

DEVELOPMENT OF LOAD TRANSFER COEFFICIENTS FOR USE WITH THE AASHTO GUIDE BASED ON FIELD MEASUREMENTS

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INTRODUCTION

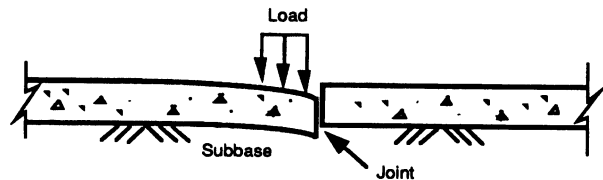
The current pavement design procedure recommended by the American Association of State Highway and Transportation Officials (AASHTO) is based largely upon the findings of the AASHO (American Association of State Highway Officials) Road Test conducted in the late 1950s at Ottawa, Illinois (1). The AASHO (currently the AASHTO) Design Committee published the first Interim Pavement Design Guide in 1961 based on the results of the Road Test (2). The Interim Pavement Design Guide was later revised in 1972 and 1981 (3 and 4). In 1986, the AASHTO Design Committee published a new guide, "AASHTO Guide for Design of Pavement Structures" for convenience, it is referred to in this paper as the AASHTO Guide (1).

During the development of the current 1986 AASHTO guide, the basic form of the AASHTO Road Test rigid pavement performance equation was expanded to include additional parameters, since the original Road Test was limited in scope (i.e., a local materials, one subgrade, one environment, etc.). One of the features in the rigid pavement performance equation in the AASHTO guide is the expansion of the load transfer coefficient (J-factor) to consider various rigid pavement slab configurations, including jointed concrete pavements and continuously reinforced concrete pavements (1).

The use of the load transfer coefficients in rigid pavement performance equations accounts for the differences in the ability of various types of concrete pavement structures to transfer loads across joints or cracks. The load transfer coefficient may be affected by many factors, including aggregate interlock, load transfer devices used in the pavement, and the presence of tied concrete shoulders.

Figure 1 presents the basic concept of the load transfer coefficient. If load transfer devices are not used at the joint, the deflection at the joint is likely to be higher than that associated with the use of load transfer devices. And, consequently, if the same performance life is required for these two types of pavement, a thicker slab is needed for the pavement not equipped with load transfer devices at its joints. This may be accomplished in the design by using different values of load transfer coefficients in the rigid pavement performance equation. A larger load transfer coefficient would be used for pavements without load transfer devices, while a lower load transfer coefficient would be used for pavements with load transfer devices.

Case 1: No load transfer devices are provided



Case 2: Load transfer devices are provided

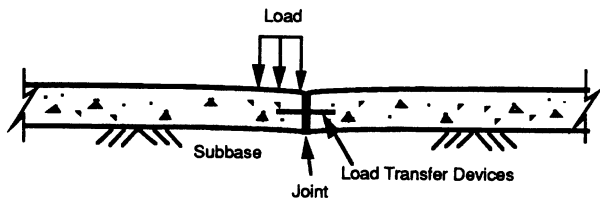


FIGURE 1 Illustration of the basic concept of load transfer coefficient.

To develop suitable load transfer coefficients for use with the AASHTO guide in designing rigid pavements in Texas, the Center for Transportation Research (CTR) at The University of Texas at Austin study team sought to develop load transfer coefficients from measurements of in-service pavements. Since the AASHO Road Test did not include the study of Continuously Reinforced Concrete Pavements (CRCP), that pavement received priority. This paper documents the development of appropriate load transfer coefficients for the design of CRCP in Texas.

AASHTO RIGID PAVEMENT EQUATION

The AASHTO Design equation, was developed by combining a mechanistic equation (i.e., Spangler) with the empirical AASHO Road Test Equation and then substituting the values for the physical constants that represent the Road Test Conditions:

$$\log W' = 7.35 \log (D + 1) - 0.06 + \frac{G}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} + (4.22 - 0.32p) \log \left(\frac{S'_c}{215.63 J} \right) \left(\frac{D^{0.75} - 1.132}{D^{0.75} - 18.42} \right) \left(\frac{Z^{0.25}}{Z^{0.25}} \right) \quad (\text{Eq 1})$$

where

P = serviceability at a given time,

S'_c = concrete modulus of rupture differing from the Road Test,

Z = E/K and,

D = Pavement thickness,

J = 3.2 for the AASHTO Road Test conditions.

It should be noted that the relationship defined in Equation 1 contained the following two important assumptions, which were made at the Road Test (5):

1. The variation in pavement life (W) for different loads at the same level of Sc/s is accounted for by the basic Road Test equation and is covered in the AASHTO design procedure by traffic equivalence factors.
2. Any change in the Sc/s resulting from changes in the physical constant E, K, D, and Sc will have the same effect on W' as varying slab thickness. This relationship is defined by Equation 1.

During the 1980s, the trend toward using a load transfer coefficient value of 3.2 increased as a result of more heavy-truck traffic on interstate highways and because of the increased use of pavement shoulders, or "off-tracking," which creates critical conditions at the pavement edge. Previous use of a load transfer coefficient of 2.2 was based on the interior loading condition, as pointed out by McCullough (6). The shift toward using 3.2 for a load transfer coefficient can be found in the 1981 version of the AASHTO Interim Guide for Design of Pavement Structures (3), as well as in the Associated Reinforced Bar Producers (ARBP) manual for the design of CRC pavements (7). The ARBP manual also gives a value of 2.56 as a load transfer coefficient for the design of CRC pavements with a tied concrete shoulder.

The mechanistic derivation of the load transfer coefficient for design of CRC pavements can be found in Volume 2 of the 1986 AASHTO Guide for the Design of Pavement Structures (8). Table 1 shows the values of load transfer coefficients derived from the mechanistic analysis. The load transfer coefficient values recommended in the AASHTO Guide are presented in Table 2.

TABLE 1 Load transfer coefficients for CRC pavements obtained from mechanistic analysis (8).

Pavement Group	Load Transfer Type	Load Transfer Coefficient
1	Load transfer devices at the cracks with a tied PCC shoulder	2.3 - 2.6
2	load transfer devices at the cracks with an AC shoulder	2.6 - 3.1

TABLE 2 Recommended load transfer coefficients for CRC pavements (1).

Pavement Group	Load Transfer Type	Load Transfer Coefficient
1	Load transfer devices at the cracks with a tied PCC shoulder	2.3 - 2.9
2	load transfer devices at the cracks with an AC shoulder	2.9 - 3.2

A comparison of load transfer coefficients presented in Tables 1 and 2 reveals some adjustments made in the AASHTO guide. Also, it can be seen that the ranges of recommended values of load transfer coefficients are quite wide. For instance, in designing CRC pavements without a tied PCC shoulder, one can choose a value ranging from 2.3 to 2.9, which may result in a large difference in thickness. Therefore, it is necessary to develop a suitable value of load transfer coefficient for local conditions if the AASHTO Guide procedure is to be used.

From the previous review of the development of load transfer coefficients, it can be concluded that the concept of load transfer coefficient is much more than a problem of percent of load transfer that will transfer across the joint/crack. It is also a problem of the ability of various pavement structures to support the traffic load under different conditions. For instance, the use of a tied PCC shoulder may reduce the load transfer coefficient value. Light traffic and reduced temperature variations may also reduce this value. Load transfer devices, aggregate interlock, subgrade reaction, and the presence of tied concrete shoulders all will have an effect on this value. Therefore, a procedure is needed that can take into account pavement thickness, pavement geometric conditions and/or boundary conditions, subgrade, and the discontinuities in the field pavements.

In summary, while a load transfer coefficient value of 2.2 was used in the 1960s, the concept of load transfer coefficient was later expanded in the 1986 AASHTO guide to consider various types of pavement structures in different conditions. A mechanistic approach was provided in Volume 2 of the 1986 AASHTO guide. However, a procedure to obtain a suitable value of load transfer coefficients for local conditions, especially a procedure based on field measurements, is not available in the AASHTO guide.

OBJECTIVES

This paper documents a study under taken to develop appropriate load transfer coefficients for use with the AASHTO guide for the design of continuously reinforced concrete pavements in Texas. Such coefficients were to be based on measurements of in-service pavements and incorporated into future pavement design guidelines. Specifically, the objectives of the study included the following:

- (1) to assess load transfer ability of in-service CRCP using available information in the rigid pavement data base (9),
- (2) to develop a procedure to estimate load transfer coefficients for rigid pavements in Texas based on field measurements,
- (3) to apply the procedure to obtain load transfer coefficients from measurements of in-service CRCP in Texas, and
- (4) to recommend load transfer coefficients for the design of CRCP in Texas.

SCOPE OF THE STUDY AND PAPER

The overall approach to the research task, along with a general outline of the study activities, is shown in Figure 2. The approach is primarily based on information obtained from Research Project 460, "Assessment of Load Transfer Across Joints or Cracks in Rigid Pavements Using the Falling Weight Deflectometer," sponsored by the Texas Department of Transportation and the Federal Highway Administration (10).

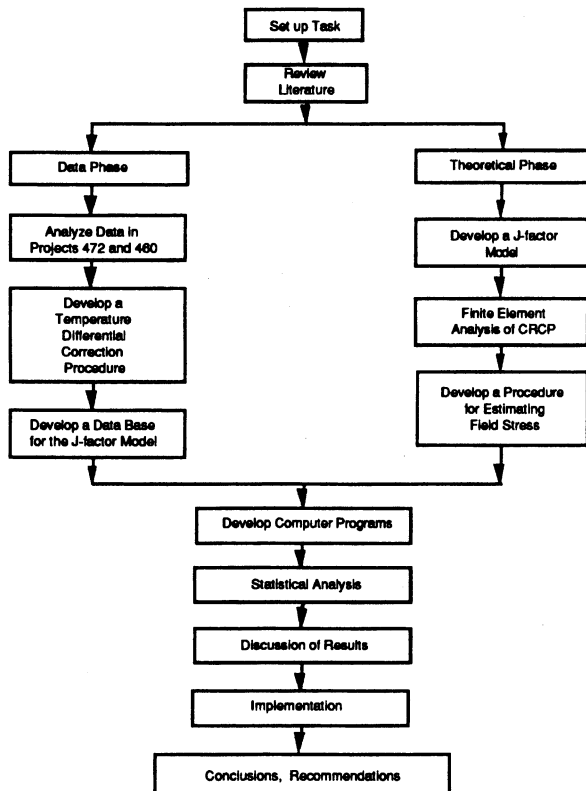


FIGURE 2 Outline of the study of load transfer coefficient. (After FIGURE 1.2 of 5.)

ASSESSMENT OF STRUCTURAL CONDITIONS OF RIGID PAVEMENTS IN TEXAS

To develop a suitable load transfer coefficient for the design of CRCP in Texas, an assessment of structural conditions of in-service CRCP in Texas was performed.

In Research Project 472, "Rigid Pavement Database," conducted in 1988 by the Center for Transportation Research (CTR) at The University of Texas at Austin, falling weight deflectometer (FWD) deflection measurements were taken on in-service CRC pavements across Texas and stored in a rigid pavement database (9). These FWD measurements were taken from both non-overlaid CRC pavements and overlaid pavements; however, only measurements from non-overlay pavements are analyzed in this paper.

This section presents the assessment of in-service CRC pavement structural conditions using the statewide deflection measurement data and other available information in the rigid pavement database. The primary purpose of this assessment is to evaluate the abilities of in-service CRC pavements to transfer load across pavement discontinuities. This assessment includes the following conditions:

1. load transfer at transverse cracks in CRC pavements;
2. difference in deflections at cracks and at mid-span between cracks; and
3. difference in deflections between the pavement edge and the interior.

In addition, variations in deflections for different loading positions are evaluated, with comparisons made between deflections from in-service CRC pavements and deflections from a test section of jointed pavement. Analytical results from the theoretical model are also compared with deflections from in-service CRC pavements and jointed pavements.

OVERVIEW OF FIELD STUDY AND EXPERIMENT

The FWD measurements taken on non-overlaid CRC pavements during the summer of 1988 covered the eleven districts shown in Figure 3. Selected pavement sections for the measurements included:

1. Three slab thicknesses of CRC pavements:
 - 8-inch
 - 9-inch
 - 13-inch
2. Two pavement shoulder types:
 - asphalt concrete shoulder (AC shoulder)
 - tied Portland cement concrete shoulder (PCC shoulder)
3. Three coarse aggregate types:
 - limestone (LS)
 - siliceous river gravel (SRG)
 - combination of limestone and siliceous river gravel (LS/SRG or SRG/LS)
4. Four different subbase types:
 - asphalt stabilized (ACT)
 - cement stabilized (CT)
 - lime treated (LT)
 - crushed stone (CRS)

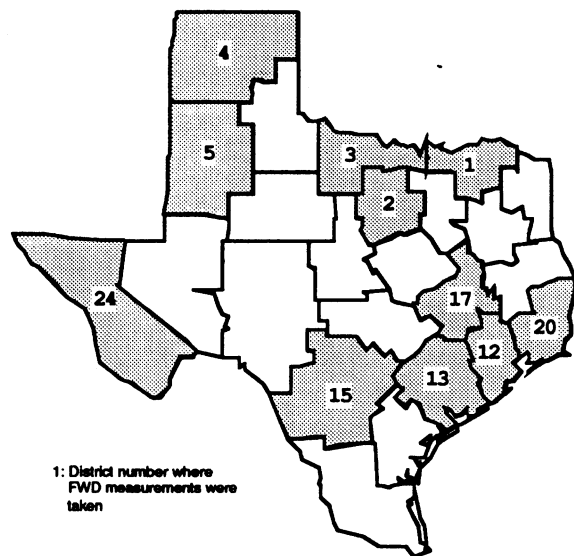


FIGURE 3 The geographical distribution of CRCP FWD measurements in Texas, displayed by districts.

Each test section was 1,000 feet long and was divided into five subsections (9). A set of FWD measurements was taken in each subsection, as shown in Figure 4. This set of FWD measurements included two stations near the crack, one station at the crack, one station at the mid-span between the cracks, and one station in the interior. These stations are numbered 1 to 5, as indicated in Figure 4. Four sets of FWD measurements were taken at each station using four different drop heights with peak loads ranging from about 6,000 pounds to 17,000 pounds.

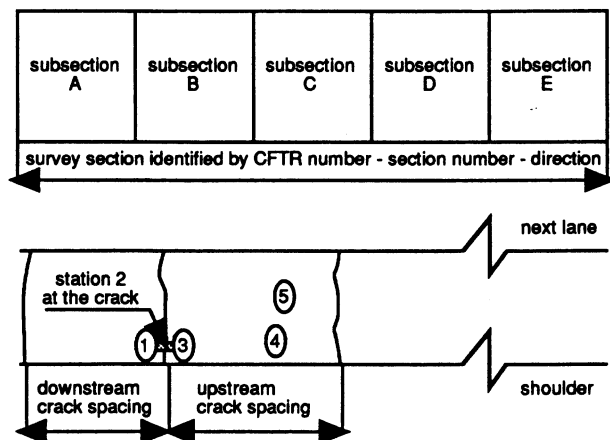


FIGURE 4 Layout for FWD measurements in a non-overlaid section (13).

PILOT STUDIES

All deflections used in the study were for Drop Height 4, which closely corresponds to an equivalent 19 kip single axle.

Three load transfer efficiency procedures were selected from the literature for evaluation as follows:

1. Teller and Sutherland (14)
2. Darter (1)
3. Ricci and Owusu-Anti (10 and 15)

After a detailed evaluation, the Teller Procedure was adopted for this study.

Many studies have shown that the temperature differential has a significant effect on FWD deflection measurements, especially measurements near the edge and joints (16, 17, and 18). The temperature differential (DT) discussed here is defined as the difference between the temperature at the top and at the bottom of a pavement (17).

Figure 5 presents the plots of load transfer efficiencies (calculated from three procedures) vs. temperature differential at the joint, while Figure 6 shows the plots at the crack. As Figure 5 shows, load transfer does vary at the joint with temperature differential, particularly from about -5 to +5 degrees Fahrenheit. As the temperature differential becomes greater than 5 degrees Fahrenheit, the DT effect is not significant. This could be a result of the expansion of two pavement slabs and/or the pavement warping downward. However, as shown in Figure 6, the effect of temperature differential on load transfer evaluation at the cracks is not significant. This could be because the cracks are held so tightly that the effect of temperature differential on load transfer evaluation is not significant. A recent study conducted by CTR confirms this to be the case (10 and 19). Therefore, the effect of temperature differential on the evaluation of load transfer at the cracks in the in-service CRC pavements was not considered.

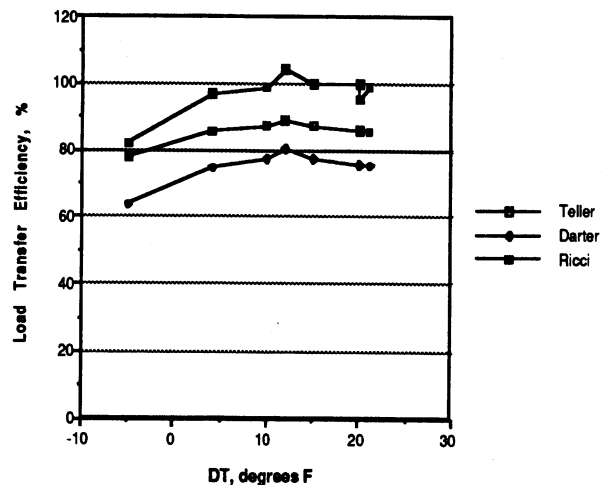


FIGURE 5(a) Effect of temperature differential (DT) on load transfer evaluation at station Ju, Slab 1.

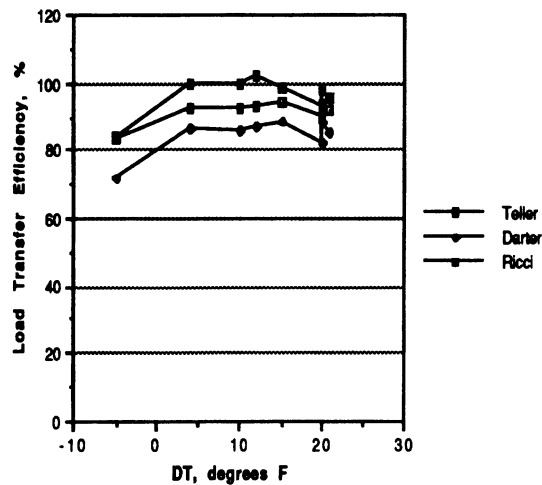


FIGURE 5(b) Effect of temperature differential (DT) on load transfer evaluation at station Jd, Slab 1.

LOAD TRANSFER AT TRANSVERSE CRACKS IN CRCP

Continuously reinforced pavements are constructed without transverse joints, therefore, volume change stresses caused by shrinkage and temperature differentials are dissipated by the formation of transverse cracks tightly closed, so that effective load transfer can be maintained at the cracks.

Load transfer at cracks in CRC pavement, as defined herein, is the ability of that pavement to transfer loads across transverse cracks from one slab of pavement to another. If a CRC pavement can maintain good load transfer (about 100 percent) at the cracks, it usually shows good pavement performance. However, as the load repetitions increase and the pavement ages, the load transfer at the cracks will decrease until a loss of load transfer is experienced.

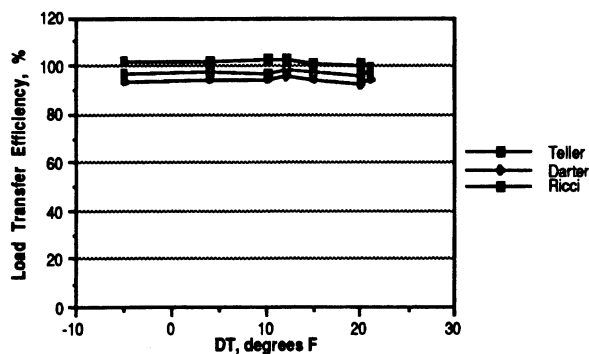


FIGURE 6(a) Effect of temperature differential (DT) on load transfer evaluation at station Cu, Slab 1.

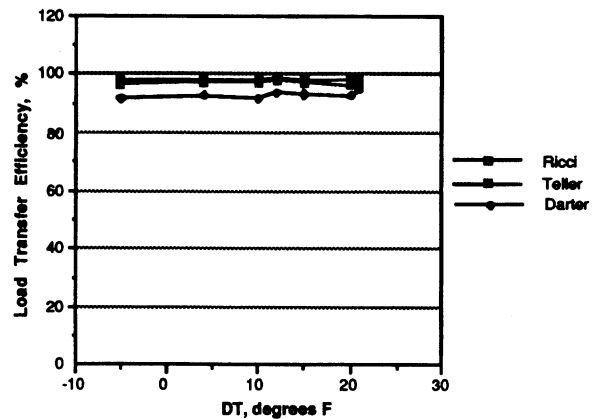


FIGURE 6(b) Effect of temperature differential (DT) on load transfer evaluation of station Cd, Slab 1.

The statewide FWD measurements were taken using two types of geophone configurations. When the Type A geophone configuration was used for deflection measurements, the load transfer efficiencies at stations 1 and 3 (refer to Figure 4) were averaged to represent the load transfer efficiency for the crack between stations 1 and 3.

The results of the evaluation of load transfer at the cracks in CRC pavements are presented in terms of groups with three different thicknesses and two shoulder types. Figure 7 shows the mean load transfer efficiency. Figures 8 and 9 present the plots of the standard deviation and the coefficient of variation of the load transfer efficiency, respectively. Table 3 shows the number of data points used for the calculation.

As observed in Figure 7, the mean values of load transfer efficiency are around 95 percent for all the groups except for the 13-inch slab pavement group with an AC shoulder. In addition, the standard deviations are very small, with the coefficients of variation of LTE less than 5 percent. These results indicate that the in-service CRC pavements in Texas have good load transfer across the cracks.

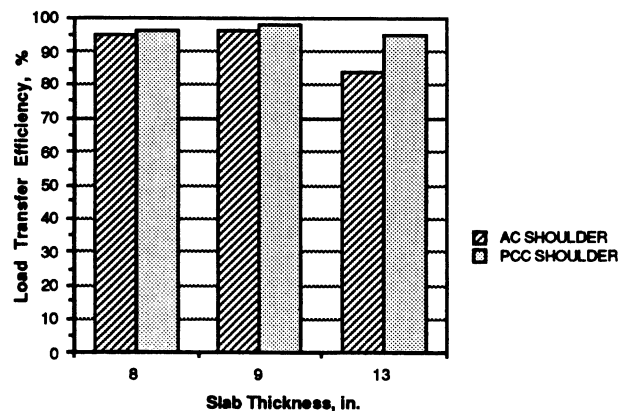


FIGURE 7 Load transfer efficiency at the cracks in CRC pavements.

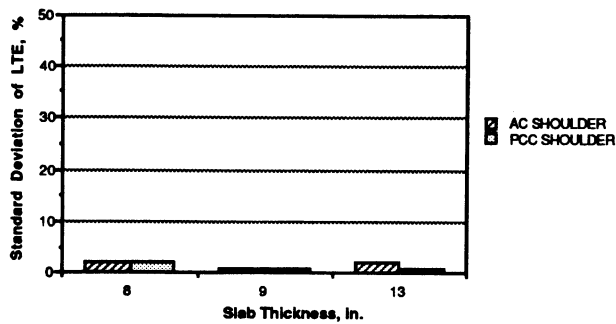


FIGURE 8 Standard deviation of LTE at the cracks in CRC pavements.

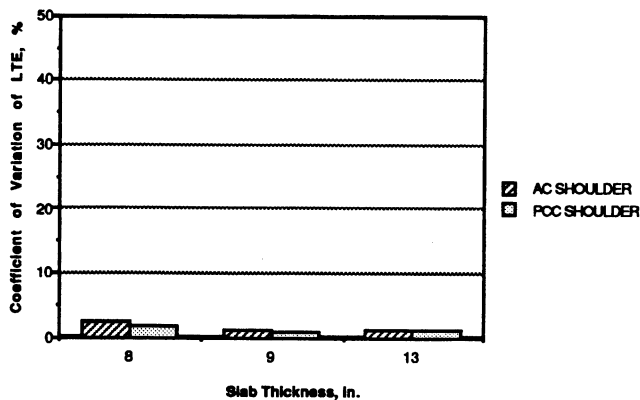


FIGURE 9 Variability of load transfer efficiency at the cracks in CRC pavements.

TABLE 3 Number of data points for calculation of the LTE.

Shoulder Type	AC			Tied PCC		
	8	9	13	8	9	13
Slab Thickness, in.	8	9	13	8	9	13
No. of Observations	408	70	5	56	15	27

As can also be seen from Figures 7 to 9, pavements with a tied PCC shoulder have a higher value of load transfer efficiency than those with an AC shoulder. The coefficients of variation of LTE for pavements with a tied PCC shoulder are also lower than those for pavements with an AC shoulder. This indicates, as expected, that a tied PCC shoulder assists the pavement in maintaining load transfer.

The cause of the unexpected results of load transfer efficiency for the 13-inch pavements group is not clear, but it is probably a result of the small number of available sections having this thickness. It is expected that the load transfer value would be greater than that associated with the 8-inch pavement group because greater thicknesses are believed to have better load transfer ability. Since only five data points were collected, no further conclusions can be made regarding this pavement group.

COMPARISON OF DEFLECTIONS AT THE CORNER OF THE CRACK AND AT MID-SPAN BETWEEN THE CRACKS

This section evaluates the difference in deflections between those at the corner (formed by the pavement edge and the crack) and those collected at the pavement edge (but at mid-span between the cracks). Using Figure 4, this difference can be defined as follows:

$$R_{ce} = \frac{\text{Deflection at Station 3}}{\text{Deflection at Station 4}}$$

Basically, if there is some loss of load transfer at the crack, the deflection at the crack will be greater than at the point of mid-span between the cracks. If the deflections are the same, the crack has very good load transfer. It can then be assumed that there is no loss of load transfer at the crack. Therefore, the pavement could be treated as a continuous slab in the longitudinal direction.

Figure 10 presents the mean values of R_{ce} for various groups of pavements in terms of slab thickness and pavement shoulder types. As the figure shows, the mean values of R_{ce} for all the pavement groups are almost equal to 1. Therefore, these results support the previous conclusion that approximately 100 percent load transfer is maintained. The standard deviation of R_{ce} is depicted in Figure 11.

COMPARISON OF DEFLECTIONS AT THE PAVEMENT EDGE AND INTERIOR

The difference in deflection between the pavement edge and the interior is defined as follows:

$$R_{ei} = \frac{\text{Deflection at Station 4}}{\text{Deflection at Station 5}}$$

For this evaluation, the effect of temperature differential on the FWD measurements was considered, in that deflections at station 4 (near the edge) were corrected to a standard condition of zero temperature differential.

The mean values of the deflection ratio R_{ei} are shown in Figure 12, with Figure 13 showing the standard deviations of R_{ei} . As shown in Figure 12, the mean values of R_{ei} for pavements with an AC shoulder range from 1.2 to 1.7 — higher than those for pavements with a tied PCC shoulder. This indicates the effect of a tied PCC shoulder on reducing edge deflections. A comparison of the mean values of R_{ei} for different slab thicknesses indicates that R_{ei} decreases as slab thickness increases.

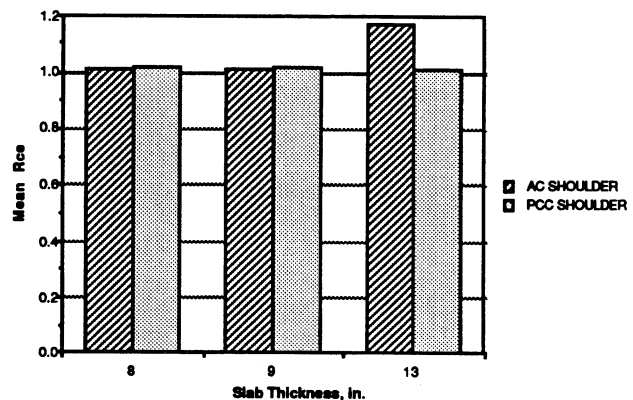


FIGURE 10 Mean value of R_{ce} for CRC pavements.

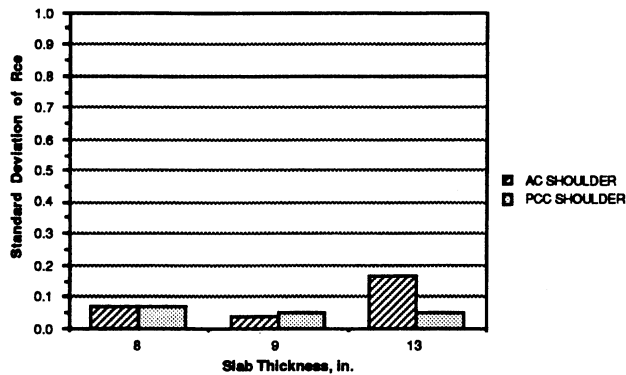


FIGURE 11 Standard deviation for R_{ce} for CRC pavements.

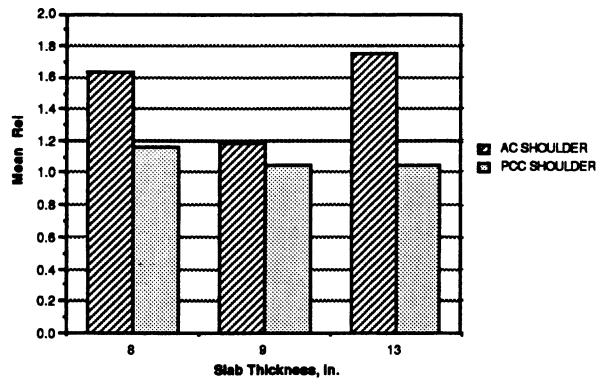


FIGURE 12 Mean value of R_{el} for CRC pavements.

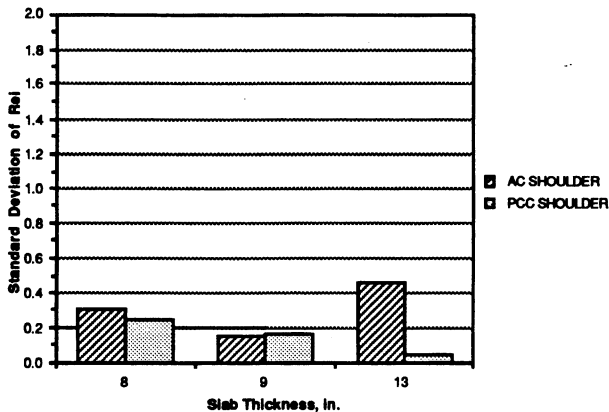


FIGURE 13 Standard deviation of R_{el} for CRC pavements.

SUMMARY OF STATISTICS

The mean, the standard deviation, and the coefficient of variation of the deflections measured at stations 1, 2, 3, 4, and 5 were also determined and are presented in Table 3. The means of the deflections at stations 1, 2, 3, and 4 are almost the same. The maximum difference in the mean deflections between these four stations is less than 5 percent of the minimum mean deflection. This indicates that the load transfer at the cracks is very good. Obviously, the means of deflections in the pavement interior are less than those near the edge and at the corner near the cracks. However, the coefficients of variation of the interior deflections are larger. This could be due to the fact that the pavement sections studied are from different subbase and soil conditions. As expected, the means of the deflections decrease as the slab thickness increases, as do the coefficient of variation values.

DEVELOPMENT OF A J-FACTOR MODEL BASED ON DEFLECTION MEASUREMENTS

As pointed out previously, the AASHTO concept of the load transfer coefficient was expanded in the rigid pavement performance equation to consider capabilities of various types of pavement structures. The use of the AASHTO guide procedure for the design of rigid pavements, such as CRC pavements, requires determining a suitable value for a load transfer coefficient for corresponding design conditions. However, a method for determining the load transfer coefficient based on field measurements is not available in the AASHTO guide.

This section describes the development of such a rational procedure, namely a J-factor model, for estimating a load transfer coefficient from field measurements on CRC pavements. In addition, a conceptual J-factor model for jointed pavements is presented.

DEVELOPING A GENERAL MATHEMATICAL FORMAT

The rigid pavement performance equation in the AASHTO guide was derived from empirical information obtained from the AASHTO Road Test (1). During the development of the equation, an equal ratio of strength to stress was assumed to give equal performance. For example, a pavement with less thickness (increased stress) but more flexural strength would show equal performance with another pavement, so long as the ratios between strength and stress were equal. Following this premise, it may also be hypothesized that a change in edge condition or load transfer at the joints/cracks results in a different degrees of stress, and, further, results in different performance if other factors are fixed. Such assumptions yield a straightforward relationship based on the above discussions. In other words, any difference in stress caused by differences in pavement physical properties and load transfer ability between the Road Test conditions and other conditions, such as Texas conditions, can be accounted for by changing a J-term value in the rigid pavement performance equation as follows.

Considering Equation 2 and assuming the concrete strength is fixed, let us further assume S_{LP} to be the maximum stress in jointed concrete pavements at the Road Test, ss_{P-A} to be the stress calculated from the Spangler equation for the Road Test conditions, S to be the maximum stress for an in-service pavement differing in physical properties from the Road Test conditions, and ss_{P-TXJ} to be the stress calculated from the Spangler equation

$$\log W = a + b \log F \quad (\text{Eq 2})$$

$$\frac{S}{\sigma_{\text{SP-TXJ}}} = \frac{S_{\text{LP}}}{\sigma_{\text{SP-A}}} \quad (\text{Eq 3})$$

where:

S = maximum stress for an in-service pavement,

$\sigma_{\text{SP-TXJ}}$ = stress calculated from the Spangler equation for an
in-service pavement with an unknown J-term,

$$S_{\text{LP}} = \frac{0.32P}{D^{4/3}}$$

which is the Loop 1 stress equation (20), and (Eq 4)

$\sigma_{\text{SP-A}}$ = stress calculated from the Spangler Equation with

J=3.2 for the AASHTO Road Test conditions.

Hudson showed (20) that Loop 1 stress can be used for a performance study.

Equation 4 can be written in the following form, with the J-term appearing:

$$\frac{S}{J[C]} = \frac{S_{\text{LP}}}{\sigma_{\text{SP-A}}} \quad (\text{Eq 5})$$

where

$$[C] = \frac{P}{D^2} \left[1 - \left(\frac{a_1}{L} \right) \right] \quad (\text{Eq 6})$$

Therefore, the J-term can be expressed as follows:

$$J = \frac{S \sigma_{\text{SP-A}}}{S_{\text{LP}}[C]} \quad (\text{Eq 7})$$

By adding 3.2 to the numerator and to the denominator, Equation 7 becomes:

$$J = 3.2 \frac{S \sigma_{\text{SP-A}}}{S_{\text{LP}} \sigma_{\text{SP-TX}}} \quad (\text{Eq 8})$$

where

$\sigma_{\text{SP-TX}}$ = stress calculated from the Spangler Equation with J=3.2 for the in-service pavement.

Considering the difference between the method used in estimating the stress S and the method used in obtaining the Loop 1 stress S_{LP} , a factor of F needs to be added to Equation 8 to account for this difference. This results in the following general mathematical form for the J-factor model:

$$J = 3.2 \frac{S \sigma_{\text{SP-A}}}{S_{\text{LP}} \sigma_{\text{SP-TX}} F} \quad (\text{Eq 9})$$

The determination of the value of the F-factor is described later in this section.

APPROACH TO ESTIMATING FIELD STRESSES

In the absence of efficient and economical instruments for directly measuring strains (stress) on in-service concrete pavements, we decided to estimate the field stress from deflection measurements. Specifically, the approach to estimating the stress was based on FWD measurements taken on in-service CRC pavements across Texas during the summer of 1988.

In the design of CRC pavements, a key hypothesis is that, although transverse cracks will always occur, they should be evenly spaced and have minimum width. This is accomplished through the design of the longitudinal reinforcement steel. The occurrence of cracks in CRC pavements gives rise to difficulties in determining critical stresses in the pavement. In order to find the critical stress loading position, a finite element program called ILLI-SLAB (21 and 22) was used to perform an analysis of CRC pavement.

Finite Element Analysis of CRC Pavement

A typical field section of an in-service CRC pavement was selected for the analysis. The thickness of the slab was 8 inches. Deflections were taken at the edge, corner of the crack, and in the interior as shown in Table 4. Figure 14 depicts these three loading positions for the deflection measurements. The concrete modulus of elasticity (E-value) was 5.2×10^6 psi. The modulus of subgrade reaction (K-value) was 295 pci. The K-value was assumed to be uniform beneath the slab. Poisson's ratio was assumed to be 0.2. A summary of data for this field section is presented in Table 4.

Two models were used to simulate the field conditions. The first considered the cracks while the other did not. The layout of the first simulation model (Model 1) is shown in Figure 4. Three slab segment was rounded up to a length of 6 feet; the other two on each side of the middle slab segments were set at 10 feet in length. The lane width was 12 feet. The middle slab segment was divided into 6 elements in the longitudinal direction. Seven elements with different sizes were used for each of the two slabs. The 12-foot lane was divided into 12 elements. The applied load was 9,000 pounds, and the loading area was 10.46×10.46 inches².

Shear transfer was considered at the cracks. Shear transfer across the cracks was simulated through spring elements in the finite element program. Different levels of shear transfer can be simulated by varying the spring stiffness parameter. In Model 1, five levels of shear transfer from zero (using a spring stiffness parameter of 1) to about 100 percent (using a spring stiffness parameter of 1.0×10^7) were considered.

The results of the finite element analysis are presented in Table 5. Figure 15 shows the plot of maximum deflection results of the finite element analysis, and Figure 16 shows the plot of maximum stress results.

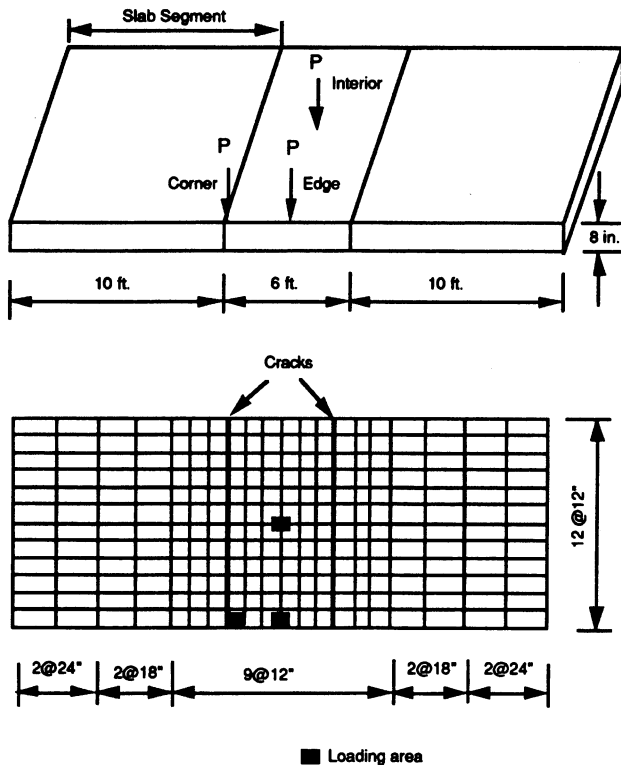


FIGURE 14 Layout of loading conditions and dimension for Model 1 and the finite-element mesh configuration used for simulation.

TABLE 4 Summary of data for the field CRCP section.

Slab thickness, $D = 8$ in.	Interior deflection, $W_i = 4.23$ mils
$E = 5.2 \times 10^6$ psi	Edge deflection, $W_e = 6.29$ mils
$K = 295$ pci	Corner deflection, $W_c = 6.48$ mils
Poisson's ratio = 0.2	Crack spacing = 5.8 feet

Note: Deflections were transformed to a load of 9000 lbs; edge and corner deflections were corrected to a zero temperature differential condition.

The second simulation model (Model 2) is a simple, continuous slab, as depicted in Figure 17. The thickness of the slab, the modulus of elasticity, the subgrade reaction, and the load are the same as in Model 1. Results of the finite element analysis for Model 2 are also presented in Table 5. For convenience of comparison, measured deflections are also included in Table 5.

As seen from Figure 15, when the spring stiffness parameter increases, deflections at the interior, corner of the crack, and edge between the cracks all decrease at varying rates. The corner deflection decreases at the fastest rate, while the interior deflection decreases at the slowest rate. For all three loading positions, when the spring stiffness parameter is greater than 1.0×10^6 , there is no significant change in deflection.

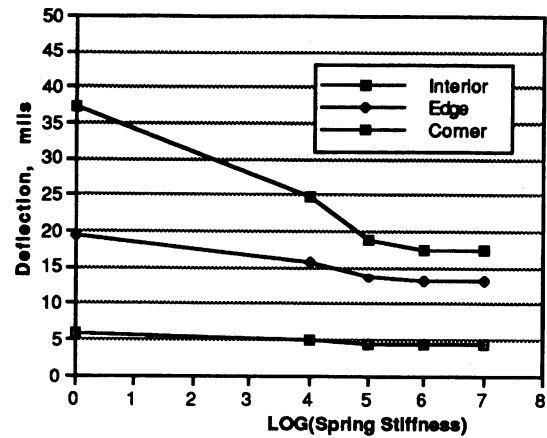


FIGURE 15 Deflections at different levels of shear transfer at the cracks for three loading positions.

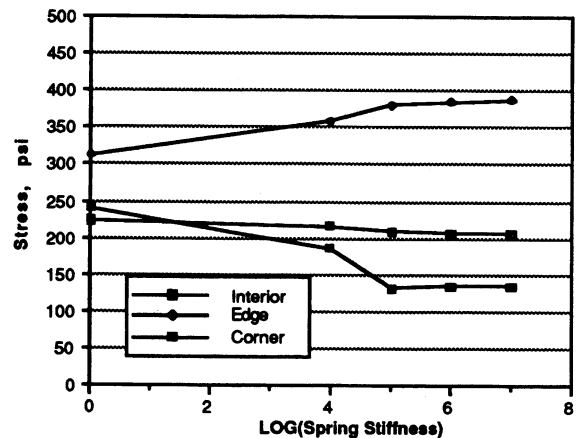


Figure 16 Stresses at different levels of shear transfer at the cracks for three loading positions.

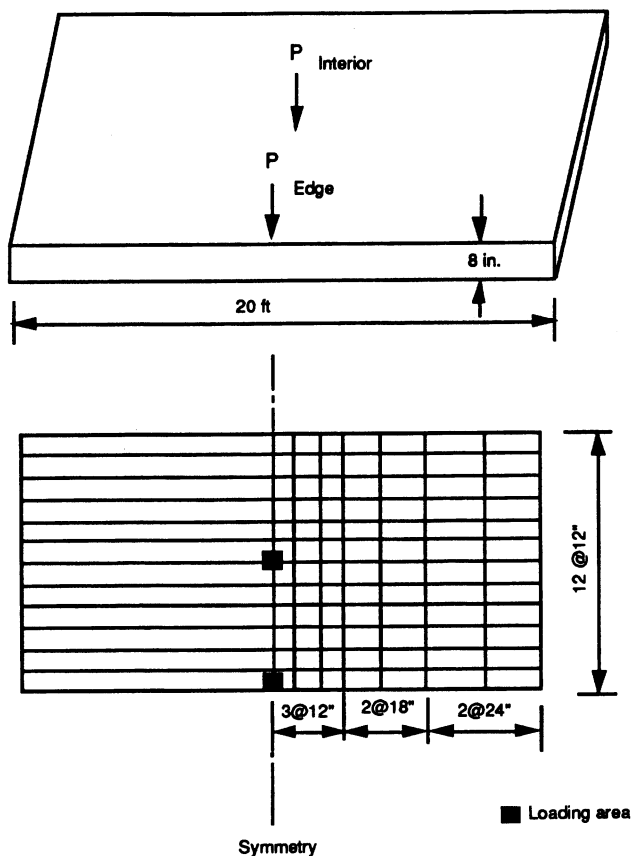


FIGURE 17 Layout of Model 2 and the finite-element mesh configuration.

TABLE 5 Finite element analysis results and corresponding field data.

	Level of Shear Transfer (spring stiffness parameter)	Loading Position					
		Interior		Edge		Corner	
		Deflection	Stress	Deflection	Stress	Deflection	Stress
Model 1	0	5.92	226	19.36	313	37.33	241
	10 ⁻⁴	4.97	217	15.79	357	24.68	186
	10 ⁻⁵	4.51	208	13.84	361	18.90	133
	10 ⁻⁶	4.43	207	13.11	366	17.61	135
	10 ⁻⁷	4.42	207	13.03	367	17.42	136
	10 ⁻⁷ + dowel	4.42	207	13.03	367	17.42	136
Model 2	Continuous Slab	4.25	200	13.02	366		
Field Data		4.23		6.29		6.48	

Deflection: mils.
Stress: psi.

It may be noted in Figure 15 that even when the spring stiffness parameter increases to a very large value, such as 1.0×10^7 , the difference in deflections at the corner and at the edge is still very large. Therefore, another strategy was sought in an attempt to reduce that difference by considering additional moment transfer across the cracks.

For the case of shear plus moment transfer at the cracks, #6 bars with 12-inch spacing were used in the simulation. The Poisson's ratio for the steel bar was 0.29, and the modulus of the steel was 2.9×10^7 psi. The spring stiffness parameter was kept at the value of 1.0×10^7 . Results of the finite element analysis are also presented in Table 5.

As seen from Table 5, the results for the case of shear plus moment transfer are the same as those having shear transfer only with a stiffness parameter value of 1.0×10^7 . This result is probably due to the finite element program limitations.

It is important to note from Figure 16 that stresses at the edge between the cracks are always larger than those at the corner near the cracks and at the interior. Therefore, for CRC pavement, the critical loading position is at the edge between the cracks.

A comparison of the results from Model 1 and Model 2 in Table 4 indicates that, with a spring stiffness parameter of 1.0×10^7 at the cracks, Model 1 yields the same deflection at the edge as does Model 2. Also, the stresses calculated from these two models are almost the same. For this case, the difference between the stress from Model 1 and that from Model 2 is less than 5 percent.

Based on these results, it may be assumed that the maximum deflection or the maximum stress from the edge loading condition in a continuous pavement slab is the same as that in a pavement with cracks with 100 percent load transfer efficiency. In other words, due to the effective 100% load transfer efficiencies, it is assumed that the subject CRC pavements act as continuous slabs in the longitudinal direction.

Relationship Between Field Stress and Theoretical Stress

As noted in Table 5, there is a significant difference in edge deflections between those calculated from the finite element analysis and those measured in the field. Therefore, stresses calculated from a theoretical model cannot be used to represent field conditions without calibrating the model. A correction for this difference between field conditions and theoretical predictions is needed before the stress obtained from the theoretical model can be used to calculate the J-value (load transfer coefficient).

For simplicity, a linear relationship is assumed between edge deflection and edge stress for estimating field edge stress. This relationship can be expressed as follows:

$$S_e = \frac{w_e}{d_e} \sigma_e \quad (\text{Eq. 10})$$

or, it may be written as

$$S_e = R_e s_e \quad (\text{Eq. 11})$$

where

S_e = estimated edge stress in the existing concrete pavement, psi

w_e = edge deflection measured in the existing concrete pavement, inch

s_e = edge stress calculated from theoretical model, psi

d_e = edge deflection calculated from theoretical model, inch, and

$$R_e = W_e/d_e. \quad (\text{Eq 12})$$

The stress calculated is assumed to be field stress, and can be used in the calculation of the J-value.

Selection of a Theoretical Model for Calculating Stresses

For the analysis of a simple slab that has free edge and corner conditions, two theoretical models are considered. They are (1) the Westergaard slab model, and (2) the finite element model. Finite element models can also be used to study the effects of cracks in the pavements; however, as can be seen from the above analyses, they cannot fully simulate actual field conditions. In other words, there are still many assumptions and limitations inherent in those finite element models. The Westergaard plate model is simple and easy to use, though it has limitations in analyzing the effects of cracks in pavement. Since the results show the in-service CRC pavements in Texas can be treated as continuous slabs in the longitudinal direction, the Westergaard slab model was selected for calculating the theoretical stress. The use of the Westergaard model provides a closed form for the J-factor model, making the calculation simple. Furthermore, only the Westergaard Interior and Edge equations are required in the stress analysis, since no critical corner condition exists in the CRC pavement.

Following his 1926 presentation of solutions to the three loading conditions, Westergaard (23) modified and developed several different solutions to the edge, corner, and interior loading conditions in a pavement slab (24 and 25; a good summary of his solution can be found in 26). The following Westergaard interior and edge equations (26) give results similar to the finite element analysis results:

(1) Interior Loading Condition

a) Stress

$$\sigma_i = \frac{3P(1+\mu)}{2\pi D^2} \left[\ln\left(\frac{2L}{a}\right) + 0.5 - \gamma + \frac{\pi}{32} \left(\frac{a}{L}\right)^2 \right] \quad (\text{Eq 13})$$

b) Deflection

$$d_i = \frac{P}{8KL^2} \left\{ 1 + \frac{1}{2\pi} \left[\ln\left(\frac{a}{2L}\right) + \gamma - 1.25 \right] \left(\frac{a}{L}\right)^2 \right\} \quad (\text{Eq 14})$$

(2) Edge Loading Condition

a) Stress

$$\sigma_e = \frac{3P(1+\mu)}{\pi(3+\mu)D^2} \left[\ln\left(\frac{ED^3}{100Ka^4}\right) + 1.84 - \frac{4\mu}{3} + \frac{(1-\mu)}{2} + 1.18(1+2\mu)\left(\frac{a}{L}\right) \right] \quad (\text{Eq 15})$$

b) Deflection

$$d_e = \frac{P\sqrt{(2+1.2\mu)}}{\sqrt{(ED^3K)}} [1 + (0.76+0.4\mu)\left(\frac{a}{L}\right)] \quad (\text{Eq 16})$$

where

$\gamma = 0.577$, Euler's constant, and other variables are as previously defined.

Therefore, the Westergaard equations given above for interior and edge loading conditions were selected for calculating theoretical edge and interior stresses.

DETERMINATION OF THE F-FACTOR

In the mathematical expression of the J-factor model as defined in Equation 9, a factor called the F-factor was introduced to account for the difference between the method for obtaining the actual pavement stress used in this study and that used in the AASHTO Road Test. Basically, the determination of the F-factor in this study is based on the interior loading condition.

In the AASHTO guide, the lowest value of the load transfer coefficient for the design of CRC pavements is 2.3. Theoretically, the interior loading represents the best condition, since the interior position of the slab has the best ability for load transfer. Also, deflections and stresses are minimum for the interior loading. Thus, a 2.3 value of the load transfer coefficient was assigned to this loading condition. In other words, if the maximum field stress is equal to the design stress calculated from the Westergaard interior stress equation, the J-factor model will give a value of 2.3 for the load transfer coefficient. For consistency, the same F-value is also recommended for jointed concrete pavements. Such an assignment yields a value of 3.2 for the load transfer coefficient for a test slab of jointed concrete pavement at the Balcones Research Center (BRC) of The University of Texas at Austin, based on field deflection measurements.

Flow Chart of the J-Factor Model for CRCP

A flow chart of the J-factor model for CRCP, depicted in Figure 18, was developed based on current conditions of the rigid pavement database. In the current rigid pavement database, slab thickness (D), concrete modulus of elasticity (E), and deflections, among other parameters for CRC pavements, are available. The deflection information includes interior deflections, edge deflections, and the corresponding load (P). In addition, the radius of the FWD load plate is known to be 5.9 inches. The Poisson's ratio of the concrete slab is assumed to be 0.2.

Assuming the deflection has been corrected for the effect of temperature differentials, the first step in the process of the J-factor model is the development of a database. The interior deflections (W_i) are used first to back-calculate the modulus of subgrade reaction (K-value) using the Westergaard interior deflection equation. Once the K-values are obtained, calculations of the interior stress, edge stress, and edge deflection are straightforward. The field edge stress is obtained by multiplying the calculated edge stress by a ratio of field edge deflection (W_e) to calculated edge deflection (d_e). Finally, maximum stress between the interior stress and the estimated field edge stress is used in the calculation of the J-value using the J-factor expression as defined in Equation 9. The F-factor can be determined once the design information is known.

Consideration of the interior stress in the calculation of the J-value can be explained by means of Figure 19. If only the edge condition is considered, then, theoretically, when edge deflection goes to zero, the ratio of edge deflection to interior deflection goes to zero. In this case, the J-value would also go to zero. However, this may not always be true for actual pavements, because, for example, when the pavement is thickened at the edge or is thickened as a result of non-uniformity, measured edge deflection may be very small compared with measured interior deflection, though interior stress may still be relatively large. In other words, the critical stress may be in the interior instead of at the edge. In this case, interior loading appears to be critical. The J-value is then determined by the interior condition instead of going to zero. The bold line represents predicted J-values for the pavements based on design values. A value of 3.0 for the ratio of edge deflection to interior deflection is used to limit the upper bound J-values as a theoretical control. The ranges of J-values recommended by the AASHTO Guide are also shown in Figure 19. It should be realized that the J-factor model is not only a function of deflection ratio R, but also a function of modulus of subgrade reaction, slab thickness, and concrete modulus of elasticity.

Figure 20 presents a plot of the obtained J-values vs. the ratio of actual edge deflection to interior deflection for Texas CRC pavements in District 20. Predicted values based on design values are also plotted in the figure, as shown by the bold line. Deviations between predicted and obtained values could be caused by variations of the thickness, K-values, and/or E-values in the field.

It should also be mentioned that the above procedure includes neither the effect of daily/seasonal temperature differentials nor the lateral distribution of load applications. This is due to the lack of actual data on these factors at the present time. It is highly recommended that such factors be included in the development of a more rational procedure.

Following a discussion of the statistical results of the J-values, this section recommends final load transfer coefficients for the design of CRC pavements in Texas. In addition, a comparison is made between the final load transfer coefficients and those previously developed.

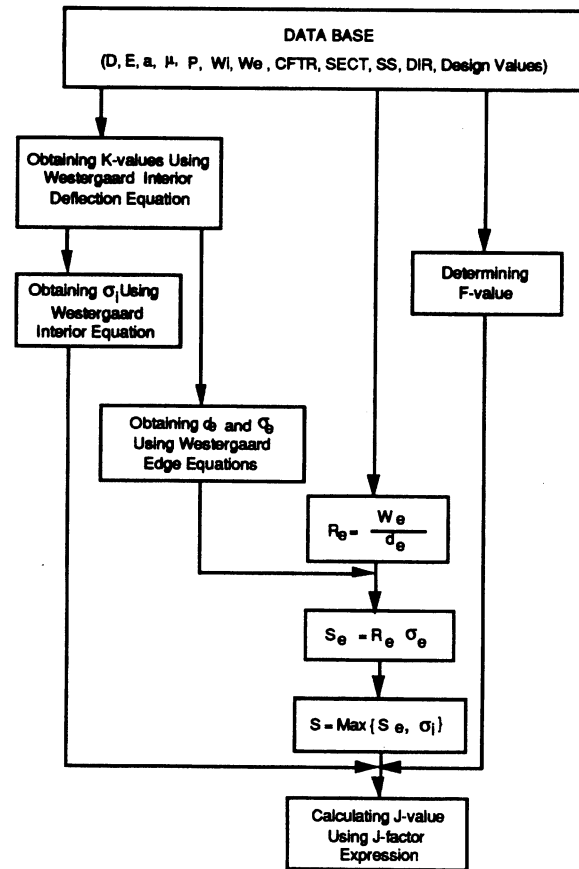


FIGURE 18 A flow chart of the J-factor model for CRCP.

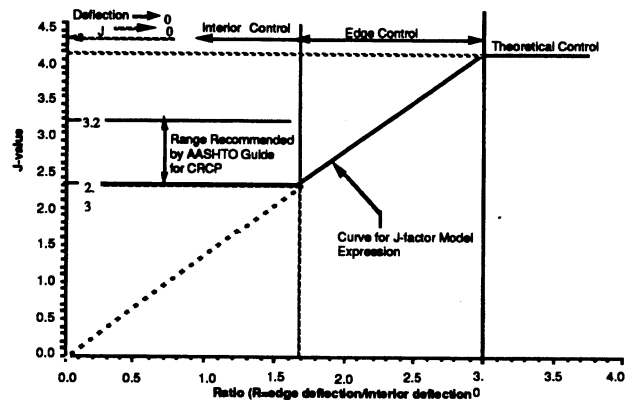


FIGURE 19 Graph of the J-Factor model for CRCP.

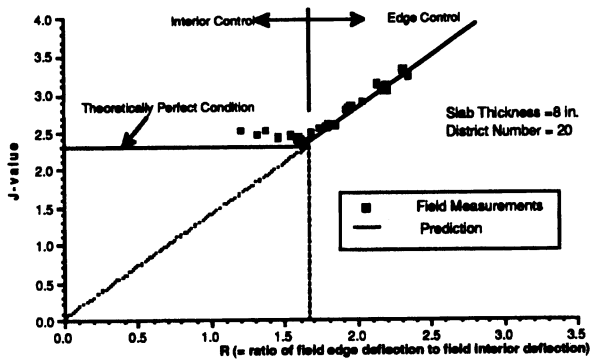


FIGURE 20 Plot of derived J-values vs. deflection ratio R.

DETERMINATION OF FINAL LOAD TRANSFER COEFFICIENTS

A summary of the statistics for the J-values determined at three different levels for CRC pavements with an AC shoulder is presented in Table 6. The summary of the statistics for the J-values at three different levels for CRC pavements with a tied PCC shoulder is presented in Table 7.

TABLE 6 Summary of statistics for the J-values, AC shoulder.

	Mean	STD	Min	Max	5%	95%
Level 1	2.4	0.30	1.7	3.7	2.0	3.0
Level 2	2.4	0.24	2.0	3.2	2.1	2.9
Level 3	2.4	0.21	2.2	3.0	2.2	2.8

As seen in Table 6, the mean J-value for CRC pavements with an AC shoulder is 2.4 at all three levels. This value can be used as a lower boundary for the J-value used in the design of CRC pavements in Texas. The 95th percentile of the J-value obtained at Level 1 — 3.0 — can be used as an upper boundary for the J-value. This value is also the maximum J-value at Level 3. In other words, there is a 95 percent confidence level this value in using in Texas. A J-value of 2.9 (the 95th percentile for the J-value at Level 2) is recommended for the design. The recommended standard deviation for the J-values of CRC pavements with an AC shoulder is 0.21.

TABLE 7 Summary of statistics for the J-values, PCC shoulder.

	Mean	STD	Min	Max	5%	95%
Level 1	2.3	0.14	2.0	2.7	2.1	2.5
Level 2	2.3	0.13	2.1	2.5	2.1	2.5
Level 3	2.3	0.10	2.2	2.4	2.2	2.4

Regarding the design of CRC pavements with a tied PCC shoulder, a J-value of 2.3 (the mean J-value determined at all three levels, as shown in Table 7, and the minimum J-value recommended by the AASHTO Guide for the design of this type of pavement) can be used as a lower boundary. The 95th percentile of the J-value determined at Level 1 is 2.5. This is recommended for the design of CRC pavements with a tied PCC shoulder. The selected standard deviation of the J-value for CRC pavements with a tied PCC shoulder is 0.08. This is a significantly lower standard deviation than that for CRC pavements with an AC shoulder, indicating the effects of tied PCC shoulders on reducing deflections and variations at the pavement edge. Considering the standard deviation of 0.08 for the J-value and a mean J-value of 2.3, a J-value of 2.6 is recommended for the design of CRC pavements with a tied PCC shoulder. This upper boundary J-value is a little higher than the maximum J-value of 2.4 at Level 3 and the maximum J-value of 2.5 at Level 2, but is lower than the maximum J-value of 2.7 at Level 1. Again, considering the J-value range of 2.3 to 2.6 for CRC pavements with a tied PCC shoulder, we selected a J-value range of 2.6 to 3.0 for CRC pavements with an AC shoulder (to minimize overlap).

COMPARISON OF THE J-VALUES DEVELOPED IN THIS PAPER WITH THOSE PREVIOUSLY DEVELOPED

Table 8 compares recommended J-values and standard deviations from this study with those developed in the past. Standard deviations are presented in parentheses near the corresponding J-values. This present study is referred to in the table as Project 1169.

It is noted that the range of J-values for CRC pavements with tied PCC shoulders recommended in this paper is the same as that in Volume 2 of the AASHTO guide. Treybig gave an estimate of the standard deviation of J-values for CRC pavements with an AC shoulder based on experience (27). This estimated standard deviation is almost the same as that obtained in this paper. Volume 1 of the AASHTO guide uses J-values somewhat higher than those obtained in this paper.

TABLE 8 Comparison of the J-values derived from this study with those previously developed.

Year	Load Transfer Coefficient		
	Procedure/Author	CRCP with an AC shoulder	CRCP with a tied PCC shoulder
1963	Hudson & McCullough (28)	2.2	/
1969	Treybig (27)	2.33 (0.19)	/
1972	NCHRP Report 128 (29)	2.2	/
1972	ACI (30)	2.2	/
1981	ARBP (7)	3.2	2.56
1986	AASHTO, Vol. 2 (27)	2.6 - 3.1	2.3 - 2.6
1986	AASHTO, Vol. 1 (1)	2.9 - 3.2	2.3 - 2.9
1990	Project 1169	2.6 - 3.0 (0.21)	2.3 - 2.6 (0.08)

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The main objective of this study was to develop load transfer coefficients for use with the AASHTO guide for the design of CRC pavements in Texas based on field measurements. In conjunction with this objective, an evaluation of structural conditions of in-service CRC pavement in Texas was performed to assess the abilities of in-service CRC pavements to transfer load across cracks and joints. A rational procedure was developed for obtaining load transfer coefficients for rigid pavements based on field deflection measurements, and field load transfer coefficients were determined using information available in the rigid pavement database at the Center for Transportation Research of The University of Texas at Austin.

The following conclusions are based on the analyses presented in previous sections and on the results obtained in this study:

1. The J-value for CRC pavements with an asphalt concrete shoulder in Texas ranges from 2.6 to 3.0. A J-value of 2.9 can be used for design of this type of pavement. The standard deviation of the J-value for this type of pavement is 0.21.
2. The J-value for CRC pavements with a tied Portland cement concrete shoulder ranges from 2.3 to 2.6. A J-value of 2.5 can be used in the design of this type of pavement. The standard deviation of the J-value for this type of pavement is 0.08.
3. Low J-value is related to subbases with high modulus of subgrade reaction, low-rainfall environments, concrete with low thermal-coefficient coarse aggregates, and light traffic conditions.
4. Texas CRC pavement design practice provides good load transfer at the cracks. Average load transfer efficiencies at the cracks are about 95 percent for in-service CRC pavements.
5. The difference between edge and interior deflections is significant. The average ratio of edge deflection to interior deflection varies with slab thickness and shoulder type. Pavements with a tied shoulder have a lower average ratio, indicating the effect of the pavement shoulder on reducing pavement edge deflections. The average ratio decreases with an increase in slab thickness.

Recommended Load Transfer Coefficients

The following recommendations are based on the results and findings of this study:

1. Different load transfer coefficients should be used for the design of CRC pavements with an AC shoulder (as opposed to those with a tied PCC shoulder). For the design of CRC pavements with an AC shoulder, the range of J-values is between 2.6 and 3.0, with a recommended value of 2.9. For the design of CRC pavements with a tied PCC shoulder, the range of J-values is between 2.3 and 2.6, with a recommended value of 2.5.
2. Different standard deviations of load transfer coefficients should be used for the reliability design of CRC pavements with an AC shoulder (as opposed to those with a tied PCC shoulder). For the design of CRC pavements with an AC shoulder, a value of 0.21 is recommended. For the design of CRC pavements

with a tied PCC shoulder, a value of 0.08 is recommended.

3. The following guidelines are recommended: a higher J-value should be used when there are large variations of temperature in the region, or when the coarse aggregates have a high thermal coefficient. Regions with more rainfall should also use a higher J-value. Pavements designed for a high percentage of heavy truck loads and high equivalent load applications should also use a higher J-value. If a stiffer subbase having a high modulus of subgrade reaction is used, a lower J-value may be acceptable.

REFERENCES

1. "AASHTO Guide for Design of Pavement Structures," Vol. 1, American Association of State Highway and Transportation Officials, 1986.
2. "AASHTO Interim Guide for Design of Rigid Pavement Structures," AASHTO Committee on Design, April 1962.
3. "AASHTO Interim Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials, 1981.
4. "AASHTO Interim Guide for Design of Pavement Structures," American Association of State Highway Officials, 1972.
5. Wei, Chao, B. Frank McCullough, W. Ronald Hudson, and Kenneth Hankins, "Development of Load Transfer Coefficients for use with the AASHTO Guide for Design of Rigid Pavements Based on Field Measurements," Research Report 1169-3, Center for Transportation Research, The University of Texas at Austin, February 1992.
6. McCullough, B. F. and Martin L. Cawley, "CRCP Design Based on Theoretical and Field Performance," Proc. of Second International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, 1981.
7. "Design of Continuously Reinforced Concrete Pavement for Highways," Associate Reinforced Bar Producers, 1981.
8. "AASHTO Guide for Design of Pavement Structures," Vol. 2, American Association of State Highway and Transportation Officials, 1986.
9. Dossey, Terry and Angela Jannini Weissmann, "A Continuously Reinforced Concrete Pavement Database," Research Report 472-6, Center for Transportation Research, The University of Texas at Austin, November 1989.
10. Owusu-Antwi, Emmanuel, A. H. Meyer, and W. R. Hudson, "Assessment of the Load Transfer Efficiency of Joints and Cracks in Rigid Pavements Using FWD," Research Report 460-2, Center for Transportation Research, The University of Texas at Austin, May 1990.
11. Weissmann, Angela Jannini and Kenneth Hankins, "A State-Wide Diagnostic Survey of Continuously Reinforced Concrete Pavements in Texas," Research Report 472-5, Center for Transportation Research, The University of Texas at Austin, August 1989.

12. Chou, Chia-pei, B. F. McCullough, and W. R. Hudson, "Development of A Long-Term Monitoring System for the Texas CRC Pavement Network," Research Report 472-2, Center for Transportation Research, The University of Texas at Austin, October 1988.
13. Weissmann, Angela Jannini, "Development of Procedures for Monitoring and Predicting the Long-term Performance of Continuously Reinforced Concrete Pavements," Ph.D. Dissertation, The University of Texas at Austin, May 1990.
14. Teller, L. W. and E. C. Sutherland, "The Structural Design of Concrete Pavements: Part 4 - The Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs," Public Roads, Vol. 17, Nos. 7 and 8, September and October 1936.
15. Ricci, Eduardo A., A. H. Meyer, W. R. Hudson, and K. H. Stokoe II, "The Falling Weight Deflectometer for Nondestructive Evaluation of Rigid Pavements," Research Report 387-3F, Center for Transportation Research, The University of Texas at Austin, November 1985.
16. "The AASHO Road Test Report 5 - Pavement Research," HRB Special Report 61E, 1962.
17. Morales-Valetín, G. E., A. H. Meyer, and W. R. Hudson, "Temperature Differential Effect on Falling Weight Deflectometer Deflections Used for Structural Evaluation of Rigid Pavements," Research Report 460-1, Center for Transportation Research, The University of Texas at Austin, February 1987.
18. Uddin, Waheed, Soheil Nazarian, W. R. Hudson, A. H. Meyer, and K. H. Stokoe II, "Investigation into Location/Temperature Parameters and Insitu Material Characterization of Rigid Pavements," Research Report 256-5, Center for Transportation Research, The University of Texas at Austin, December 1983.
19. Owusu-Antwi, Emmanuel, Alvin H. Meyer, and W. R. Hudson, "Preliminary Evaluation of Procedures for Assessment of Load Transfer Across Joints and Cracks in Rigid Pavements Using Falling Weight Deflectometer," Proc. of Fourth International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, 1989.
20. Hudson, W. R. and F. H. Scrivner, "AASHO Road Test Principle Relationships-Performance with Stress, Rigid Pavements," HRB Special Report 73, 1962.
21. Tabatabaie, A. M. and E. J. Barenberg, "Finite Element Analysis of Joined or Cracked Concrete Pavements," Paper Presented at the 57th Annual Meeting of Transportation Research Board, Washington D.C., January, 1978.
22. Barenberg, E.J., A. M. Tabatabaie, and R. E. Smith, "Longitudinal Joint System in Slip-Formed Rigid Pavements," Vol. II, Interim Report, Department of Civil Engineering, University of Illinois, Urbana, 1979.
23. Westergaard, H. M., "Stresses in Concrete Pavements Computed by Theoretical Analysis," Public Roads, April 1926.
24. Westergaard, H. M., "New Formulas for Stresses in Concrete Pavements of Airfields," Proceedings, American Society of Civil Engineers, Vol. 73, No. 1, January 1947.
25. Westergaard, H. M., "Stresses in Concrete Runways of Airports," Proc., 19th Annual Meeting, HRB, 1939.
26. Ioannides, A. M., M. R. Thompson, and E. J. Barenberg, "Westergaard Solutions Reconsidered," Transportation Research Record 1043, 1985.
27. Treybig, H. J., "Sensitivity Analysis of the Extended AASHO Rigid Pavement Design Equation," Thesis, The University of Texas at Austin, 1969.
28. Hudson, W. R. and B. F. McCullough, "An Extension of Rigid Pavement Design Methods," Highway Research Record, No. 60, 1964.
29. McCullough, B. F., C. J. Van Til, and R. G. Hicks, "Evaluation of AASHO Interim Guides for Design of Pavement Structures," NCHRP 128, 1972.
30. "A Design Procedure for Continuously Reinforced Concrete Pavements for Highways," ACI Journal, June 1972.