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16. Abstract

This report addresses important factors associated with the design of steel reinforcement in terms of layer configuration, bond characteristic, climatic affect, and others relative to an assessment of the suitability of the CRCP 8 program to represent and predict steel stresses in continuously reinforced concrete (CRC) pavement systems. It was necessary to instrument an actual section of CRC pavement for concrete and steel strains as they fluctuated under climatic and seasonal changes. The steel rebars were instrumented in a manner that would limit disturbance of the bond between the steel and the concrete, yet allow for precise measurements of the steel strain at various distances from the crack face. Other field sections containing Grade 70 steel were also included in this study. Crack spacing and crack width data were collected and reported.

In light of this emphasis, a key aspect of the steel design considerations is how important parameters—such as the steel surface area, degree of bond, the grade of steel, and the amount of steel—relate to the maximum width that transverse cracks will be allowed to open over the design life of the pavement. Inherent in configuring the reinforcement in CRC pavement to perform a safe level below its yield limit is the maintenance of the transverse crack widths below specified levels to ensure adequate stiffness at the transverse cracks. Crack width data varied as a function of the distance from the pavement surface, and it was noted in the report that the vertical position of the steel within the slab affects this variation and consequently should be a consideration in determining the vertical position of the reinforcing layer in construction.

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SUMMARY OF ANALYSIS OF CRC PAVEMENT EXPERIMENTAL DATA USING GRADE 70 STEEL

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DISCLAIMER

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IMPLEMENTATION RECOMMENDATIONS

Findings have indicated that use of the CRCP 8 program for design purposes is very promising. Based on the results of this research project, the authors propose the following recommendations for TxDOT:

1. Pending further improvements in the CRCP 8 program, application of correction factors to the program results relative to steel stresses and crack widths, and
2. That future updates of the program code in terms of improved characterization of creep and drying shrinkage models are highly encouraged. Improvements of this nature will advance the overall utility of the program for use in project design and should eliminate the need to apply correction factors to the program results.

CHAPTER 1
PROJECT BACKGROUND

The purpose of this report is to provide background data, analysis, and information relative to the use and design of Grade 70 reinforcing steel configured in a single mat for the construction of continuously reinforced concrete (CRC) pavement. In order to develop a basis for this report, a CRC pavement test section was established on I-45 in North Central Houston near the FM 1960 interchange to establish a database of field-measured concrete and steel strains and movements in which to analyze relative to the identification and delineation of findings regarding the use of Grade 70 reinforcement. This report includes: 1) a brief theoretical discussion of the cracking behavior of CRC pavement in terms of environmentally and load-induced concrete and steel strains, 2) a description of the available analysis tools applicable to the behavior of CRC pavement systems, 3) an instrumented test site, 4) collected data categories, and 5) an analysis derived from the collected data. Verification of the available analytical models is accommodated through a variety of comparisons to the typical responses that characterize the structural behavior of CRC pavement systems.

CRC pavement, widely used in the Houston District, ideally should develop a transverse crack pattern that manifests average crack spacings and crack widths within certain performance limits. Although structural performance limits for CRC pavement with respect to crack spacing have been well established and delineated for several years [1, 2], performance limits with respect to the width of the transverse cracks have not, particularly in terms of structural design criteria. The consequence of this negligence is reflected in the lack of attention to crack width limits and their relationship to assured levels of load transfer efficiency as reflected in current versions of the *AASHTO Design Guide* and other design procedures for CRC pavements. Nonetheless, one of the purposes of longitudinal reinforcing steel in CRC pavement is to hold the widths of the transverse cracks within a certain range. Over the history of the development of the use of CRC pavement in the Houston District, performance limits relative to crack spacing have been emphasized and included in the design criteria and, to some extent, the factors which affect the development of the ultimate crack pattern. The percentage of steel reinforcement, bonding area between the reinforcing steel and the volume of concrete (q), coarse aggregate type, weather conditions at the time of construction, and the degree of bond between the steel and the concrete have been identified as the key factors that affect the characteristics of the cracking pattern (i.e., the average crack spacing and crack width). The first two are under the control of the design engineer towards meeting the criteria of the design.

Underlying the design engineer's choices of the controllable cracking factors is the selection of steel grade. The grade is selected to ensure that the stress levels in the reinforcing steel are at a safe level below the yield limit which is assured, according to design practice in the Houston District, by keeping the calculated stresses less than a limit of 75 percent of the yield strength. Although the basis of the 75 percent limit is not clearly supported, the same limit is used in the *AASHTO Design Guide*. Discussion and definition of a safe level below the yield limit is provided in the research report. The greatest strains in the reinforcing steel typically occur at the locations of the transverse cracks. It is generally accepted that the performance of CRC pavement would be compromised if the steel stress was allowed to exceed the yield strength at these locations. Yielding of the steel most likely would result in excessive crack widths causing loss of pavement stiffness and load transfer across the transverse cracks, which would dramatically affect performance. Unfortunately, this is the extent to which most CRC pavement design procedures consider the effect of crack width in the design process. Nonetheless, in terms of design and performance, it is important to understand how the steel reinforcement parameters (percent steel, bond area, yield strength, etc.) relate to the development of the crack pattern.

These parameters were of particular interest in this study with respect to the field experience that was gained from the Grade 70 CRC pavement sections placed in I-45 (previously noted) and on SH 249 in Houston. The SH 249 section consisted of pavement sections containing Grade 60 steel (at $p = 0.67$ percent steel and $q = 0.036$) and sections containing Grade 70 steel (at $p = 0.49$ percent steel and $q = 0.026$). Data collected from these sections since construction comparing the pavement crack patterns are shown in Figure 1.1 (along with average crack spacing and standard deviation data) at various ages after construction. This pavement, located near the Willow Brook Mall on SH 249 near Tomball, Texas, was constructed 13 inches thick during the last week of September 1996 and was actually the first project in the Houston District to incorporate Grade 70 steel. Grade 70 steel rebars in a single mat were used in place of Grade 60 steel rebars that were in a two-layer configuration. The accumulative crack spacing shown in Figure 1.1 (at various ages) is based upon crack spacings between adjacent consecutive cracks. The crack pattern, as characterized in this figure, is more favorably distributed in the Grade 60 section than in the Grade 70 section because the crack pattern is not as widely spaced. This trend was still evident 15 months after construction. As noted in Figure 1.1, the average crack spacing of the Grade 60 steel section was 5.2 ft, which was within the allowable range of the *AASHTO Guide*—3.5 ft to 8 ft—but the average crack spacing of the Grade 70 steel section was 9.9 ft and, as surveyed in December 1997, was far beyond the upper limit of 8 ft. However, in terms of cluster cracking, the Grade 70 section showed better characteristics than the Grade 60 section if consideration is given to the spacing between groups of two adjacent consecutive cracks and groups of five adjacent consecutive cracks. A comparison of this nature, shown in Figure 1.2, serves as a measure of cluster cracking which can be derived from distributions made from these groupings. Cluster cracking is the occurrence of adjacent or consecutive groups of closely and widely spaced transverse cracks and is considered to be an undesirable feature in the crack pattern. It is characterized in terms of the cluster ratio (CR) as:

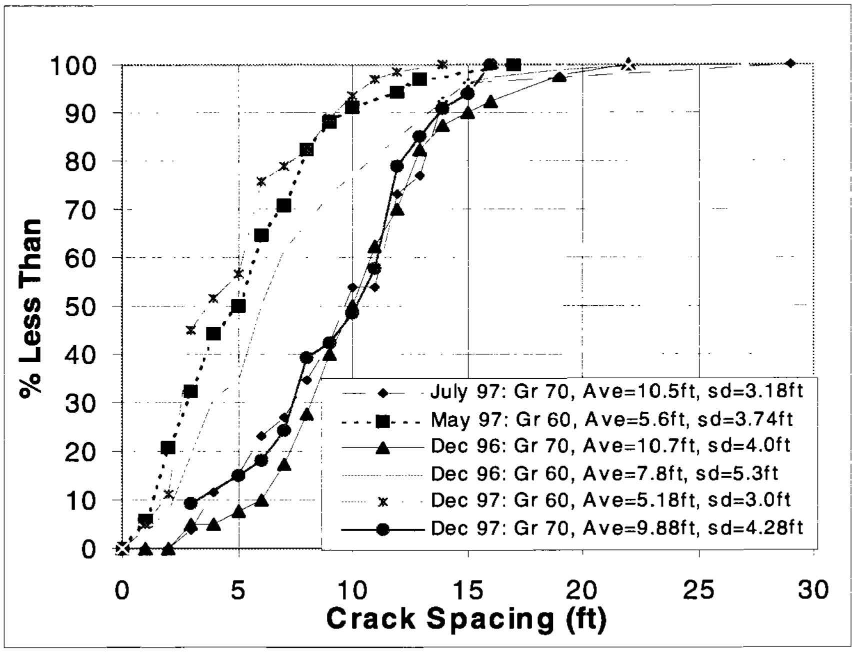

Figure 1.1 CRC Pavement Crack Spacing Distribution—SH 249, Houston District Grade 60 and 70 Sections.

$$CR = \left[1 - (NC - 1) * \frac{X_1}{X_2}\right] * 100$$

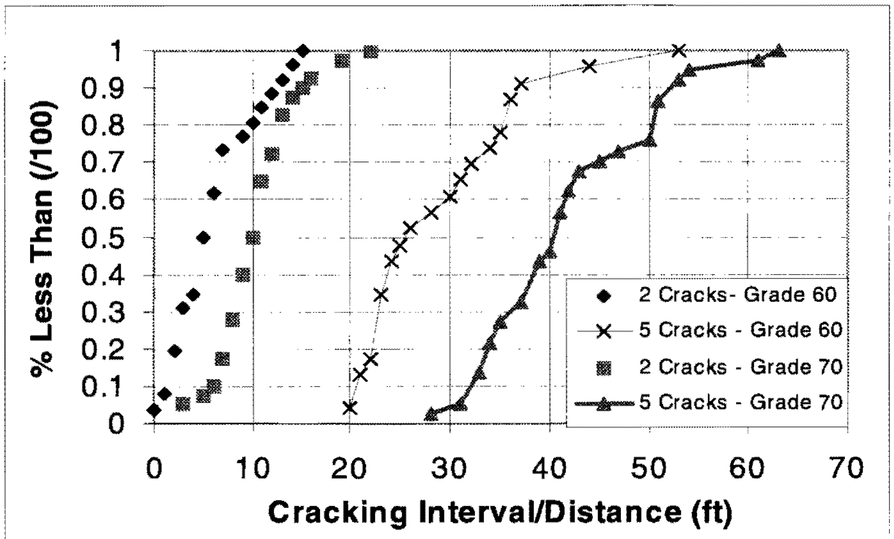

Figure 1.2 Cluster Cracking: Grade 60 and 70, SH 249.

where X_1 is the crack spacing that corresponds to a given probability of being less than all possible crack spacings in the pavement; X_2 is the distance between the first and last crack of a given number of consecutive cracks at a given probability of being less than all possible consecutive cracks of the same number, and NC is the number of consecutive cracks considered at a time (which was five in this case). A perfect crack pattern would display 0 percent clustering, but a clustering level of 20 percent should be considered acceptable [3]. Although the details associated with the development and application of the cluster cracking concept are explained elsewhere [3], the Grade 60 CRC sections indicated 31 percent clustering while the Grade 70 section showed only 10 percent. It should be pointed out, the lower clustering manifested by the Grade 70 pavement section has less to do with the grade of steel and more to do with the use of one layer of steel reinforcement and the variability of the curing process.

Although all the crack widths on SH 249 sample sections were below the limit established by the *AASHTO Guide*, the Grade 70 steel section presented larger average crack widths. An average crack width of 19.8 mils in the Grade 70 steel section was observed in January 1997 [4], much larger than the average crack spacing of 6.2 mils in the Grade 60 steel section observed at the same time. Crack width distribution data surveyed in July 1997 comparing both Grade 60 and Grade 70 steel sections on SH 249 (Figures 1.3 and 1.4) indicated nearly similar average crack widths but very different crack width distributions, as noted in the figures. The standard deviation of the Grade 70 was calculated at 6.6 mils, and the Grade 60 was 3.4 mils. Crack width (and crack width deviation) has an important effect on CRC pavement performance.

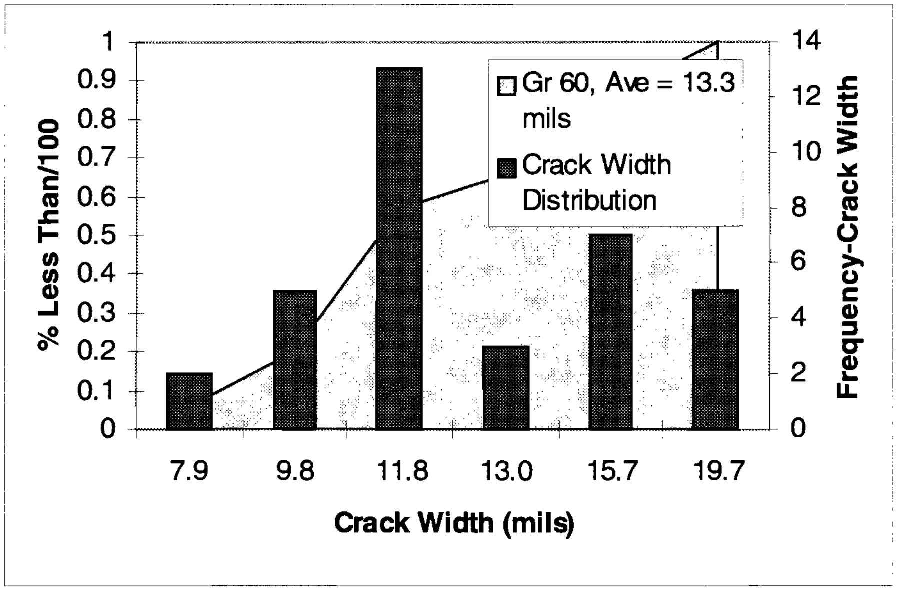

Figure 1.3 Crack Width Distributions for the Grade 60 SH 249 Pavement Section—July 1997.

The trends in crack width between the Grade 60 and Grade 70 sections continued to be manifest in later surveys (Figure 1.5), with the Grade 70 section cracks developing greater widths. In addition to the wider crack trends, the Grade 70 steel section also manifests noticeably more minor severity spalling at the transverse cracks. This difference may be due to the wider crack widths displayed by the Grade 70 section. Again, the differences manifested in crack spacing, crack

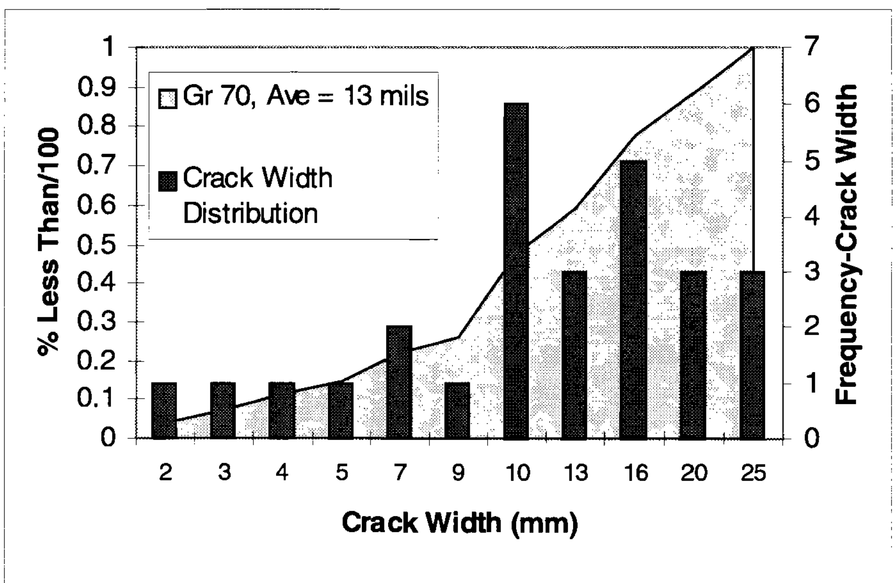

Figure 1.4 Crack Width Distributions for the Grade 70 SH 249 Pavement Section—August 1997.

widths, and crack spalling between the Grade 60 and the Grade 70 sections on SH 249 have less to do with the grade of the reinforcing steel and more to do with the q factor (or the amount of steel). The use of Grade 70 steel appears to have some merit, and the intent of this report is to examine the feasibility of using this grade of reinforcing steel in CRC pavement (particularly in a single layer configuration). It is important to point out that key findings can be derived from the performance observations of the SH 249 Grade 70 section—primarily that q factor and weather conditions at the time of construction must be carefully considered in the design and construction of CRC pavement systems. An important aspect is the sensitivity of the q factor and construction weather conditions to crack width and their combined effect on design requirements relative to the selected grade of reinforcing steel.

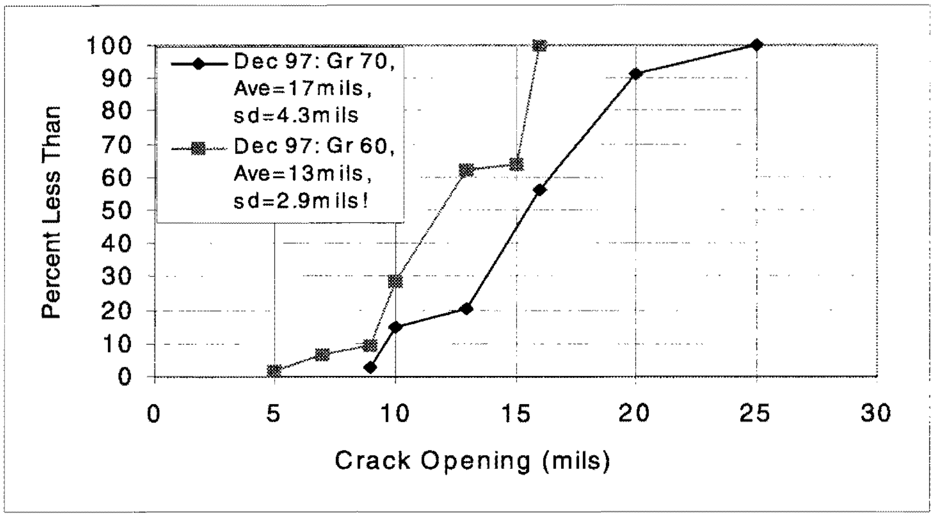

Figure 1.5 CRC Pavement Crack Width Distribution—SH 249, Houston District Grade 60 and 70 Sections.

Project Objectives

The objectives associated with this study are as follows:

1. Instrument Grade 70 longitudinal reinforcing bars; place them in actual CRC pavements, and monitor the strains in the bars during the placement and hardening of the concrete for selected days during the development of the cracking pattern.

2. Conduct an evaluation of the behavior of the instrumented sections based upon analysis of the collected test data using the CRCP 8 computer program. Assess the suitability of the CRCP 8 program to predict steel and other strains related to the structural behavior of CRC pavement systems.

3. Summarize findings from the analysis relative to the use of Grade 70 reinforcing steel in the construction of CRC pavements in the Houston District. The results of this investigation will be the provision of data and information relative to the best use of Grade 70 steel in CRC pavement construction.

CHAPTER 2

TEST SECTION DATA COLLECTION AND SYNTHESIS

Relative to the objectives of this project, a section of CRC pavement constructed on a section of I-45 in North Houston was instrumented in order to monitor the behavior of both the reinforcing steel and the concrete. The resulting data were also used to assess the predictability of current analytical models based on the interaction between the steel reinforcement and the concrete.

In this project, it was of prime interest to analyze measured steel strain and calculated slip displacements of steel reinforcing bars in CRC pavements based on instrumented measurement of strains of the steel bars and the concrete in the field as part of the effort to develop input data for the CRC pavement analysis models. Strain gages were installed in the concrete adjacent to the strain gages in the reinforcement, and movements in the steel and concrete were measured directly by these strain gages. These measured strains were used to evaluate the stresses in the reinforcement. Strain measurements of this nature made it possible to determine interaction between the reinforcement and the concrete. From strains in the steel and concrete, slip displacements between the steel and the concrete along the steel bar were calculated. These results were then compared with those obtained from computer simulations performed using the inputs derived from the analyzed data.

Test Site and Data Collection

The instrumented pavement segment is located on the southbound lanes of I-45 in Houston, about one-third of a mile south of FM 1960. The instrumented segment was a CRC pavement that was placed on August 22, 1997, as part of a 555 ft long pavement construction section. The pavement was paved 15 inches thick, on a 1 inch thick asphalt bond breaker. In addition, a 6 inch thick stabilized base course and a 6 inch thick lime-treated subgrade were placed underneath the asphalt bond breaker. A single layer of grade 70 steel reinforcement (representing 0.49 percent steel) was placed near the mid-plane of the pavement slab.

The construction materials used for the pavement section included use of #6-sized Grade 70 steel reinforcement and a crushed limestone concrete mixture. Grade 70 steel reinforcement possesses a minimum yield strength of 72.5 ksi. The #6 reinforcement has a nominal diameter of 0.75 in and a nominal area of 0.44 in^2. The reinforcement was approximately placed at a 6-inch spacing interval. The details of the concrete mix proportion used for this project are presented in Table 2.1.

Table 2.1 Concrete Mixture Proportions Used for I-45 Site.

CAF	0.68 - Limestone Redland Gr #2 BSG_{ssd} = 2.56 $DRUW_{ssd}$ = 95.84	**% Air**	5.0 Daravair
CF	6.0 Texas Lehigh	**WF**	4.5
% Fly Ash	25	**WRA**	4-8 oz/100 wt Lubricon - R

Data Synthesis

Each of the computer simulation programs (the CRCP 8 and the TTICRCP programs) required a good deal of input information about the pavement in question. Many of the program inputs do not change over time, but some of the inputs are a function of time. However, in this report it is assumed the reader is familiar with the type of inputs required for these programs. Consequently, only limited discussion of the program inputs will be given, but some attention was given to the characterization of key parameters such as strength or the time of setting. A general list of inputs is displayed in Table 2.2 and was obtained from the design plans for the project or through laboratory testing. The concrete set temperature was based on the concrete temperature at the time of final setting as defined by ASTM C 403. However, McCullough and Schindler [6] recommended using 93 percent of the peak temperature as the input set temperature (which in this case is 111 °F). The concrete coefficients of thermal expansion (CTE) was also evaluated using the concrete strain readings from the gage nearest the induced crack face and temperature data at days 30, 162, and 270. These values are listed at the bottom of Table 2.2. The later two values are approximately double what was expected, which may be explained in part due to the effect of moisture levels in the concrete at the level of the steel reinforcement that is less than saturation in the concrete at that point in time. Neville [5] indicated this effect can insignificantly increase the CTE of concrete. The ultimate shrinkage listed in Table 2.2 was based on laboratory shrinkage data following a procedure similar to that prescribed in ASTM C 157. The general inputs presented here were used in both of the computer simulation programs.

Table 2.2 Computer Simulation Inputs.

Initial Crack Spacing (in)	120
Steel Diameter (in)	0.75
# of Steel Layers	1
E - Steel (psi)	30600000
Coeff. of Thermal Contraction (Steel)	5.0E-6
Steel Spacing (in)	6
Slab Thickness (in)	15
Ultimate Shrinkage	0.000700
Curing Temperature (°F)	106
Concrete (CTE) x 10^{-6}/°F	6.1*
Wheel Load (lb)	0
Tire Contact Radius (in)	0
E - Subgrade (psi)	100

*Note: CTE at days 30, 162, and 270 were determined to be 5.5, 14.2, and 13.2 microstrains/°F, respectively.

The time-dependent inputs required for each simulation program were distinct. In order to provide a thorough comparison of the computer-simulated predictions with the actual measured values, the research team chose to make comparisons at four different stages of the pavement life. The pavement ages chosen for the comparisons were at 16, 30, 162, and 270 days from the placement of the concrete. All pertinent data dealing with the concrete and steel strain, the crack widths, the pavement temperature, etc. were measured at each of these points in time. Some of the data, such as concrete strength, had to be estimated for the last two time periods because measurements were unfeasible due to further construction in the surrounding area.

Steel Stress and Strain

It was of interest to develop bond stress and slip data from the field measurements, which primarily involved the steel strain data. The first step in this process required developing charts showing the steel strain as a function of distance from the crack face (Figure 2.1). While not used as a program input itself, the change in strains and stresses with distance measured in the reinforcing steel was needed in order to calculate the bond stress and slip. The applied force present in the reinforcing steel was calculated from the data collected by the data logger. These data were then used to calculate the strain in the reduced cross section (or cross-sectional area of the gaged area, which was smaller than the total bar cross-sectional area) and the stress in the steel reinforcement. The strain in the cross section was obtained by dividing the force value by the constant of proportionality (C). The steel stress was obtained by dividing the force calculated for the reinforcing bar by the total cross-sectional area of the bar. Theoretically, these curves should coincide, but due to experimental error, some differences (although small) exist between them. The resulting values are illustrated in Figure 2.1, showing strain and stress as a function of distance from the induced crack. Based on theoretical trends described in Chapter 3 of Report 4925-1, "Analysis of Field Monitoring Data of CRC Pavements Constructed with Grade 70 Steel," it appears the data points at 36 inches from the crack face are in error, as

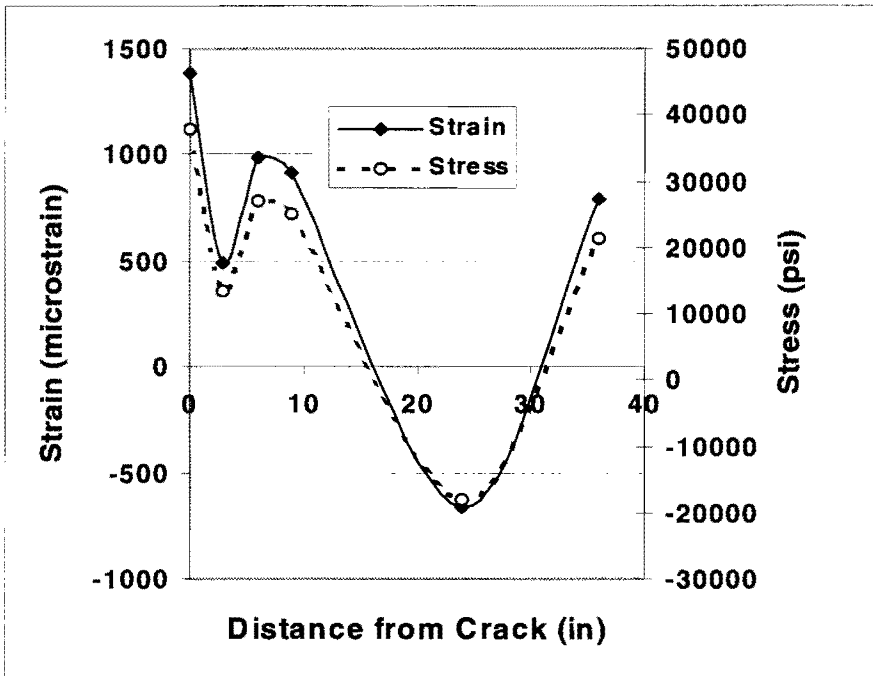

Figure 2.1 Steel Stress/Strain Versus Distance from the Induced Crack as Measured on Day 30. (Note: Horizontal axis is vertically centered.)

are the data 3 inches from the crack face. The maximum stress indicated in Figure 2.1 is 37,700 psi, and the maximum stress of 43,600 psi was recorded on day 162 in January 1998.

Concrete Stress and Strains

The concrete data, similar to the steel data, were required in order to obtain values for the bond behavior. Concrete gages were used to measure the strain in the concrete surrounding the reinforcing steel. The concrete strain behavior was very different from the steel strain behavior in that the concrete gage reading indicated a degree of relaxation. In other words (Figure 2.2), the development of stress in the concrete is directly related to the restrained level of strain or the restrained strain, which is a component of strain that must be extracted from the reading of the concrete strain gage. The restrained strain is determined by calculating the difference between the free shrinkage strain (at a certain point in time) and the strain indicated by the concrete gage (less the amount of creep that has occurred at that point in time).

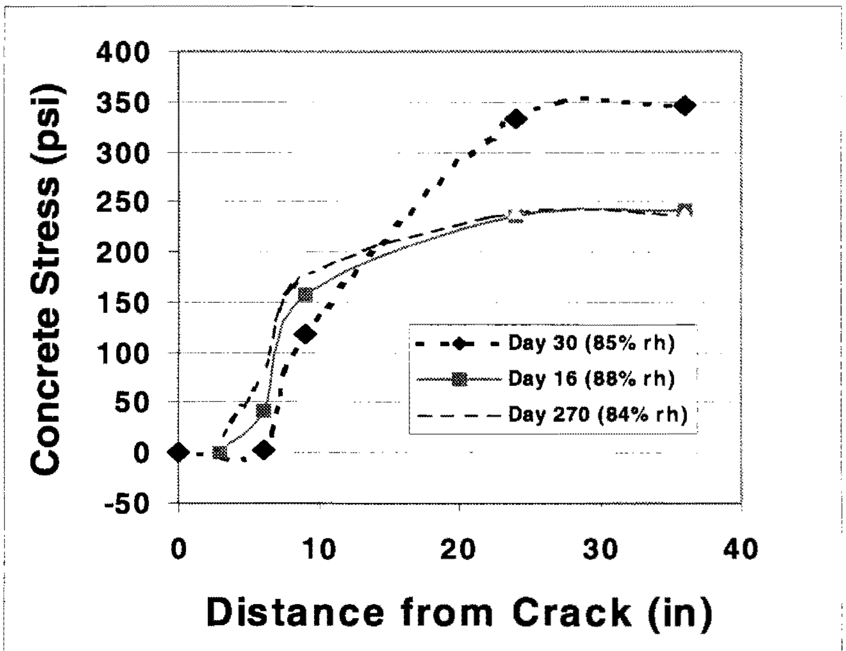

Figure 2.2 Concrete Stress Versus Distance from the Induced Crack. (R.H. values are at 1" below the surface.)

The measured characteristics of creep are elaborated further below, but a large percentage of creep occurred in the first three days of age while the concrete was stiffening. In order to calculate the restrained strain, it was necessary to estimate the ultimate shrinkage (ε_{ult}) that would occur in the concrete placed at the project site. This determination was based on shrinkage measurements made on-site over a 28-day period using the following model [7]:

$$\varepsilon_{shr}(t) = \varepsilon_{ult}\left[\frac{t}{n+t}\right]$$

where t is time of shrinkage and n is the half of the time to achieve the ultimate shrinkage. Fitting this mode to the measured data yielded an ultimate shrinkage of 700 microstrains and a value of n = 25.6 days. The development of the shrinkage strain was projected as shown in Figure 2.3. The relative humidity of the concrete was used to determine the amount of drying shrinkage at any point in time based on [8]:

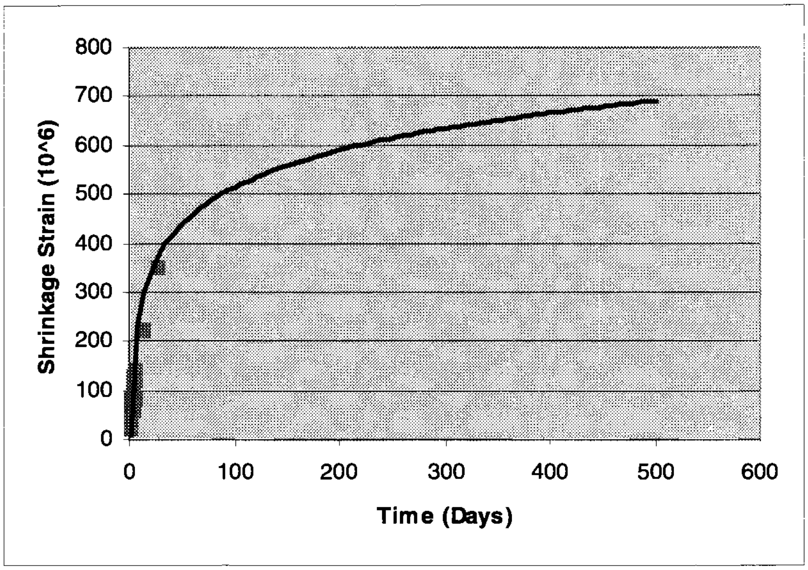

Figure 2.3 Projection of Concrete Shrinkage Based on Field Measured Concrete Shrinkage Strains.

$$\varepsilon_{shr}(t) = \varepsilon_{ult}(1 - rh^3)$$

As previously noted, the restrained strain was determined from the difference in the gage reading and drying shrinkage less the amount of creep. This quantity was converted into a stress value by simply multiplying by the modulus of elasticity of the concrete calculated for that particular concrete age. The concrete modulus, for each day of the analysis, was obtained from the compressive strength measurements based on compressive tests on the cylinder specimens cured at the test site. The variation in concrete stress with respect to distance from the induced crack, as measured at days 16, 30, and 270 of the project, is displayed in Figure 2.2. Given the measured relative humidity values of the concrete, the corresponding calculated shrinkages, and the noted creep stains subsequently discussed, stress levels in the concrete matched reasonably well with the concrete stress predictions discussed later in the chapter from the CRCP 8 program. It is interesting to note that the creep strain nearly canceled out any development of restrained strain in the vicinity of the crack face. Even though the degree of drying was greater at day 270, the total state of stress was lower in the concrete since the temperature difference was not as much as it was on days 16 and 30.

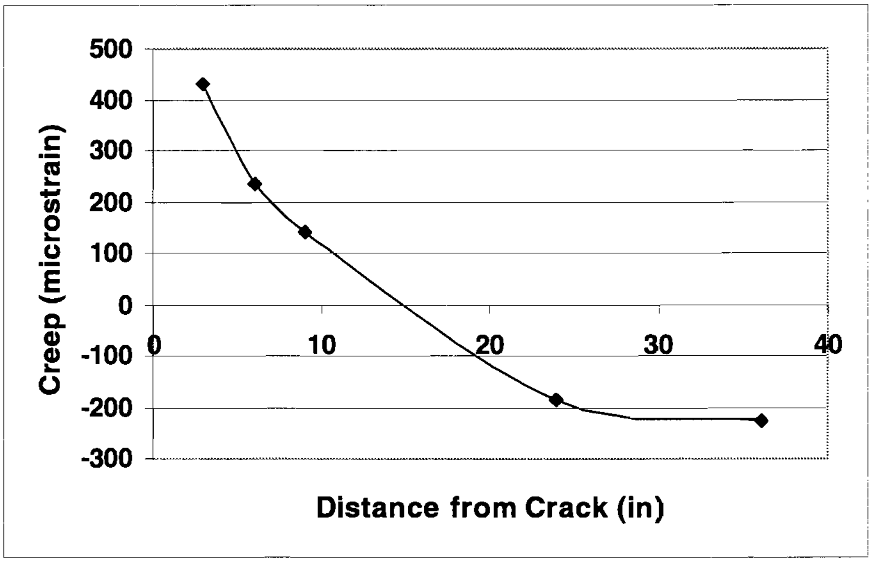

Figure 2.4 Concrete Creep Strain Variation with Distance from the Crack Face.

Concrete Creep

As noted previously, the majority of creep strain within the concrete occurred during the first five days of the pavement life. The creep was determined by comparing the shift in the reference point of the concrete strain reading from day-to-day at 6:00 am. The strains that were measured on each day are summarized in Figure 2.4 as a function of distance from the crack face. This behavior seems to demonstrate a sensitivity of the creep to the state of stress state in the concrete in the vicinity of and along the axis of the reinforcing steel, based on the direction of the shift in the reference point of the gage. Essentially all of the creep ended after day 5. It is also noteworthy to point out that cracking was observed to initiate at the sawcut notch on the morning of the fourth day after placement of the concrete at the time where the creep strain had nearly diminished to zero. Due to the effects of creep, the steel strain measurements used for the calculation of bond stress and bond slip were adjusted in order to take creep into account. The

Table 2.3 Daily Minimum Pavement Temperature Values.

Set	106 °F
Day 1	105 °F
Day 2	102 °F
Day 4	90.5 °F
Day 5	89.9 °F
Day 6	88.9 °F
Day 7	88.8 °F
Day 16	75 °F
Day 30	79 °F
Day 161	61 °F
Day 270	75 °F

creep is mathematically subtracted from the concrete strain, and the resulting value is subtracted from the change in steel strain to arrive at bond stress value.

Simulation Comparisons

Computer simulations of the conditions on days 16, 30, 162, and 270 were conducted with the CRCP 8 program and were compared to the measured field results on the basis of matching the average crack spacing predicted by the CRCP 8 program to the crack spacing of the instrumented section, which was 10 ft. On this basis, it was necessary to adjust the input slab temperature distribution over the first 28 days of pavement age from those indicated in Table 2.3 in order to obtain a 10-ft average crack spacing from the CRCP 8 program at each of the pavement ages noted above. The slab temperature distributions used in each case are noted in Table 2.4, which can be compared to those listed in Table 2.3. The simulation results for day 162 are shown in Figure 2.5. It appears, in this figure, some error in the field data exist, particularly in the data at 3 inches from the crack face. Relative to the prediction of the average crack spacing, it appears the CRCP 8 program manifests a lack of sensitivity to early-aged drying shrinkage since the drops in temperature at early pavement ages needed by the program to match the instrumentation site 10-ft controlled spacing exceeded those recorded at the instrumented site. Given the temperature distributions in Table 2.4, the CRCP 8 tended to overestimate the steel strain at the instrumented crack face but appears to represent them well at distances away from the crack face. Also noted in Figure 2.5 are TTICRCP results which were calibrated to the steel strain at the crack

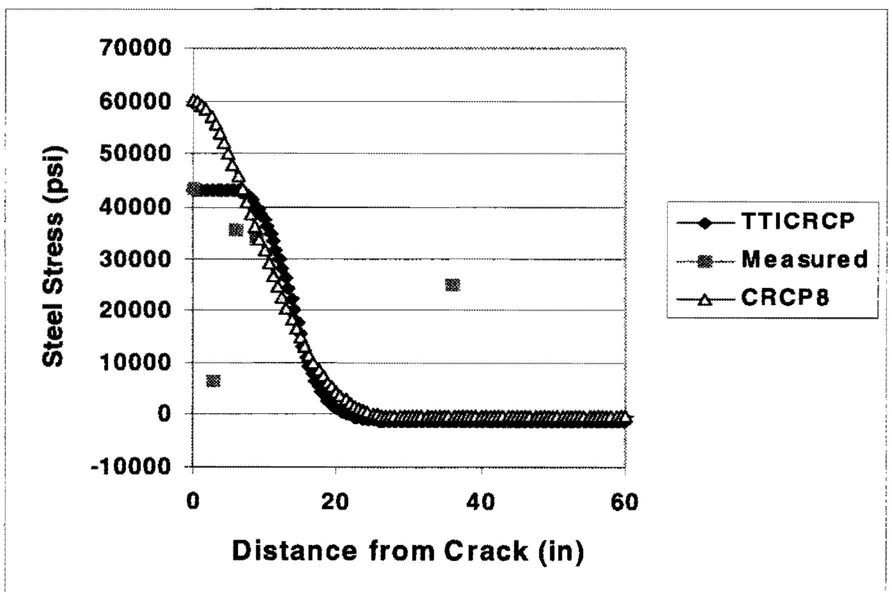

Figure 2.5 Comparison of Steel Stress Distribution between Measured and Predicted Stresses at Day 162.

face by adjustment of the parameter K_1.

Concrete strains were also compared in a similar manner. The CRCP 8 results appeared to compare reasonably well with the field results. However, it should be noted, the field strains were determined based on the gage reading, the amount of shrinkage adjusted according to the measured relative humidities 1 inch below the pavement surface, and the amount of creep determined on day 5. Although neither the CRCP 8 nor the TTICRCP programs take into account creep effect directly, it appears feasible that the amount of creep may be indirectly assessed by matching the predicted stress distributions with the field distributions, indicating that the effect of creep can be accounted for through adjusted values of the concrete modulus of elasticity. In this manner, the early-aged development of creep and shrinkage and their effects upon the predicted stress pattern needs further consideration in future updates of the CRCP 8 program.

Table 2.4 Adjusted CRCP 8 Daily Minimum Pavement Temperature Values to Achieve a 10-Foot Crack Spacing.

Concrete Age (Days)	**Day 16** (°F)	**Day 30** (°F)	**Day 162** (°F)	**Day 270** (°F)
Set	106	106	106	111
1	88	82	79	80
2	81	75	72	72
3	71	66	66	66
4	64	60	60	60
5	58	60	60	60
6	58	60	60	60
7	58	60	60	60
16*	58	60	60	60
30*(28)		52	60	60
161*			40	60
270*				50

Analysis of General Design Conditions

Analysis was also conducted using the material characteristics of the instrumented test site to develop a sense of the range of steel stresses and crack widths that may be encountered in design. These ranges were developed by considering typical ranges in bar diameter, percent steel reinforcement, maximum temperature drop, and drying shrinkage. Eight combinations were derived from the variety of ranges for each parameter. For each combination, the results of the programs were compared and used to develop correction factors for the CRCP 8 results, based on the calibrated TTICRCP results. The actual correction factors are discussed further in the research results, but it appears that in comparison to the TTICRCP results, the CRCP 8 program overestimates the steel stresses and underestimates the crack widths.

However, the capability of CRCP 8 to predict crack spacing distribution has been well documented in previous research reports on the program development and application. Consequently, this particular aspect is given no further consideration in this report. Emphasis, however, is given to comparative analysis of the predicted steel stresses and crack widths and implications associated with these comparisons.

Crack Width - Steel Stress Considerations

Detailed analysis was presented indicating the accuracy of the CRCP 8 program to predict stress in the reinforcing steel and the opening of the transverse cracks. Based on the comparisons to the recorded field strains and the tabulated results derived from the calibrated TTICRCP bond-slip model, correction factors should be applied to the results of the CRCP 8 model to adjust the overprediction of the steel stress and the underprediction of the crack width. These correction factors are conveniently illustrated in Figure 2.6. The correction is not constant across the range of the parameter depicted along the x axis, which is a dimensionless combination of concrete and steel CTE, drying shrinkage (z in micro strains ($\mu\epsilon$)), design temperature drop (Delt), q factor, and crack spacing (L). The correction factor for both the steel stress and the crack width is determined for the same value of the x axis and divided into the result obtained from the CRCP 8 program.

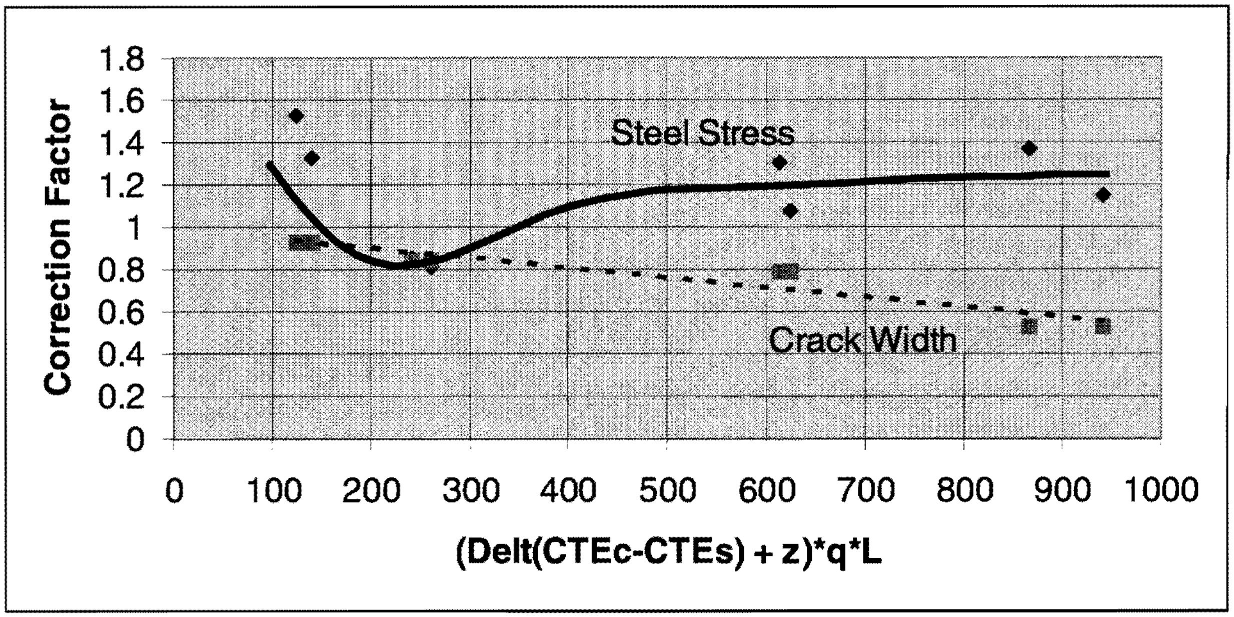

Figure 2.6 CRCP 8 Steel Stress and Crack Width Correction Factors.

Project Findings

As a part of meeting the objectives of this project, the following findings are provided:

1. The methods used to instrument the reinforcing steel in the I-45 CRC pavement test site proved to be a beneficial and resourceful technique to minimize disturbance of the bond-slip of the reinforcing bar and to obtain steel strains at various distances along the bar from the crack face. The effect of creep on calculated concrete stresses was significant, and demonstration of the sensitivity of creep to the state of stress at various distances from the face of the crack signals a need to pay greater attention to this phenomena in future updates of the CRCP 8 program. (This consideration could be made in the current study, Project 0-1831, "Improvement of Concrete Performance Model.") Creep strains, under applied loads, have traditionally been treated on a long-term basis. Although shrinkage-induced creep initially is very large, it diminishes within a few days, to where stress levels are allowed to increase enough to initiate cracks in the concrete. However, prior to this occurring, it appears that early-aged creep significantly relaxes stress development.

2. The performance surveys of the SH 249 Grade 70 steel sections, placed at a q factor of 0.026 and under cool weather conditions, indicated undesirably wide (in comparison to the Grade

60) average crack spacings. However, the Grade 70 sections, which consisted of a single layer of steel, demonstrated the desirable feature of lower clustering within the resulting crack pattern. Also a similar design, placed at the I-45 instrumented section under hot weather conditions, yielded a desirable crack pattern. As many previous studies have noted, the weather conditions at the time of construction are a major factor in the early performance behavior of CRC pavement systems and can eventually impact its later performance—for better or for worse. Climatic factors should be a major consideration in the future Project 0-1700, "Improving Portland Cement Concrete Pavement Performance" research study.

3. The numerical algorithm used in the present version of the CRCP 8 program is suitable as a design tool for the prediction of crack spacing, crack width, and steel stress and should be used as a base on which to make future improvements. The numerical algorithm of the TTICRCP program is most suitable as a calibration tool to represent the bond-slip of reinforcement in CRC pavement.

4. The evaluation of the CRCP 8 program indicated that it can be used as a design tool for the prediction of the CRC pavement structural responses, but corrections should be applied to the predicted average steel stresses and crack widths. Improvements to the program are encouraged and warranted on the basis of its sensitivity to the concrete temperature assigned to the first day after construction, which tended to dominate the effect of temperature inputs for other days of the analysis. In this same vein, the program also seems to offset a lack of sensitivity to drying shrinkage that appears to be presently compensated for by a larger than expected first day temperature drop. As a consequence, the 10-ft controlled crack spacing at the instrumentation site was not well predicted when actual concrete pavement temperatures over the first 28 days of age were input into CRCP 8. When the 28-day temperature profile was appropriately adjusted such that the average predicted crack spacing matched the instrumented 10-ft crack spacing, the program tended to overpredict the steel stress and underpredict the crack width. A correction chart was developed to provide factors to adjust the CRCP 8 results for use in design.

However, as previously noted in the introduction of this chapter, these statements are not made in any reference to the program's capability to predict trends in the crack pattern. But it does appear that current versions of CRCP 8 are perhaps better suited to represent later cracking behavior rather than early cracking behavior of CRC pavement systems. In this respect, the field data clearly indicated a high degree of relaxation in the first three to four days after construction of the pavement, which effectively eliminated the buildup of early-age shrinkage stress in the concrete based upon the time that initiation of the observed cracking took place. It is apparent that concrete setting temperature models for CRCP 8 may take the effect of the early creep into account to some extent by the selection of a reference temperature 7 percent below the concrete peak temperature. This aspect appears to be an area that further research (i.e., 0-1700 research study) could yield improved models to advance the capability of CRCP 8 to represent early-aged cracking behavior.

5. The design of CRC pavement systems must include consideration for crack width and its affect upon the load transfer and stiffness of the transverse cracks over the design life of the pavement system. This parameter should be given a greater precedence in the design process—even more than the design level of steel stress. Nonetheless, the average steel stress is an important design consideration relative to the selection of the proper grade of steel.

6. Vertical positioning of the steel layer appears to affect the development of cluster cracking. Relative to statement 2) above, data collected at the SH 249 test site indicated a distinct difference in clustering between pavements constructed with one layer versus pavements constructed with two layers of reinforcement. It is clear that the vertical position of the steel layer also influences the degree of restraint in the concrete near the pavement surface and characteristics of the cracking pattern, particularly relative to the development of clustering. Given the fact that restraint by the reinforcement is constant at any vertical position of the steel in the slab, a plausible explanation for cluster cracking is nonuniformity in the depth of curing from point to point along the pavement. Apparently, if the depth of drying varies from point to point, then the induced cracking stress will vary accordingly relative to the vertical position of the reinforcing steel. The deeper the steel layer, the less effect the variation in the depth of drying will have on cracking stress. More uniform curing should help to minimize cluster cracking and allow shallower placements of the steel layer and narrower crack widths at the pavement surface. This observation is further supported based on information in the literature suggesting the

vertical position of the reinforcing steel influences the variation in crack width with distance below the pavement surface. Finite element models can represent this type of behavior as it may be affected by the position of the reinforcement in the presence of temperature and moisture gradients. The advancement of the design and analysis of CRC pavement systems will depend upon the reflection of the finite element results in design models to better account for differential slab behavior.

Recommendations

The CRCP 8 program is a well-founded, computerized approach to the prediction of crack spacing, steel stress, and crack width and is consequently well suited for future improvements to the process it uses to represent the behavior of CRC pavement systems. The current study 0-1831 and the future study 0-1700 should be used to make improvements to material models used in the program to represent both temperature and moisture changes in profile as they vary with time during the early ages of the concrete and the translation of the profile changes into strain and stress. The roles of drying shrinkage and creep also need further definition in the crack development process. Tools that have the capability to take into account the heat of hydration and the quality of curing during the hardening process have recently been developed to accomplish such a task. Efforts to develop such products and additions to the CRCP 8 program should be immediately undertaken to improve how the CRCP 8 program characterizes the effect of moisture and temperature change over the first 28 days of analysis. The consideration of crack width as a function of distance from the surface of the slab will allow for more accurate assessment of the crack opening at the level of the steel based on surface measurements. Changes are also needed and suggested to the bond-slip algorithm to improve its capability to be calibrated and to represent the partial bond types region similar to the process used in the TTICRCP program but modified with other bond stress distribution that may accelerate the calculation time while improving the representation of bond stress between the steel and the concrete. The improvements recommended should be complemented with suitable laboratory tests and studies to verify the accuracy of the program models under controlled conditions and followed up with additional field sections to validate their application to design.

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