG concrete and that larger size bars increase crack spacing, which in turn produces wider cracks. This is as expected, since the crack width is a function of crack spacing. Figure 3.15 presents the crack spacing and crack width relationship for various bar sizes and coarse aggregate types. It shows that the crack width is directly proportional to the crack spacing in a linear relationship regardless of bar size used. It also shows that the relationship is affected by the coarse aggregate types used. In Ref 27, the relative bonding efficiencies of eight bar types were studied a supplementary series of bond tests. Figure 3.16 shows the reinforcement bars used in the study. Figure 3.17 shows that an approxi-

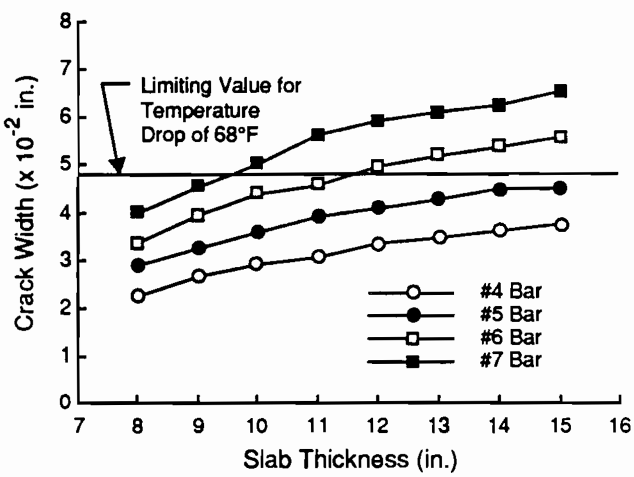


Fig 3.13. Predicted maximum crack widths for various bar sizes and thicknesses of slabs with SRG aggregate.

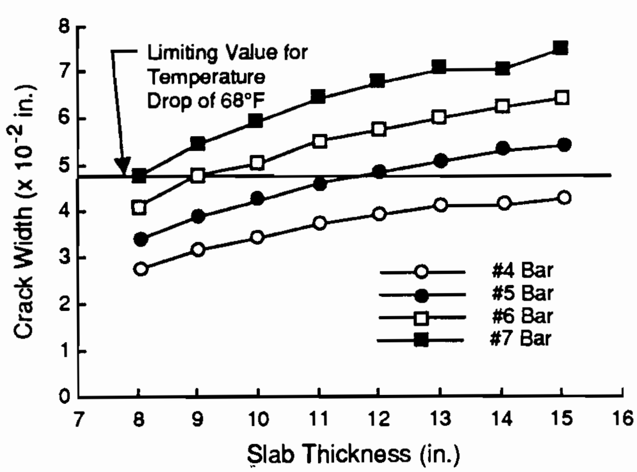


Fig 3.14. Predicted maximum crack widths for various bar sizes and thicknesses of slabs with LS aggregate.

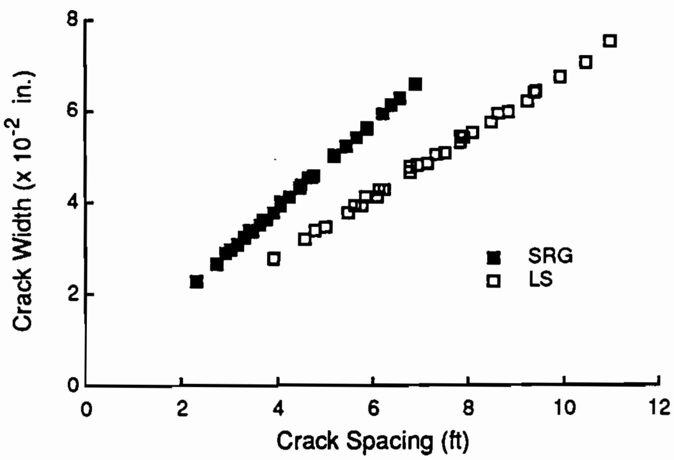


Fig 3.15. The relationship between crack spacing and crack width.

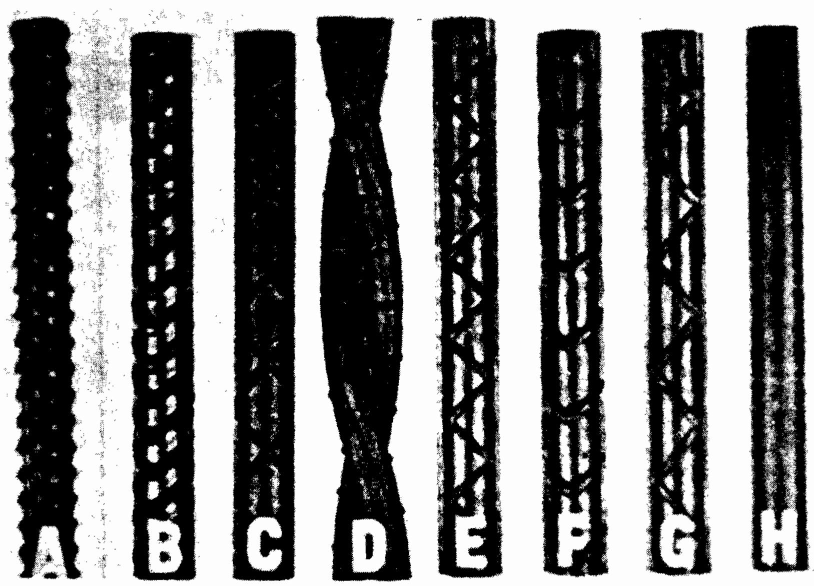


Fig 3.16. Reinforcement bars used in the experiment in Ref 27.

mately linear relation exists between crack spacing and crack width obtained in the laboratory tests for various types of bars. In this test, the steel stress at the crack was maintained at 40,000 psi. The bearing area of lugs is maximum for bar A, followed by B, C, D, E, F, G, and minimum for H. In the test, the steel reinforcement was kept at the same value for various bar types. Hence, any difference in the test results is attributed to the bond efficiency between concrete and steel among various bar types. Figure 3.17 illustrates that although bond efficiency causes various crack spacing (the higher the bond efficiency, the smaller the crack spacing and crack width), the crack spacing and crack width relationship is not affected by bond efficiency. In other words, for a given design combination, there is a unique linear relationship between crack spacing and crack width. Concrete properties, such as thermal coefficient and drying shrinkage, are the only variables affecting the relationship as shown in Fig 3.15. This is an important design aspect which should be considered in CRCP design, and suggests that different designs be developed for concretes with different coarse aggregate types.

Steel Stress

The steel stress at a crack should not exceed the yield stress, since, once the steel stress reaches the yield stress, the steel cannot resist more force, resulting in wide crack width. Figures 3.18 and 3.19 show the predicted maximum steel stress for various bar sizes and slab thicknesses for SRG and LS concrete pavements, respectively. Even though the predicted steel stresses are the values when the temperature is the lowest, they exceed the yield stress for thicker slabs. The figures show that, as far as steel stress is concerned, bar size has no effect.

The crack spacing-steel stress at a crack relationships for various bar sizes are shown in Fig 3.20 for SRG concrete and in Fig 3.21 for LS concrete. They indicate that, for the same crack spacing, the larger the bar size, the lower the steel stress. This was explained in the previous section in terms of the various development lengths for various bar sizes. A similar relationship exists between crack width and steel stress at a crack, as shown in Figs 3.22 and 3.23, for SRG and LS aggregate concretes, respectively. The curves in both graphs disclose the same general relationship between the variables, as might be expected, in view of the approximately linear relationship between crack spacing and crack width shown in Fig 3.15.

In Ref 27, the relationship between crack width and steel stress was investigated for the various bar types shown in Fig 3.16. As shown in Fig 3.24, the relationship between crack width and steel stress at a crack varies, depending on the bond efficiency. For a given crack width, bar A, with the maximum bearing area of lugs, developed maximum steel stress while the plain bar, H, developed minimum steel stress. Since, for deformed bars, bonding efficiency depends on the bar size, the relationships shown in Figs 3.22 and 3.23 should be considered in the development of the optimum CRCP design.

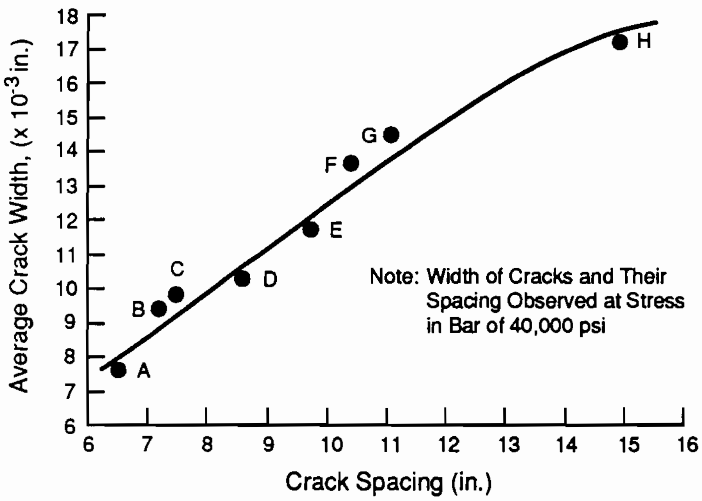


Fig 3.17. The relationship between crack spacing and crack width (A through H represent the bar types shown in Fig 3.16) (Ref 27).

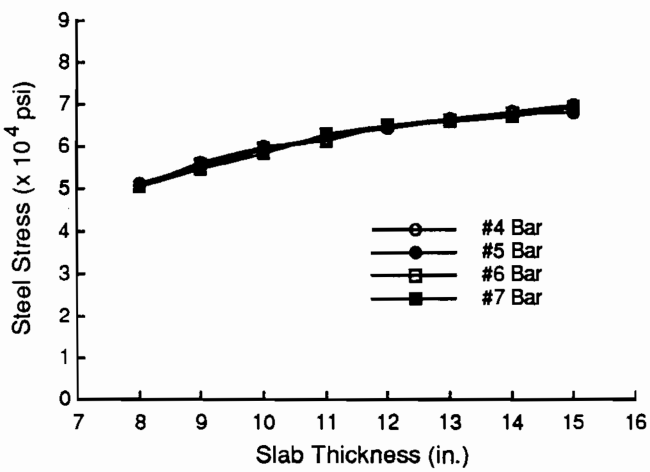


Fig 3.18. Predicted maximum steel stress at a crack for various bar sizes and slab thicknesses with SRG aggregate.

DISCUSSION OF THE RESULTS

In the study described in the previous sections, it was found that different concrete properties resulting from different coarse aggregate types have a significant effect on the structural responses of CRC pavements. It was also found that the proposed SDHPT Design Standard is reasonable for most of the slab thicknesses. However, the results of the previous section indicate that some modifications are needed in the Design Standard, as follows:

For SRG concrete:

(1) for an 8-inch thickness, the steel reinforcement should be reduced to 0.6 percent.

(2) for an 11-inch thickness with two-layer reinforcement, the Q value (0.0476) is too high, which results in too narrow crack spacing. The Q value should be reduced by using #5 bars instead of #4 bars.

For LS concrete:

(1) for 13,14, and 15-inch thicknesses with one-layer reinforcement, the Q value should be increased to 0.038. This can be done by using #5 bars instead of #7 bars.

(2) for an 11-inch thickness with one-layer reinforcement, the steel reinforcement should be increased to 0.6 percent.

The study of the effect of the Q factor in terms of various bar sizes on the structural responses shows that the Q factor plays an important role in CRCP behavior. The importance of the Q factor is discussed by McCullough and Ledbetter in Ref 20. Mechanistically, it is shown in the study by McCullough et al (Ref 2). Consider a free body of a CRCP slab segment in the bond slip zone which is under the effect of volumetric change (Fig 3.25). At the crack, the steel is under considerable tension since no resistance is provided by the concrete. Beyond the crack, steel stress is transferred to the surrounding concrete, creating a variable stress in the bar. Since the bar must be in equilibrium, this change in the bar force is resisted at the contact surface between the steel and concrete by an equal and opposite force produced by the bond between the steel and concrete. For the free body of the steel bar segment shown in Fig 3.25, if U is the magnitude of the average bond force per unit length of the bar, then $\Sigma F_x = 0$ yields

$$F_s + dF_s - F_s - U dx = 0 \text{ or } \frac{dF_s}{dx} = U \qquad (3.4)$$

Assuming that the bond force per unit length, U, is the resultant of shear type bond stresses, u, uniformly distributed over the contact area, then

$$U = u p f \qquad (3.5)$$

and

$$F_s = \frac{\sigma_s p \phi^2}{4} \qquad (3.6)$$

where

p is percent longitudinal steel reinforcement,

f is bar diameter, and

σ_s is steel stress.

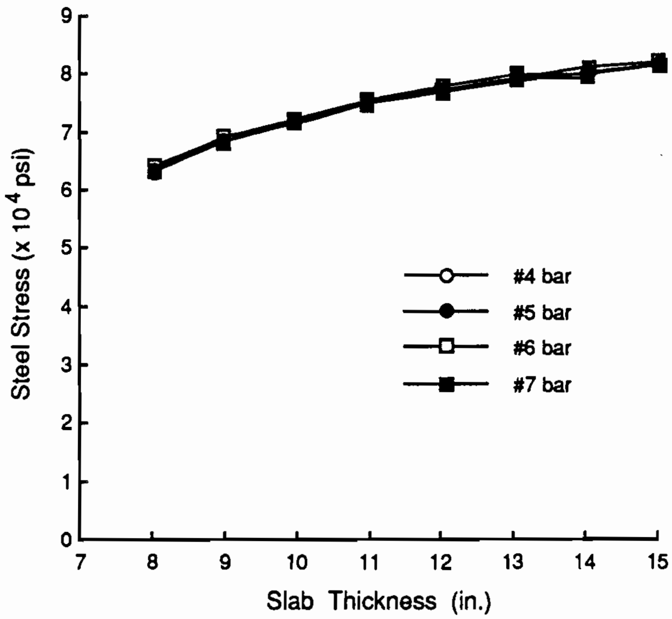


Fig 3.19. Predicted maximum steel stress at a crack for various bar sizes and thicknesses of slabs with LS aggregate.

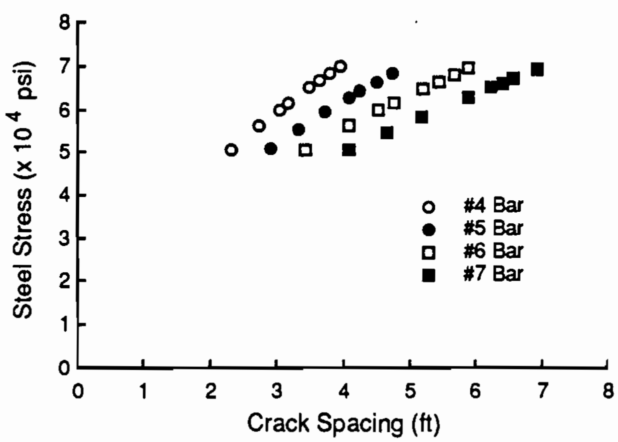


Fig 3.20. The relationship between crack spacing and steel stress at a crack for SRG aggregate.

Substituting the values of U and F_s in Eq 3.4 gives

$$\frac{d\sigma_s}{dx} = \frac{4u}{\phi} \tag{3.7}$$

Equation 3.7 indicates that the rate of the steel stress change in the bond development zone is directly proportional to the bond stress and inversely proportional to the bar diameter.

For the free body of the concrete segment shown in Fig 3.25, applying $\Sigma F_x=0$ gives

$$dF_c + F_i\, dx + U\, dx = 0$$

or

$$\frac{dF_c}{dx} = - F_i - U \tag{3.8}$$

Since $F_c = A_c s_c$ and $A_s = p f^2/4$, where A_c and A_s are the concrete and steel cross-sectional areas, respectively, and s_c and p are the concrete stress and steel reinforcement, respectively, Eq 3.8 can be written for a unit width slab as

$$\frac{d\sigma_c}{dx} = - \frac{F_i}{D} - \frac{4uP}{\phi} \tag{3.9}$$

in which F_i is friction force per unit length along the slab, s_c is the concrete stress, u is the bond stress, and D is the slab thickness. Equation 3.9 shows that the rate of the increase in the concrete stress within the bond development zone is a function of subbase frictional resistance, slab thickness, bond stress, and the Q factor. The maximum bond stress available for a given concrete volume change largely depends on the bonding efficiency. The bonding efficiency depends not only on the physical shape of the bar surface but on the available steel surface area. For the same steel reinforcement, a smaller bar size provides a larger steel surface area. ACI (Ref 11) recognizes this and specifies that, for tension bars, bond stress is governed by

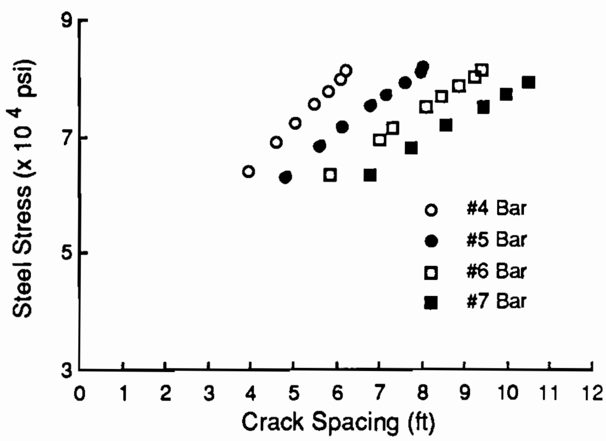


Fig 3.21. The relationship between crack spacing and steel stress at a crack for LS aggregate.

$$\frac{9.5 \sqrt{f'_c}}{\phi} \leq 800 \text{ psi} \tag{3.10}$$

where

f'_c is compressive strength of concrete.

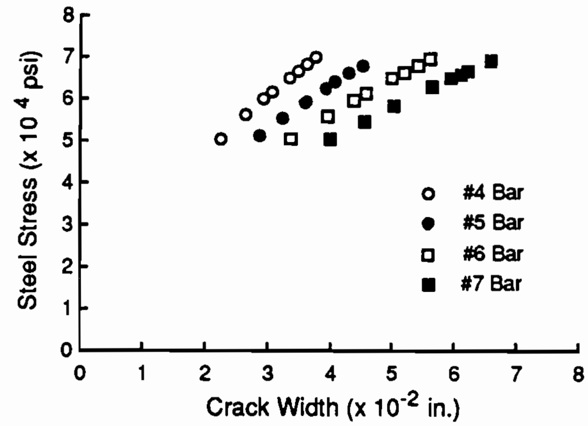


Fig 3.22. The relationship between crack width and steel stress at a crack, with SRG aggregate.

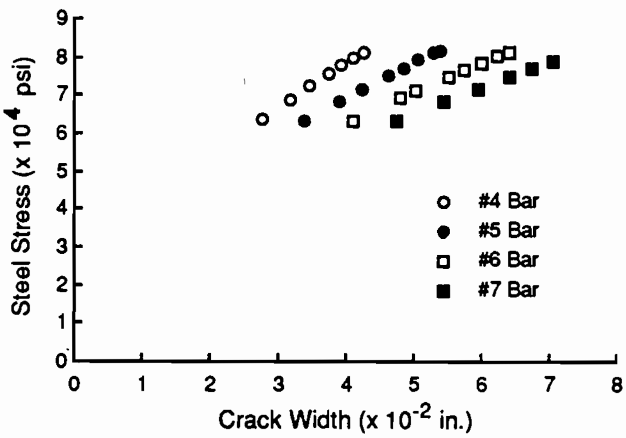


Fig 3.23. The relationship between crack width and steel stress at a crack, with LS aggregate.

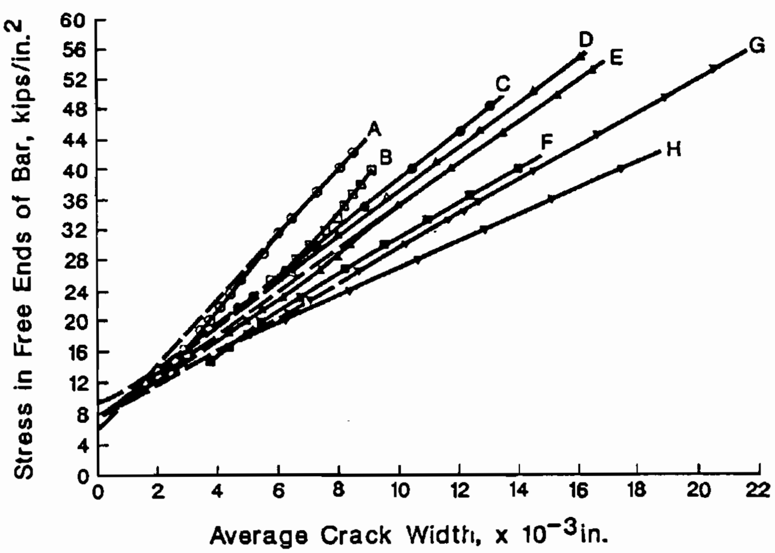


Fig 3.24. The relationship between crack width and steel stress (A through H represent the bar types shown in Fig 3.16) (Ref 27).

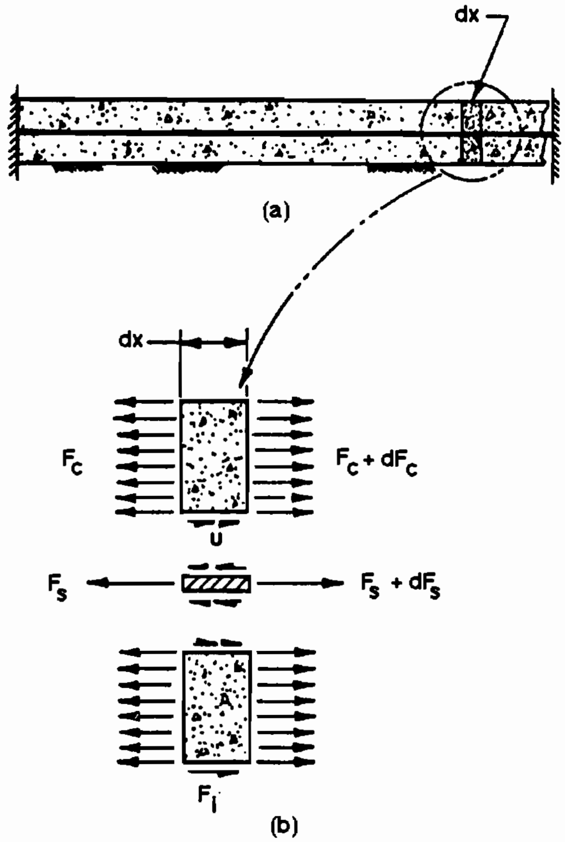


Fig 3.25. Free-body diagram of an element in the CRCP model in the bond-slip zone (Ref 2).

Equation 3.10 indicates that, for concretes with the same strength, using a smaller size bar gives a higher available bond stress. From Eqs 3.9 and 3.10, it is expected that, for a given steel reinforcement, using smaller size bars increases the available bond stress and Q, resulting in the higher rate of concrete stress increase and, in turn, smaller crack spacing. On the other hand, using larger size bars will result in larger crack spacing. This explains why the ratio of bond area to concrete volume is so important in CRCP slab behavior. Figure 3.26 shows the concrete stress distribution along the slab for various Q values obtained from computer program CRCP-2. The input values of all the other variables except Q were kept the same. It shows vividly the effect of Q on the rate of concrete stress increase in the bond development zone and on the crack spacing. Note that the higher the Q value, the smaller the bond development length and the higher the rate of concrete stress increase. The higher rate of concrete stress increase for a higher Q value resulted in a smaller crack spacing. This relationship between Q and crack spacing was confirmed by the field data (Ref 18).

Many factors affect the structural responses of CRCP. However, concrete properties, such as tensile strength, shrinkage and thermal coefficient of concrete, steel percentage, and Q values are the controllable design variables with a large effect. From this standpoint, it is necessary to distinguish different coarse aggregate types and to include Q effect in CRCP design.

A rational design should include the regional environmental effect, and the CRCP-2 computer program can be used for this purpose.

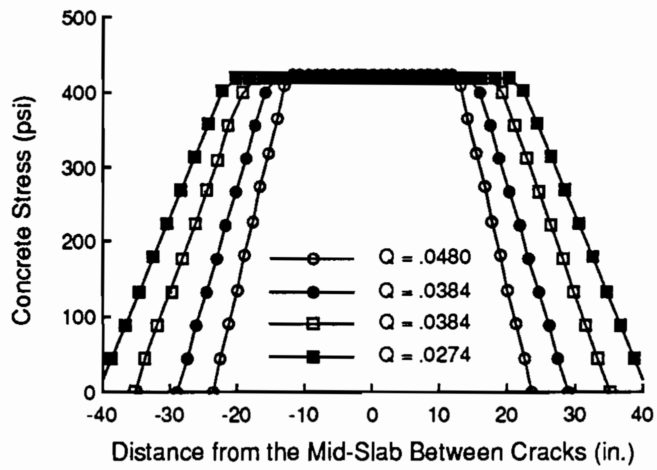


Fig 3.26. Concrete stress distribution along the slab length between the cracks for various Q values.

CHAPTER 4. EVALUATION OF CRCP DESIGN STANDARD FOR DEFORMED WIRE FABRIC REINFORCEMENT

Welded wire fabric is prefabricated reinforcement consisting of a parallel series of high-strength, cold-drawn wires welded together in square or rectangular grids. Each wire intersection is electrically resistance welded by a continuous automatic welder. Pressure and heat fuse the intersecting wires into a homogeneous section and fix all wires in their proper positions.

There are two types of wire fabric: smooth welded wire fabric and deformed welded wire fabric; smooth wire fabric is no longer used in pavement because it has given unsatisfactory performance (Ref 28). Deformed welded wire fabric utilizes both wire deformations and welded intersections for bond and anchorage.

INPUT VALUES FOR THE ANALYSIS

As described in Chapter 2, there are five categories of input variables in the CRCP-2 program, i.e., steel properties, concrete properties, slab-base friction, environmental inputs, and external load characteristics.

For slab-base friction, environmental inputs, and external load characteristics, the values used for the study of deformed bar reinforcement were selected.

TABLE 4.1. INPUT VALUES FOR THE EVALUATION OF THE CRCP (DEFORMED WIRE FABRIC) STANDARD

(1) Steel Properties	
(a) Reinforcement	0.6 percent
(b) Bar diameter	0.356 inch (D10)
(c) Modulus of elasticity	29×10^6 psi
(d) Yield stress	60,000 psi
(e) Wire spacing	12, 16, 18, 20, and 24 inches
(2) Slab-base Friction (Multi-Linear Curve)	
(a) Maximum frictional pressure	2 psi
(b) Movement at sliding	0.06 inch
(3) Environmental Inputs	
(a) Curing temperature	85°F
(b) Minimum daily temperature for 28 days	75°F
(c) Minimum temperature in the first winter	17°F
(d) The number of days until the minimum temperature occurs	210 days
(4) External Load Inputs	
(a) Wheel load	9,000 pounds
(b) Wheel base radius	6 inches
(c) Modulus of subgrade reaction	640 psi/inch
(d) Number of days before the wheel load is applied	14 days
(5) Concrete Properties	See Table 3.2

The steel reinforcement was fixed at 0.6 percent, and five transverse wire spacings, i.e., 12, 16, 18, 20, and 24 inches, were selected.

A slab thickness of 8 inches and two aggregate types, i.e., siliceous river gravel and limestone, were studied in this analysis. Table 4.1 shows the input values selected for this study.

RESULTS OF THE ANALYSIS

The major functional difference between deformed bars and deformed wire fabric is that, for deformed bars, the steel stress at the crack is transferred to the surrounding concrete by the chemical adhesion and mechanical bond between concrete and steel, resulting in relative movement between steel and concrete for some distance from the crack. For deformed wire fabric reinforcement, the transverse wires act as flexural beams, restrained from bending by the compression of the surrounding concrete. Laboratory tests have shown that almost all of the steel stress at the crack is dissipated at each of the welded transverse wires adjacent to the crack (Ref 29).

It was found that there was no significant difference in the distress manifestations or in the rate of deterioration of riding quality between the two types of reinforcement, but the type of reinforcement had an effect on mean crack spacing; i.e., the crack spacing was higher for deformed bars (Ref 2). Table 4.2 presents the results of the analysis.

TABLE 4.2. PREDICTED STRUCTURAL RESPONSES FOR VARIOUS WIRE SPACINGS AND COARSE AGGREGATES

Wire Spacing (inches)	Structural Responses: Crack Spacing (feet)		Crack Width ($\times 10^{-2}$ in.)		Steel Stress ($\times 10^4$ psi)	
	SRG	LS	SRG	LS	SRG	LS
12	1.58	2.55	1.55	1.79	4.62	6.31
16	2.11	3.40	2.06	2.39	4.62	6.31
18	2.46	5.81	2.41	4.07	4.98	6.37
20	4.16	6.41	4.06	4.50	5.04	6.33
24	3.29	7.74	3.21	5.42	4.98	6.37

Crack Spacing

Figure 4.1 shows the predicted crack spacing for both aggregate types and transverse wire spacings. It shows that

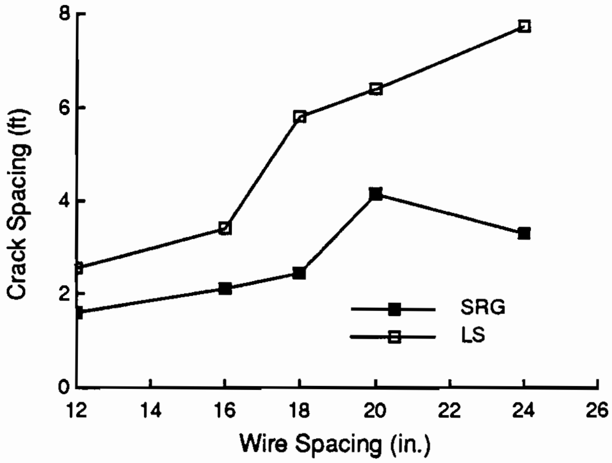


Fig 4.1. The effect of aggregate types and transverse wire spacings on crack spacing.

wider wire spacings generally give higher crack spacing and that there is an interaction between wire spacings and aggregate type, i.e., wire spacings and concrete properties.

Considering the limiting criteria described in the previous chapter, transverse wire spacings of and greater than 18 inches give satisfactory results for crack spacing. Figure 4.1 also shows that there is a considerable variation in predicted crack spacings for concrete with different aggregates.

Crack Width

Figure 4.2 presents the predicted crack width for both aggregates and transverse wire spacings. As described earlier, the crack width is temperature dependent, and, for the condition considered in this study, the maximum allowable crack width for controlling spalling is 0.047 inch. Comparison of Fig 4.1 with Fig

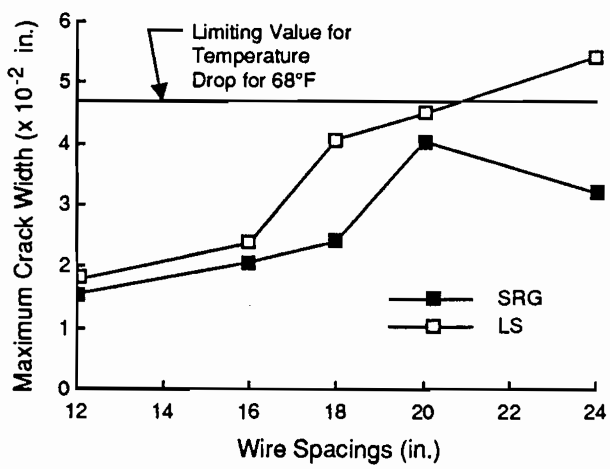


Fig 4.2. The effect of aggregate types and transverse wire spacings on crack width.

4.2 shows that crack spacing and crack width increase with increased wire spacings. Except for 24-inch wire spacing with LS aggregate, all the wire spacings are satisfactory for both aggregates as far as crack width is concerned.

Steel Stress

Figure 4.3 shows the maximum steel stress for both aggregate types and transverse wire spacings. Considering the limiting criteria for steel stress (Tables 3.6 and 3.7), all the wire spacings are satisfactory for both aggregate types.

In the previous chapter, for deformed bar reinforcement, steel stress was directly related to crack spacing or crack width for a given bar size. Figure 4.3 indicates transverse wire spacing does not have a practical effect on steel stress while it does on crack spacing and crack width. The reason for this difference in the relationship of crack width to steel stress between two types of reinforcement is that, in

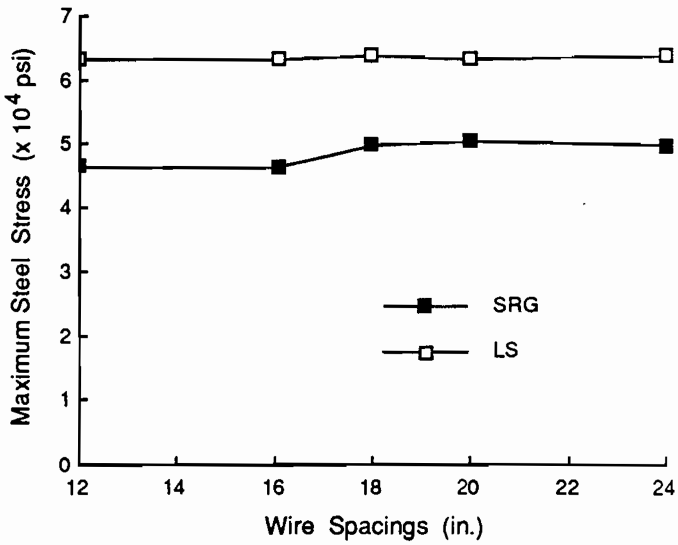


Fig 4.3. The effect of aggregate types and transverse wire spacings on steel stress.

deformed wire fabric reinforcement, bond development length is limited to the first transverse wire from the crack, resulting in almost identical steel strain for various transverse wire spacings. In other words, larger wire spacings result in larger crack width; however, because of its larger bond development length, steel strain at a crack is not significantly different from that at a smaller crack width resulting from smaller wire spacings.

It implies that, for this type of reinforcement, a large crack width, which occurs at early ages through a large crack spacing, might result in steel failure at a crack for smaller wire spacing.

Discussion of the Results

The study described above shows there is an interaction between wire spacings and structural responses. These interactions result from the bond anchorage characteristics of welded wire fabric. However, in general, larger transverse wire spacings produce larger crack spacings and crack widths, while the steel stress at the crack remains almost the same for various wire spacings. From the general trend, wire spacings of 18 inches or greater seem to be the optimum spacings for welded wire fabric reinforcement.

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

This report presents the results and findings of the study performed in order to evaluate the proposed Texas SDHPT CRCP Design Standard. This is a summary of the results and basic conclusions reached in this study .

CONCLUSIONS

(1) The Texas SDHPT CRCP(B)-85 Design Standard was evaluated using the theoretical model CRCP-2 with two coarse aggregates, i.e., siliceous river gravel and limestone. Basically, the Standard provided a good design with the following considerations.

  (a) For siliceous river gravel aggregate, two-layer reinforcement is more desirable with 13, 14, and 15-inch slab thicknesses. For an 11-inch-thick slab with two-layer reinforcement, #5 bars should be used instead of #4 bars.

  (b) For limestone aggregate, two-layer reinforcement is more desirable with thicknesses of 11 inches or greater. In 13, 14, and 15-inch thicknesses with one-layer reinforcement, the Q value should be increased by increasing the steel percentage and/or by using #6 bars instead of #7 bars.

(2) The CRCP (Deformed Wire Fabric) Design Standard was evaluated using the theoretical model CRCP-2 for SRG and LS aggregates. Wire spacings of 18 inches or greater should be used to provide the best performance.

(3) Structural responses in concrete pavement are the outcome of interactions between environmental conditions and material properties, and between wheel load applications and material properties. Thermal expansion, drying shrinkage, modulus of elasticity of concrete, and tensile strength are the concrete properties affecting the interactions with environmental conditions. Modulus of elasticity, modulus of rupture, and modulus of subgrade reaction are the variables affecting the interactions with wheel loading conditions. Hence, under identical environmental and wheel loading conditions, different structural responses, or pavement distresses, are developed for pavement constructed with materials of different characteristics. The minimum value currently required for modulus of rupture in the construction of concrete pavement is not sufficient for a satisfactory design standard. The different performances resulting from different material characteristics should be considered.

(4) A major factor causing cracking is embedded steel restraining the concrete volume change. The degree of restraint provided by steel reinforcement is an important design factor. The degree of restraint, hence, structural response, is better explained by the ratio of bond area to concrete volume (Q) than by steel reinforcement alone. In the development of CRCP design, consideration should be given to the selection of the Q values.

RECOMMENDATIONS

The following recommendations concerning the design of continuously reinforced concrete pavement resulted from this study.

(1) The major factors causing cracking in CRC pavement are drying shrinkage and thermal contraction of concrete. In order to achieve the desired pavement performance, regardless of coarse aggregate used, the development of different design standards for various aggregate types is recommended.

(2) The ratio of bond area to concrete volume is a very important variable in continuously reinforced concrete pavement. It is recommended that this variable be considered in the development of the SDHPT Design Standard. The optimum value of the ratio of bond area to concrete volume can be obtained through proper combinations of bar size and percent steel. The theoretical model CRCP-2 is recommended for this purpose.

(3) To provide reasonable Q values for slabs with thicknesses of more than 11 inches, two-layer reinforcement construction is recommended if feasible.

(4) To improve the crack width and steel stress predictions, the creep effect should be included in the theoretical model. This will require information on creep in tension.

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APPENDIX A

EXAMPLE OUTPUT OF COMPUTER PROGRAM CRCP-2

```
     PROJECT 472,  EVALUATION OF CRCP DESIGN STANDARD
     8-INCH SLAB, LS AGGREGATE


PROB
PROJEC  472

        ************************************************
        *                                              *
        *               STEEL PROPERTIES               *
        *                                              *
        ************************************************


             TYPE OF LONGITUDINAL REINFORCEMENT IS
                        DEFORMED BARS


             PERCENT REINFORCEMENT = 6.140E-01
             BAR DIAMETER          = 7.500E-01
             YIELD STRESS          = 6.000E+05
             ELASTIC MODULUS       = 2.900E+07
             THERMAL COEFFICIENT   = 5.000E-06


        ************************************************
        *                                              *
        *             CONCRETE PROPERTIES              *
        *                                              *
        ************************************************


             SLAB THICKNESS      = 8.000E+00
             THERMAL COEFFICIENT = 3.800E-06
             TOTAL SHRINKAGE     = 4.000E-04
             UNIT WEIGHT CONCRETE= 1.428E+02
             COMPRESSIVE STRENGTH= 4.500E+03


             TENSILE STRENGTH DATA AS INPUT BY USER

            AGE,   TENSILE
           (DAYS)  STRENGTH
               0      0
             1.0   164.6
             3.0   262.0
             5.0   309.4
             7.0   337.3
            14.0   384.9
            21.0   412.1
            28.0   425.0

        ************************************************
        *                                              *
        *       SLAB-BASE FRICTION CHARACTERISTICS     *
        *               F-Y RELATIONSHIP               *
        *                                              *
        ************************************************
```

TYPE OF FRICTION CURVE IS A MULTILINEAR CURVE

F(I)	Y(I)
0	-0
1.0000	-.0150
2.0000	-.0300
4.0000	-.0600
4.0000	-.0700
4.0000	-.0800
4.0000	-.0900
4.0000	-.1000
4.0000	-.1500

```
******************************
*                            *
*      TEMPERATURE DATA      *
*                            *
******************************
```

CURING TEMPERATURE= 85.0

DAY	MINIMUM TEMPERATURE	DROP IN TEMPERATURE
1	70.0	15.0
2	70.0	15.0
3	70.0	15.0
4	70.0	15.0
5	70.0	15.0
6	70.0	15.0
7	70.0	15.0
8	70.0	15.0
9	70.0	15.0
10	70.0	15.0
11	70.0	15.0
12	70.0	15.0
13	70.0	15.0
14	70.0	15.0
15	70.0	15.0
16	70.0	15.0
17	70.0	15.0
18	70.0	15.0
19	70.0	15.0
20	70.0	15.0

```
21          70.0          15.0
22          70.0          15.0
23          70.0          15.0
24          70.0          15.0
25          70.0          15.0
26          70.0          15.0
27          70.0          15.0
28          70.0          15.0

  DAYS BEFORE CONCRETE GAINS
  FULL STRENGTH                          =   28 DAYS
  MINIMUM TEMPERATURE EXPECTED AFTER
  CONCRETE GAINS FULL STRENGTH           = 17.0 DEGREES FAHRENHEIT
  DAYS BEFORE REACHING MIN. TEMP.        =210.0 DAYS
```

```
************************************************
*                                              *
*                EXTERNAL LOAD                 *
*                                              *
************************************************
```

```
WHEEL LOAD (LBS)          = 9.000E+03
WHEEL BASE RADIUS (IN)    = 6.000E+00
SUBGRADE MODULUS (PCI)    = 6.400E+02
CONCRETE MODULUS (PSI)    = 4.002E+06
LOAD APPLIED AT           =  14. TH DAY
CALC.LOAD STRESS (PSI)    = 1.557E+02
```

```
************************************************
*                                              *
*       ITERATION AND TOLERANCE CONTROL        *
*                                              *
************************************************
```

```
MAXIMUM ALLOWABLE NUMBER OF ITERATIONS=   60

RELATIVE CLOSURE TOLERANCE=  1.0 PERCENT
```

AT THE END OF THE ANALYSIS PERIOD

CRACK SPACING = 5.507E+00 FEET
CRACK WIDTH = 3.866E-02 INCHES
MAX CONCRETE STRESS= 4.064E+02 PSI
MAX STEEL STRESS = 6.135E+04 PSI,
CONC.TENS.STRENGTH = 4.602E+02 PSI

STA-TION	DIS-TANCE	CONCRETE MOVEMENT	FRICTION FORCE	CONCRETE STRESS	STEEL STRESS
1	0	0	0	4.188E+02	-6.431E+03
2	3.3	-1.856E-04	1.238E-02	4.187E+02	-6.431E+03
3	6.6	-3.713E-04	2.476E-02	4.187E+02	-6.431E+03
4	9.9	-5.569E-04	3.714E-02	4.187E+02	-6.431E+03
5	13.2	-7.425E-04	4.951E-02	4.187E+02	-6.431E+03
6	16.5	-9.281E-04	6.189E-02	4.187E+02	-6.431E+03
7	19.8	-1.114E-03	7.427E-02	4.187E+02	-6.431E+03
8	23.1	-1.299E-03	8.665E-02	4.187E+02	-6.431E+03
9	26.4	-1.485E-03	9.903E-02	4.187E+02	-6.431E+03
10	29.7	-1.671E-03	1.114E-01	4.187E+02	-6.431E+03
11	33.0	-1.856E-03	1.238E-01	4.187E+02	-6.431E+03
12	36.3	-2.042E-03	1.362E-01	4.187E+02	-6.432E+03
13	39.7	-2.228E-03	1.485E-01	4.187E+02	-6.432E+03
14	43.0	-2.413E-03	1.609E-01	4.187E+02	-6.432E+03
15	46.3	-2.599E-03	1.733E-01	4.187E+02	-6.432E+03
16	49.6	-2.784E-03	1.857E-01	4.187E+02	-6.432E+03
17	52.9	-2.970E-03	1.981E-01	4.187E+02	-6.432E+03
18	56.2	-3.156E-03	2.104E-01	4.187E+02	-6.432E+03
19	59.5	-3.341E-03	2.228E-01	4.187E+02	-6.432E+03
20	62.8	-3.527E-03	2.352E-01	4.187E+02	-6.432E+03
21	66.1	-3.713E-03	2.476E-01	4.186E+02	-6.432E+03
22	69.4	-3.898E-03	2.599E-01	4.186E+02	-6.432E+03
23	72.7	-4.084E-03	2.723E-01	4.186E+02	-6.432E+03
24	76.0	-4.270E-03	2.847E-01	4.186E+02	-6.432E+03
25	79.3	-4.455E-03	2.971E-01	4.186E+02	-6.432E+03
26	82.6	-4.641E-03	3.095E-01	4.186E+02	-6.432E+03
27	85.9	-4.826E-03	3.218E-01	4.186E+02	-6.432E+03
28	89.2	-5.012E-03	3.342E-01	4.186E+02	-6.433E+03
29	92.5	-5.198E-03	3.466E-01	4.186E+02	-6.433E+03
30	95.8	-5.383E-03	3.590E-01	4.185E+02	-6.433E+03
31	99.1	-5.569E-03	3.714E-01	4.185E+02	-6.433E+03
32	102.4	-5.755E-03	3.837E-01	4.185E+02	-6.433E+03
33	105.7	-5.940E-03	3.961E-01	4.185E+02	-6.433E+03
34	109.0	-6.126E-03	4.085E-01	4.185E+02	-6.433E+03
35	112.3	-6.312E-03	4.209E-01	4.185E+02	-6.433E+03
36	115.7	-6.497E-03	4.333E-01	4.184E+02	-6.433E+03
37	119.0	-6.683E-03	4.456E-01	4.184E+02	-6.433E+03
38	122.3	-6.869E-03	4.580E-01	4.184E+02	-6.434E+03
39	125.6	-7.054E-03	4.704E-01	4.184E+02	-6.434E+03
40	128.9	-7.240E-03	4.828E-01	4.184E+02	-6.434E+03
41	132.2	-7.426E-03	4.952E-01	4.183E+02	-6.434E+03
42	135.5	-7.611E-03	5.075E-01	4.183E+02	-6.434E+03
43	138.8	-7.797E-03	5.199E-01	4.183E+02	-6.434E+03
44	142.1	-7.983E-03	5.323E-01	4.183E+02	-6.434E+03
45	145.4	-8.168E-03	5.447E-01	4.183E+02	-6.435E+03
46	148.7	-8.354E-03	5.571E-01	4.182E+02	-6.435E+03
47	152.0	-8.540E-03	5.694E-01	4.182E+02	-6.435E+03
48	155.3	-8.725E-03	5.818E-01	4.182E+02	-6.435E+03
49	158.6	-8.911E-03	5.942E-01	4.182E+02	-6.435E+03
50	161.9	-9.097E-03	6.066E-01	4.181E+02	-6.435E+03
51	165.2	-9.282E-03	6.190E-01	4.181E+02	-6.436E+03
52	168.5	-9.468E-03	6.313E-01	4.181E+02	-6.436E+03
53	171.8	-9.654E-03	6.437E-01	4.174E+02	-6.326E+03
54	175.1	-9.840E-03	6.561E-01	4.087E+02	-4.916E+03
55	178.4	-1.003E-02	6.686E-01	4.000E+02	-3.507E+03
56	181.7	-1.021E-02	6.811E-01	3.913E+02	-2.097E+03
57	185.0	-1.040E-02	6.936E-01	3.826E+02	-6.869E+02

58	188.3	-1.059E-02	7.062E-01	3.740E+02	7.230E+02
59	191.7	-1.078E-02	7.188E-01	3.653E+02	2.133E+03
60	195.0	-1.097E-02	7.315E-01	3.566E+02	3.543E+03
61	198.3	-1.116E-02	7.442E-01	3.479E+02	4.953E+03
62	201.6	-1.135E-02	7.570E-01	3.392E+02	6.363E+03
63	204.9	-1.154E-02	7.698E-01	3.305E+02	7.772E+03
64	208.2	-1.174E-02	7.826E-01	3.218E+02	9.182E+03
65	211.5	-1.193E-02	7.955E-01	3.131E+02	1.059E+04
66	214.8	-1.212E-02	8.085E-01	3.044E+02	1.200E+04
67	218.1	-1.232E-02	8.214E-01	2.958E+02	1.341E+04
68	221.4	-1.251E-02	8.345E-01	2.871E+02	1.482E+04
69	224.7	-1.271E-02	8.475E-01	2.784E+02	1.623E+04
70	228.0	-1.291E-02	8.606E-01	2.697E+02	1.764E+04
71	231.3	-1.310E-02	8.738E-01	2.610E+02	1.905E+04
72	234.6	-1.330E-02	8.870E-01	2.523E+02	2.046E+04
73	237.9	-1.350E-02	9.003E-01	2.436E+02	2.187E+04
74	241.2	-1.370E-02	9.135E-01	2.349E+02	2.328E+04
75	244.5	-1.390E-02	9.269E-01	2.262E+02	2.469E+04
76	247.8	-1.410E-02	9.403E-01	2.175E+02	2.610E+04
77	251.1	-1.430E-02	9.537E-01	2.088E+02	2.751E+04
78	254.4	-1.450E-02	9.671E-01	2.001E+02	2.892E+04
79	257.7	-1.471E-02	9.807E-01	1.914E+02	3.033E+04
80	261.0	-1.491E-02	9.942E-01	1.827E+02	3.174E+04
81	264.4	-1.511E-02	1.008E+00	1.740E+02	3.315E+04
82	267.7	-1.532E-02	1.021E+00	1.653E+02	3.456E+04
83	271.0	-1.552E-02	1.035E+00	1.566E+02	3.597E+04
84	274.3	-1.573E-02	1.049E+00	1.479E+02	3.738E+04
85	277.6	-1.594E-02	1.063E+00	1.392E+02	3.879E+04
86	280.9	-1.614E-02	1.076E+00	1.305E+02	4.020E+04
87	284.2	-1.635E-02	1.090E+00	1.218E+02	4.161E+04
88	287.5	-1.656E-02	1.104E+00	1.131E+02	4.302E+04
89	290.8	-1.677E-02	1.118E+00	1.044E+02	4.443E+04
90	294.1	-1.698E-02	1.132E+00	9.572E+01	4.584E+04
91	297.4	-1.719E-02	1.146E+00	8.702E+01	4.725E+04
92	300.7	-1.740E-02	1.160E+00	7.831E+01	4.866E+04
93	304.0	-1.761E-02	1.174E+00	6.961E+01	5.007E+04
94	307.3	-1.783E-02	1.189E+00	6.090E+01	5.148E+04
95	310.6	-1.804E-02	1.203E+00	5.220E+01	5.289E+04
96	313.9	-1.825E-02	1.217E+00	4.349E+01	5.430E+04
97	317.2	-1.847E-02	1.231E+00	3.478E+01	5.571E+04
98	320.5	-1.868E-02	1.246E+00	2.607E+01	5.712E+04
99	323.8	-1.890E-02	1.260E+00	1.737E+01	5.853E+04
100	327.1	-1.912E-02	1.275E+00	8.657E+00	5.994E+04
101	330.4	-1.933E-02	1.289E+00	-5.283E-02	6.135E+04

PROJECT 472, EVALUATION OF CRCP DESIGN STANDARD

PROB
PROJEC 472

TIME (DAYS)	TEMP DROP	DRYING SHRINKAGE	TENSILE STRGTH	CRACK SPACING	CRACK WIDTH	MAXIMUM CONCRETE STRESS	MAXIMUM STRESS IN THE STEEL
.37	10.0	3.056E-11	99.6	514.1	7.653E-04	2.720E+01	4.792E+03
.50	15.0	2.458E-09	116.4	514.1	2.050E-03	4.654E+01	8.768E+03
1.50	15.0	7.326E-06	193.6	514.1	4.582E-03	8.334E+01	1.798E+04
2.50	15.0	3.629E-05	241.4	514.1	1.005E-02	1.409E+02	2.851E+04
3.50	15.0	7.204E-05	274.6	514.1	1.834E-02	2.069E+02	4.037E+04
4.50	15.0	1.054E-04	298.3	514.1	2.727E-02	2.600E+02	5.061E+04
5.50	15.0	1.344E-04	316.6	514.1	3.579E-02	2.760E+02	5.929E+04
6.50	15.0	1.589E-04	330.6	385.6	3.720E-02	2.917E+02	6.115E+04
7.50	15.0	1.797E-04	340.9	192.8	2.711E-02	3.175E+02	5.183E+04
8.50	15.0	1.975E-04	348.1	192.8	2.985E-02	3.215E+02	5.466E+04
9.50	15.0	2.127E-04	355.0	192.8	3.225E-02	3.383E+02	5.715E+04
10.50	15.0	2.259E-04	361.9	192.8	3.437E-02	3.421E+02	5.939E+04
11.50	15.0	2.374E-04	368.6	192.8	3.626E-02	3.513E+02	6.142E+04
12.50	15.0	2.475E-04	375.2	192.8	3.794E-02	3.617E+02	6.329E+04
13.50	15.0	2.565E-04	381.7	192.8	3.954E-02	3.682E+02	6.475E+04
14.50	15.0	2.645E-04	386.9	81.3	2.035E-02	3.742E+02	4.425E+04
15.50	15.0	2.716E-04	390.8	81.3	2.092E-02	3.817E+02	4.475E+04
16.50	15.0	2.781E-04	394.8	81.3	2.144E-02	3.886E+02	4.519E+04
17.50	15.0	2.839E-04	398.7	81.3	2.192E-02	3.912E+02	4.560E+04
18.50	15.0	2.892E-04	402.5	81.3	2.236E-02	3.952E+02	4.596E+04
19.50	15.0	2.941E-04	406.4	81.3	2.277E-02	3.994E+02	4.630E+04
20.50	15.0	2.985E-04	410.2	81.3	2.314E-02	4.042E+02	4.660E+04
21.50	15.0	3.026E-04	413.0	81.3	2.348E-02	4.063E+02	4.687E+04
22.50	15.0	3.064E-04	414.9	81.3	2.378E-02	4.082E+02	4.711E+04
23.50	15.0	3.099E-04	416.7	81.3	2.407E-02	4.108E+02	4.733E+04
24.50	15.0	3.131E-04	418.6	81.3	2.433E-02	4.126E+02	4.754E+04
25.50	15.0	3.161E-04	420.4	81.3	2.458E-02	4.137E+02	4.773E+04
26.50	15.0	3.190E-04	422.3	81.3	2.481E-02	4.163E+02	4.791E+04
27.50	15.0	3.216E-04	424.1	81.3	2.503E-02	4.182E+02	4.807E+04

APPENDIX B

RESULTS OF THE ANALYSIS FOR THE IMPROVEMENTS OF CRCP DESIGN STANDARD

TABLE B.1. CRCP STRUCTURAL RESPONSES FOR #4 BAR AND SRG AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x 10^{-2}$ inch)	Max Steel Stress ($x 10^{4}$ psi)
8	0.6	4	0.0480	2.31	2.25	5.06
9	0.6	4	0.0480	2.73	2.65	5.62
10	0.6	4	0.0480	3.04	2.93	5.98
11	0.6	4	0.0480	3.19	3.06	6.16
12	0.6	4	0.0480	3.49	3.34	6.50
13	0.6	4	0.0480	3.65	3.48	6.66
14	0.6	4	0.0480	3.80	3.61	6.82
15	0.6	4	0.0480	3.95	3.75	6.98

TABLE B.2. CRCP STRUCTURAL RESPONSES FOR #5 BAR AND SRG AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x 10^{-2}$ inch)	Max Steel Stress ($x 10^{4}$ psi)
8	0.6	5	0.0384	2.93	2.86	5.11
9	0.6	5	0.0384	3.33	3.23	5.53
10	0.6	5	0.0384	3.73	3.59	5.92
11	0.6	5	0.0384	4.08	3.92	6.25
12	0.6	5	0.0384	4.26	4.08	6.41
13	0.6	5	0.0384	4.50	4.29	6.61
14	0.6	5	0.0384	4.74	4.51	6.81
15	0.6	5	0.0384	4.74	4.51	6.81

TABLE B.3. CRCP STRUCTURAL RESPONSES FOR #6 BAR AND SRG AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x 10^{-2}$ inch)	Max Steel Stress ($x 10^{4}$ psi)
8	0.6	6	0.0320	3.44	3.36	5.05
9	0.6	6	0.0320	4.08	3.95	5.60
10	0.6	6	0.0320	4.53	4.37	5.97
11	0.6	6	0.0320	4.76	4.57	6.14
12	0.6	6	0.0320	5.21	4.99	6.48
13	0.6	6	0.0320	5.44	5.19	6.64
14	0.6	6	0.0320	5.67	5.40	6.80
15	0.6	6	0.0320	5.89	5.60	6.95

TABLE B.4. CRCP STRUCTURAL RESPONSES FOR # 7 BAR AND SRG AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x\ 10^{-2}$ inch)	Max Steel Stress ($x\ 10^{4}$ psi)
8	0.6	7	0.0274	4.09	3.99	5.04
9	0.6	7	0.0274	4.67	4.53	5.47
10	0.6	7	0.0274	5.19	5.01	5.83
11	0.6	7	0.0274	5.88	5.64	6.28
12	0.6	7	0.0274	6.23	5.95	6.49
13	0.6	7	0.0274	6.40	6.11	6.59
14	0.6	7	0.0274	6.57	6.27	6.70
15	0.6	7	0.0274	6.92	6.58	6.90

TABLE B.5. CRCP STRUCTURAL RESPONSES FOR # 4 BAR AND LS AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x\ 10^{-2}$ inch)	Max Steel Stress ($x\ 10^{4}$ psi)
8	0.6	4	0.0480	3.95	2.76	6.38
9	0.6	4	0.0480	4.58	3.17	6.89
10	0.6	4	0.0480	5.03	3.45	7.24
11	0.6	4	0.0480	5.49	3.74	7.57
12	0.6	4	0.0480	5.80	3.92	7.78
13	0.6	4	0.0480	6.10	4.11	7.98
14	0.6	4	0.0480	6.10	4.11	7.98
15	0.6	4	0.0480	6.24	4.26	8.13

TABLE B.6. CRCP STRUCTURAL RESPONSES FOR # 5 BAR AND LS AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x\ 10^{-2}$ inch)	Max Steel Stress ($x\ 10^{4}$ psi)
8	0.6	5	0.0384	4.82	3.38	6.30
9	0.6	5	0.0384	5.63	3.90	6.83
10	0.6	5	0.0384	6.16	4.23	7.16
11	0.6	5	0.0384	6.79	4.62	7.53
12	0.6	5	0.0384	7.14	4.84	7.72
13	0.6	5	0.0384	7.50	5.06	7.92
14	0.6	5	0.0384	7.86	5.28	8.10
15	0.6	5	0.0384	7.90	5.39	8.18

TABLE B.7. CRCP STRUCTURAL RESPONSES FOR # 6 BAR AND LS AGGREGATE

Slab Thickness (inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($x\ 10^{-2}$ inch)	Max Steel Stress ($x\ 10^{4}$ psi)
8	0.6	6	0.0320	5.85	4.10	6.34
9	0.6	6	0.0320	6.94	4.79	6.93
10	0.6	6	0.0320	7.32	5.03	7.13
11	0.6	6	0.0320	8.09	5.51	7.50
12	0.6	6	0.0320	8.48	5.75	7.68
13	0.6	6	0.0320	8.86	5.99	7.85
14	0.6	6	0.0320	9.25	6.22	8.02
15	0.6	6	0.0320	9.38	6.40	8.13

TABLE B.8. CRCP STRUCTURAL RESPONSES FOR # 7 BAR AND LS AGGREGATE

Slab Thickness (Inches)	Steel Percent	Bar No.	Q	Crack Spacing (feet)	Max Crack Width ($\times 10^{-2}$ inch)	Max Steel Stress ($\times 10^{4}$ psi)
8	0.6	7	0.0274	6.78	4.75	6.32
9	0.6	7	0.0274	7.86	5.44	6.83
10	0.6	7	0.0274	8.64	5.94	7.17
11	0.6	7	0.0274	9.43	6.42	7.50
12	0.6	7	0.0274	9.95	6.74	7.70
13	0.6	7	0.0274	10.48	7.06	7.91
14	0.6	7	0.0274	10.48	7.06	7.91
15	0.6	7	0.0274	11.00	7.50	8.16