

CALIBRATED MECHANISTIC DESIGN PROCEDURE TO PREVENT TRANSVERSE
CRACKING OF JOINTED PLAIN CONCRETE PAVEMENTS

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ABSTRACT

A comprehensive design procedure for jointed plain concrete pavements to control transverse cracking has been developed. A rigorous mechanistic procedure to compute the slab edge stress at the critical location was developed. The design procedure directly considers the following design components:

- o slab thickness, length and width to prevent repeated load and thermal curling stress related cracking
- o shoulder/lane joint load transfer system
- o base support of the slab
- o transverse joint load transfer system to prevent faulting
- o transverse joint sealant system to prevent spalling
- o subdrainage of the cross section.

The analysis system can include a two or three layer system subjected to different axle load configurations: single, dual, dual-tandem, and tridem. Different boundary conditions or shoulder types can be analyzed including bituminous, tied concrete, or widened outer lane. Thermal curl stress is considered assuming a linear temperature differential through the slab thickness, which makes it possible to directly consider its effect on joint spacing.

The structural thickness and length design of the concrete slab is based on a fatigue damage analysis. The main improvement in this procedure is the development of a fatigue relationship based on field slab tests. Selected sections of the AASHO Road Test and the accelerated traffic tests performed by the Corps of Engineers were utilized in developing the above relationship. A user friendly computer program was developed that computes fatigue damage and estimates transverse cracking as well as joint faulting, joint spalling and PSR.

INTRODUCTION

The paper provides some results from NCHRP Project 1-26 "Calibrated Mechanistic Structural Analysis Procedures for Pavements," conducted at the University of Illinois, in design of jointed plain concrete pavements (JPCP) to control transverse cracking.

JPCP may or may not have doweled transverse joints (depending on climate and traffic level) which are spaced at no more than 20 feet.

In the past, rigid pavement design normally consisted of either a fatigue analysis or a serviceability analysis to determine the slab thickness required for the expected traffic level. Generally these procedures result in adequate designs. However, many other problems have developed due to other features being largely ignored or independently designed. Little consideration has been given to the design of the concrete pavement structure as an overall structural system. All components of a rigid pavement must be designed not only to be structurally adequate by themselves but also to function together successfully as a structural system. Failure to adequately consider each of these components and to consider their interaction as a structural system in the past has led to premature failures of many pavements.

To achieve an economical design, many design components must be combined, including slab thickness, shoulder type, type and thickness of the subbase, drainage, type of load transfer, joint spacing, etc. To achieve an optimum design, all these options must be considered simultaneously. The effect of one design component on another, that is the interaction between them, must be known. This kind of analysis is only possible with the use of a mechanistic-based design approach, which is a procedure that analytically evaluates the behavior of all components of the pavement system.

The research approach used to develop the design procedure is distress prevention oriented. Basically, the design process involves the determination of pavement characteristics that minimize the development of slab transverse cracking. One of the most important features of the proposed procedure is that the structural design of the concrete slab, is coordinated with the joint, shoulder, and subsurface drainage design to obtain an optimum pavement system. The overall procedure is illustrated in Figure 1.

SLAB STRESS ANALYSIS

Since transverse cracking of slabs is the major structural distress that develops in JPCP, the basic concept involved in the proposed slab thickness design procedure is limiting the level of transverse cracking. Therefore, it is vital to have an adequate structural model to accurately determine the significant stresses

due to traffic loads, and temperature under different boundary conditions.

With the development of the finite element method, almost any structural situation can be analyzed by using this powerful tool. However, the problem faced is that the finite element analysis cannot be easily implemented as part of a design procedure due to its complexity, computer requirements, and run time. What is needed, are alternate procedures to determine the structural responses that can handle different pavement conditions and different loading conditions based on easily usable analytical procedures. There exist regression equations based on finite element results that compute the edge stress in the slab under different loading conditions. The main drawback of these regression equations is that they are limited to the cases considered in their development.

As a part of the study, a fully mechanistic procedure to determine the maximum bending stress at the longitudinal edge of a JPCP slab, midway between the transverse joints, due to both traffic loads and temperature differentials through the depth of the slab was developed. Different axle load types under curling and different boundary conditions are analyzed using the principles of dimensional analysis (1).

The two available procedures used in developing new equations for the structural response are the finite element program ILLI-SLAB (2,3) and the computer program H-51 (4). The classical solutions, being of closed form, are not as general in application as is the finite element method. The basic differential equation for elastic slabs with unspecified boundary and support conditions is the Lagrange equation (5). This equation is the basis for both the Westergaard equation of a slab on a dense liquid (Winkler) subgrade and the slab on an elastic solid subgrade. Closed form solutions for slab edge stress are available for slabs resting on a dense liquid foundation and slabs resting on an elastic solid foundation (6). The study of slab stresses at joints is of great interest. The use of an elastic solid foundation to study stresses at joints is not possible because there is a discontinuity in the deflection profile at the joint. Thus, the only alternative is to use a dense liquid foundation (7). Therefore, in this study the Westergaard equation is used for infinite slab under free edge loading condition (8). This equation is an excellent basis for the analysis of more complex situations that are solved here.

The procedure described herein takes the Westergaard equation and introduces multiplication factors for stress due to load configuration, slab size, presence of a widened outer lane or tied shoulder, and temperature curling to eliminate Westergaard's restrictive assumptions (9). The procedure followed to obtain the equations or multiplication factors to account for the different boundary and loading conditions was based on a comparison between the Westergaard equation and results from H-51 or ILLI-SLAB computer programs using non dimensional analysis. The slab edge stress is obtained by the following steps:

(1) The maximum bending stress in the slab is obtained using Westergaard's equation with the equivalent loaded area. An equivalent radius, a_{eq} , of a multiple - wheel assembly is used as the radius of the Westergaard equation, resulting in the same stress as the one obtained from finite element method with the real wheel

configuration.

(2) Since the Westergaard equation is for an infinite slab, the first multiplication factor introduced is to correct for slab size effect.

(3) In the case of a widened outer lane, reduction factors must be introduced for the load at a specified certain distance from the slab edge.

(4) In the case of a tied concrete shoulder, the reduction factor can be applied to the stress to account for the protection that the shoulder gives to the slab.

(5) The effect of thermal curling is accounted for through the use of a modified curling stress proposed by Westergaard.

The most practical way to approach the problem of slabs with joint is to use medium-thick plate theory on which the foundation is modeled as a dense liquid. The closed form solution used in this study is the Westergaard "new" formula for edge loading for a circular load (10).

$$\sigma = \frac{3(1+\mu)P}{\pi(3+\mu)h^2} \left[\ln \frac{Eh^3}{100ka^4} + 1.84 - \frac{4\mu}{3} + \frac{1-\mu}{2} + 1.18*(1+2\mu)(a/l) \right] \quad (1)$$

where:

P= total applied load

E= elastic modulus of the slab

h= slab thickness

k= modulus of subgrade reaction

a= radius of the loaded area

l=radius of relative stiffness

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

Ioannides et al.(10) introduced the normalized length term, L/l , in order to account for slabs of finite length. In a similar way the effect of dual wheel loads may be quantified by S/a or S/l where S is the spacing between wheels (11).

Equivalent Single Area

The idea involved in this concept is to find an equivalent single loaded area that would produce the same bending stress of a multiple wheel assembly with the same applied load and tire pressure. Three load configurations were considered, dual, tandem and tridem. The equivalent radius of the loaded area was obtained by applying the Newton-Raphson iterative procedure to the Westergaard "new" formula for a circular load at the edge, using the slab edge stress obtained from ILLI-SLAB or H-51. Linear regression technique was performed using the SAS statistical software (12).

Dual Wheels

$$\begin{aligned} \frac{a_{eq}}{a} = & 0.909 + 0.339485 \left(\frac{S}{a} \right) + 0.103946 \left(\frac{a}{l} \right) - \\ & -0.017881 \left(\frac{S}{a} \right)^2 - 0.045229 \left(\frac{S}{a} \right) \left(\frac{a}{l} \right) + 0.000436 \left(\frac{S}{a} \right)^3 - \\ & -0.301805 \left(\frac{S}{a} \right) \left(\frac{a}{l} \right)^3 + 0.034664 \left(\frac{S}{a} \right) \left(\frac{a}{l} \right)^2 + 0.001 \left(\frac{S}{a} \right) \left(\frac{a}{l} \right) \end{aligned} \quad (2)$$

S = spacing between dual tires
 a = radius of the imprint of a single tire
 a_{eq} = radius of equivalent loaded area
 l = radius of relative stiffness

Statistics:

Number of Observations = 20
 $R^2 = 1$ Limits: $0 \leq (S/a) \leq 20$
 SEE = 0.022 $0.05 \leq (a/l) \leq 0.5$
 C.V. = 1.2%

Another way to analyze the effect of the interior wheel load over the slab edge stress is to consider the contribution in terms of (S/l) as displayed in Figure 2.

Tandem Wheels

$$\begin{aligned} \frac{a_{eq}}{a} = & 2.199479 + 0.74761 \left[\ln \left(\frac{t}{a} \right) \ln \left(\frac{a}{l} \right) \right] + \\ & + 0.548071 \left[\ln^2 \left(\frac{t}{a} \right) \right] - 0.486597 \left[\ln^2 \left(\frac{t}{a} \right) \right] \left[\ln \left(\frac{a}{l} \right) \right] - \\ & - 0.29507 \left[\ln^3 \left(\frac{t}{a} \right) \right] - 0.028116 \left[\ln^3 \left(\frac{a}{l} \right) \right] \end{aligned} \quad (3)$$

t = longitudinal spacing between tires
 other variables as defined above

Number of observations = 16
 $R^2 = 0.997$ Limits: $4 \leq (t/a) \leq 16$
 CV = 2.1% $0.05 \leq (a/l) \leq 0.5$
 SEE = 0.069

In a similar way as for the dual wheels, the effect of the longitudinal wheel spacing can be analyzed in terms of the contribution that one wheel has on the other wheel load, as a percentage of the edge stress, as a function of the ratio (t/l) , that is, using the principle of superposition, as displayed in Figure 3.

With the results already obtained, the edge bending stress for the case of a dual-tandem axle can easily be obtained by applying the principle of superposition.

Tridem Wheel Configuration

$$\begin{aligned} \frac{a_{eq}}{a} = & 1.542864 + 0.567442 \left(\frac{t}{a} \right) + 5.465697 \left(\frac{a}{l} \right) + \\ & + 17.371416 \left(\frac{a}{l} \right)^2 - 0.704714 \left(\frac{S}{l} \right) - 2.61563 \left(\frac{t}{l} \right) + \\ & + 0.087538 \left(\frac{t}{a} \right) \left(\frac{S}{a} \right) - 0.000359 \left(\frac{t}{a} \right) \left(\frac{S}{a} \right)^2 \end{aligned} \quad (4)$$

Number of Observations = 16
 $R^2 = 0.996$ Limits: $3 \leq (S/a) \leq 15$
 SEE = 0.17 $4 \leq (t/a) \leq 10$
 CV = 2.8% $0.05 \leq (a/l) \leq 0.3$

Slab Size Effect

The edge bending stress computed by the Westergaard solution is for infinite slabs (13). To account for the finite extent of the concrete pavement

slabs, Ioannides et al. (10) introduced the normalized length term, L/l with which a multiplication factor to account for a slab with finite length is derived.

$$\begin{aligned} \frac{\sigma_L}{\sigma_{\infty}} = & 0.582282 - 0.533078 \left(\frac{a}{l} \right) + 0.181706 \left(\frac{L}{l} \right) - \\ & - 0.019824 \left(\frac{L}{l} \right)^2 + 0.109051 \left(\frac{a}{l} \right) \left(\frac{L}{l} \right) \end{aligned} \quad (5)$$

σ_{∞} = edge stress for infinite slab length
 σ_L = edge stress for slab with length L
 other variables as defined above

Number of observations = 12
 $R^2 = 0.996$ Limits: $3 \leq (L/l) \leq 5$
 SEE = 0.0028 $0.05 \leq (a/l) \leq 0.3$
 CV = 0.29%

With respect to the slab width effect, there is no need to include any correction factor.

Widened Outer Lane

This type of concrete pavement is gaining popularity because two critical conditions (edge stress and corner deflection) are diminished, resulting in less transverse cracking and problems associated with joint deflection such as joint faulting. In this case the independent variable is the ratio $(\bar{D}/l)$. An extended outer lane of normalized width produces a reduction in the edge stress given by the $\sigma_D/\sigma_{D=0}$ ratio where σ_D is the slab edge stress with a load at a distance $\bar{D}$ from the slab edge and $\sigma_{D=0}$ is the slab free edge stress under edge loading condition. As the distance between the load and the slab edge increases, the maximum stress decreases until it converges to the interior loading condition. This effect is shown in Figure 4.

- For edge stress:

$$\begin{aligned} \frac{\sigma_D}{\sigma_{D=0}} = & -0.15743211 + 0.26935303 \left(\frac{a}{l} \right) + 0.357644 \left(\frac{1}{D/l} \right) - \\ & - 0.0589073 \left(\frac{1}{D/l} \right)^2 + 0.003486 \left(\frac{1}{D/l} \right)^3 \end{aligned} \quad (6)$$

Number of observations = 28
 $R^2 = 0.995$ Limits: $0.125 \leq \bar{D}/l \leq 3$
 SEE = 0.02 $0.05 \leq a/l \leq 0.3$
 CV = 6.9%

- For maximum bending stress:

$$\begin{aligned} \frac{\sigma_D}{\sigma_{D=0}} = & 0.454147 + \frac{0.013211}{D/l} + 0.386201 \frac{(a/l)}{(D/l)} - \\ & - 0.24565 \left(\frac{(a/l)}{(D/l)} \right)^2 + 0.053891 \left(\frac{(a/l)}{(D/l)} \right)^3 \end{aligned} \quad (7)$$

Number of observations = 20
 $R^2 = 0.98$ Limits: $0.125 \leq (\bar{D}/l) \leq 1.5$
 SEE = 0.017 $0.05 \leq (a/l) \leq 0.3$
 CV = 2.8%

For $\bar{D}/l$ ratios smaller than 0.125 it is recommended to linearly interpolate between the

correction factor for $\bar{D}/l = 1.125$ and 0 for which the multiplication factor is 1.

Tied Concrete Shoulder

A tied concrete shoulder reduces the slab edge stress due to the shear transfer across the longitudinal joint. Ioannides and Korovesis (14) demonstrated that the independent variable for this case is the (AGG/k) ratio, where AGG is the aggregate interlock factor, k the modulus of subgrade reaction, and l the radius of relative stiffness of the slab-subgrade system. As the aggregate interlock ratio increases, the edge stress decreases as it can be seen in Figure 5. The model derived from regression analysis is the following:

$$\frac{\sigma_{AGG}}{\sigma_{AGG=0}} = 0.99864 - 0.51237\left(\frac{a}{l}\right) - 0.0762\left[\ln\left(\frac{AGG}{kl}\right)\right] + 0.00315\left[\ln^2\left(\frac{AGG}{kl}\right)\right] + 0.015936\left[\left(\frac{a}{l}\right)\ln^2\left(\frac{AGG}{kl}\right)\right] \quad (8)$$

Number of observations = 16
 $R^2 = 0.988$
 $SEE = 0.0155$
 $CV = 2.45\%$

Limits:
 $5 \leq (AGG/k) \leq 50000$
 $0.05 \leq (a/l) \leq 0.3$

$$\frac{\sigma_{AGG}}{\sigma_{AGG=0}} = 1 - 0.025169\left(\frac{AGG}{kl}\right) - 0.086034\left(\frac{a}{l}\right)\left(\frac{AGG}{kl}\right) \quad (9)$$

Number of observations = 8
 $R^2 = 0.999$
 $SEE = 0.0048$
 $CV = 0.53\%$

Limits:
 $0 \leq (AGG/k) \leq 5$
 $0.05 \leq (a/l) \leq 0.3$

σ_{AGG} is the edge bending stress with aggregate interlock (at AGG/k) and $\sigma_{AGG=0}$ is the free edge slab bending stress, that is, without tied shoulder.

To estimate the nondimensional joint stiffness, AGG/k , it is suggested to use the chart that is given in Ioannides and Korovesis paper (14) which is reproduced in Figure 6 by measuring LTE with Falling Weight Deflectometer.

Effect of Temperature Differential

The maximum combined tensile bending stress under an edge load on a slab that is curled down (daytime condition) occurs at the bottom fiber of the slab at the location of the load. This is the controlling stress for design purposes due to its magnitude and location. For the case when the slab is curled in the other direction (nighttime condition), a smaller combined edge tensile bending stress occurs at the bottom fiber of the slab under the load. When the load is at the corner, the critical stress is at the top fiber of the slab but depends on the load transfer at the joint.

For nighttime conditions, the combined edge stress decreases dramatically, and in some conditions, the resulting combined edge stress is tensile at the top of the slab.

To estimate the combined stress due to load and temperature curling, some problems were encountered in analyzing the data using dimensional analysis.

Therefore, the procedure that has been common practice in the past (15,16) was followed, that is, separately analyzing the stresses due to load and curl. These two stresses are then combined using the finite element program. It is clear that they cannot simply be added; the principle of superposition does not apply because it violates the full contact assumption. Therefore, a modification to the curling must be made as follows:

$$\sigma_{combined} = \sigma_{load} + R * \sigma_{curl}$$

where:

$\sigma_{combined}$ = combined edge stress, psi

σ_{load} = Westergaard solution for edge loading, psi

R = multiplication or adjustment factor

σ_{curl} = curl stress, psi, given by the following expression:

$$\sigma_{curl} = \frac{CE\alpha\Delta T}{2}$$

C = coefficient that depends upon (L/l) ratio reproduced in Figure 7

E = modulus of elasticity of the concrete, psi

α = coefficient of thermal expansion of the concrete, $\epsilon/^{\circ}F$

ΔT = temperature differential through the slab, $^{\circ}F$

Validation of Proposed Procedure

The results obtained by following the proposed procedure were validated by comparing the results with available data obtaining excellent agreement. For more information the reader is referred to (17,18).

SLAB THICKNESS DESIGN PROCEDURE

In this section a comprehensive calibrated mechanistic-empirical procedure to design the thickness of pavement concrete slabs is presented. The basic concept involved in the proposed design procedure is to minimize the development of transverse cracking in slabs. This is achieved by performing a fatigue damage analysis using Miner's hypothesis which is then correlated with slab transverse cracking. A validation of the design procedure developed is given, as well as a sensitivity analysis.

Fatigue Model

Current Concepts

It has long been recognized that cracking can develop in concrete pavements as a result of fatigue of the concrete slabs caused by repeated wheel load applications combined with curl stress. Since the tensile bending stresses in concrete pavement slabs are critical, fatigue due to bending stress is used for slab thickness design. The Palmgren-Miner hypothesis is utilized to determine the degree of fatigue damage. This hypothesis is very simple because it assumes a linear accumulation of damage with the number of applied cycles of a given stress level. The mean failure criterion is written as:

$$\sum_{i=1}^{int} (n_i / N_i) = 1.0 \quad (10)$$

where:

n_i = number of cycles applied at stress level i

N_i = number of cycles applied to crack failure at stress level i

This hypothesis was first suggested by Palmgren (19) in 1924 for ball bearings and was used by Miner in 1945 (20) in tests with aluminum alloy specimens. According to Hilsdorf and Kesler (21), results obtained using this hypothesis on PCC beams may give conservative or unsafe predictions of the fatigue strength depending on the load program. However, additional tests indicate that the inaccuracy of the Miner's hypothesis is not significant compared to the large variability in strength and fatigue life that is typical of PCC. Hence, Ballinger (22) concluded that "Miner's hypothesis represents the cumulative damage characteristics of concrete in a reasonable manner". A Dutch investigation (23) shows that the dispersion in the number of cycles to failure in a constant amplitude test is fully explained from the dispersion in the static strength. Furthermore, by using a probabilistic approach, it can be shown that the total number of stress applications to failure is stochastic and therefore the Miner number is stochastic with an approximately logarithmic normal distribution (23, 24, 25). On the other hand, a recent experimental study performed by Oh (26) concludes that the fatigue failure of concrete is highly dependent on the magnitude and sequence of applied variable amplitude loading. Oh ends up concluding that the Palmgren-Miner hypothesis "is not directly applicable to concrete under such loading cases" (26).

Number of Load Applications to Crack Failure, N .

Due to the complexities of the mechanism of fatigue of in-service concrete slabs, many investigators and design procedures have used the results of simple loaded laboratory beam tests in predicting the allowable load applications that a concrete slab can withstand before cracking (15, 16, 27). However, researchers have concluded that "the complexities are so great and available information so limited that any laboratory curves used to estimate fatigue damage in field slabs must be 'calibrated' based on field data" (15). In contrast to this approach, the fatigue curves developed by the US Army Corps of Engineers and used for design of military and civilian airports are based on full-scale field slab tests (28).

There have been a few studies to obtain results from field slab performance data, which are incorporated into design procedures. A summary of those results obtained by different investigators is shown in Figure 8. Different approaches and assumptions were made in the development of these models using the results from the Corps of Engineers data or the AASHTO road test. With the progress made on slab stress analysis, it is believed that a more comprehensive model can be obtained using these two field slab data sources referred previously. The first source of field slab data examined was the full-scale accelerated traffic tests conducted by the Corps of Engineers from 1943 to 1973 under very controlled conditions at several locations in Ohio and at the Waterways Experiment Station in Vicksburg, Mississippi. The main purpose of this investigation was to study the behavior of full-scale concrete pavement

slabs under accelerated traffic of aircraft wheels to obtain data to be used in formulating basic design criteria for airport pavements. The properties of the pavement as constructed and after traffic were measured. The type and volume of the traffic applied as well as the pavement condition were recorded. A description of each test section, including the design and construction, is contained in references 28, 29, 30 and 31. The resulting data as given by Barker (32) were used for this study which includes 52 sections. There were actually over 60 sections in total but a few sections which did not fail were excluded.

Failure is defined when 50% of the slabs developed a first crack. Some modifications to the data were necessary to make it appropriate for use in this study. The first modification relates to the use of coverages as a measure of number of load repetitions. In highway pavements the wheel loads are highly channelized, unlike airfield pavements which are subjected to a variety of gear configurations and lateral distribution of loads. In an attempt to convert the coverages to failure into an equivalent number of stress repetitions at the edge of the slab, it was decided that a reasonable method was to increase the logarithm of the number of coverages to failure proportionally to the stress ratio between the edge stress and the stress of the loads that passes at some distance from the edge. The second modification to the data relates to the curling effect that it is not considered in the above reference. Temperature differentials for most of the tests sections throughout the year were estimated based on temperature data published in references 28, 29, 30 and 31. With these data a weighted annual average temperature differential was computed, and the combined edge bending tensile stress obtained as shown in Table 1. It should be pointed out that the critical stress in the slab given by Barker, was computed with the H-51 code and reduced by 25% to account for the approximate edge support that existed in the sections from doweled joints. This corresponds to approximately a 70 percent deflection load transfer. Finally, it was found that the flexural strength was not referred to the same age for all of these sections. It is known that the flexural strength is very significant on fatigue damage analysis, therefore, the results were normalized to 28 days, third point loading, mean flexural strength.

The second source of data subjected to analysis is the JPCP AASHTO Road Test (33). Only the sections that failed by fatigue were considered (most of the sections that failed did so as a result of excessive pumping). The placement histograms of the axle loads for the AASHTO results for each loop are known and given in reference (34). In addition to these data, the JPCP sections that were left in service on I-80 until 1974 and subjected to normal traffic were considered. In this case, the lateral distribution of the traffic was assumed to be the same as that recorded for the loops. The number of 18-kip ESALs were obtained from IDOT (35). The equivalent number of stress repetitions at the edge of the slab was computed in a similar way as previously described. It should be noted that one pass of a dual tandem axle on the slab edge is considered as a one stress application. A brief summary of the data is given in Table 2.

The failure criterion adopted in this study for the Road Test was the number of stress repetitions at 50 percent slabs cracked. It should be noted that the recorded distress corresponded to a cracking level in

ft/1000ft² of class 3 and class 4 cracks, with no distinction as to type. Most of the pavements considered in this analysis failed by transverse cracking, and the thicker slabs did not exhibit excessive pumping. Each test section was 120 ft long with 8 slabs of 15 ft length each.

A procedure similar to that proposed by McCall (36) was applied to obtain a relationship between the number of edge stress repetitions, the stress ratio, and the probability level.

The extended AASHTO data were not considered in the development of the regression equation, primarily because only one of all the sections reached the 50% slab cracking failure criteria.

The shape of the regression equation postulated corresponds to the McCall model,

$$\text{Log}_{10}N = \left(\frac{-R^{-b}\text{Log}_{10}(1-P)}{a} \right)^{1/c} \quad (11)$$

where $R = \sigma/M_r$

N = allowable number of stress repetitions to failure at constant stress level σ

P = probability level

The regression analysis yield the following results:

$a = 0.0032$

$b = 5.367$

$c = 4.394$

statistics:

Number of Observations = 62

$R^2 = 0.87$

SEE = 0.07 (of Log(Log N))

Figure 9 shows the NCHRP 1-26 50th percentile regression equation and the 50th percentile beam equation used in the Zero Maintenance study (15). From this graph it is clearly seen that the mean beam equation gives much higher numbers of allowable stress repetitions than the model derived here, especially for stress ratios that are commonly found in pavement design (i.e., < 0.8). It is interesting to note that as the stress ratio increases, the beam model and the field slab model become closer to a stress ratio of about 0.8. Beyond this value, beam data and field data start to separate. This can be explained by the plastic behavior of the material when subjected to high stress levels.

At this regard it is important to describe the process of crack formation when the slab is loaded at the edge until failure occurs. Meyerhof (37) and Losberg (38) have studied the slab behavior under ultimate strength theory. The process of failure under an edge loading condition can be described in the following manner: as the load increases, the bending stresses under the load becomes equal to the flexural strength of the concrete and the slab begins to yield. As the load continues to increase, yielding continues and the maximum negative moment increases until it also yields. At that time a complete hinge mechanism is formed and a crack develops on top of the slab extending to the free edge. This is the collapse load. Meyerhof ends up concluding that the theoretical collapse load of an edge loaded slab is more than twice the corresponding yield load; that is the flexural strength. These results are close enough to the slab field data displayed in Figure 9.

There are of course some additional differences that might affect the results. These are related to the

differences in the structural systems between the simply supported beam and fully supported slabs, and implicit differences such as size of defect relative to specimen, rate of loading, rate of crack propagation and also the laboratory control of climatic conditions for tests on beams which is believed to be a very important factor.

It is concluded that the equation developed from field slab performance data should be used in the analysis of fatigue damage. This model will likely give a more accurate estimate of slab fatigue life than the beam model.

PCC Strength

To estimate the concrete strength it is recommended that the third point flexural test be used. It seems to be the most appropriate test in that the failure is in a zone subjected to a constant moment. This test better simulates the field conditions of the slab bending due to load. In a study performed by Raville and McCormick (39), they theoretically demonstrated that the flexural strength obtained from third point loading test is about 11.5 percent lower than those resulting from center point loading. This led the authors to conclude that "center and third point loading systems cannot be interchanged if a direct comparison of modulus of rupture values is to be made". This is aggravated by the greater likelihood of imperfections in the zone of constant moment in the third point loading tests where failure occurs. A wheel rolling along a slab edge would stress each point along the slab similar to a third-point load constant moment condition.

Regarding the PCC strength increase with time, although there are some equations available in the literature, it is believed that it is better not to include this effect in the fatigue damage analysis at this time. The strength increase in conventional PCC, as well as new PCC such as fast track, is unknown. Furthermore, the error that might be introduced in the results by neglecting the PCC strength increase with time is somewhat adjusted for by the field fatigue curve in which the pavement sections strength increased over time also.

The mean concrete flexural strength at 28 days obtained from third point loading tests on beam specimens is used in the analysis. This information is available for every project that is used in the calibration between fatigue damage and slab cracking. The standard deviation and the PCC strength increase with time are not known for these projects.

With respect to the variability of the flexural strength along a project, it is pointed out that each slab will have a different strength. Therefore, each slab accumulates a different amount of fatigue damage and cracks at a different number of load applications. No adjustments are made in the fatigue damage analysis due to the flexural strength variability. At this point it is important to mention that this effect is indirectly included in the development of the fatigue curve from field slabs described previously.

Slab Support

In this study, the results of stress analysis of the concrete slab are based on plate theory in which the slab rests on a dense liquid foundation (Winkler). Therefore, the slab support is represented by the

modulus of subgrade reaction. It is known that this value is not constant over time but changes throughout the year. The fatigue damage procedure developed allows the user to use different k values on a monthly basis when the analysis is performed in that way.

Traffic Loads

Since traffic loads are one of the most important inputs in any slab thickness design procedure, the more accurate the traffic can be estimated, the better. In existing design procedures, two alternative ways to handle traffic are found. The first one and most widely used is the use of ESALs, a concept that was derived as a result of the AASHO Road Test. The second option is to use traffic load spectra.

The idea behind the ESAL concept is that any axle load can be converted into an equivalent number of 18-kip single axle load that will produce the same serviceability loss. The total traffic during the design period is the sum over the years of all the equivalent single axle load traffic. The available load equivalency factors are given in the AASHTO Guide (40). The traffic in the AASHO Road Test covered a range of axle loads from 2 to 30 kips on single axles and 24 to 48 kips on tandem axles. No tridem axles were considered. As stated above, ESALs are based on serviceability loss empirically determined from the road test. The use of ESALs in a mechanistic design approach is not adequate because of its nature and should be avoided as much as possible. The main problem of using ESALs is that the equivalency factors will have a different effect on the result of the computed fatigue damage. As an example consider a 36-kip dual tandem axle load over a 10-inch pavement slab. The equivalency factor for a terminal serviceability of 3.0 is 2.45. This implies that the serviceability loss produced by this axle is 2.45 times greater than one produced by an 18-kip single axle. In terms of stresses, the resulting edge stress due to the 18-kip single axle load is 235 psi, while the maximum edge stress for the dual tandem axle is 233 psi. These results are assuming a free edge loading, 15 foot slab length, elastic modulus of the concrete of 4,000,000 psi, and a modulus of subgrade reaction of 100 psi/in..

In lieu of these results, although load equivalency factors can be developed, it is considered that the best approach is to directly input the real projected traffic load spectra in the design procedure.

Temperature Differential

The temperature differential through the slab is defined as the difference between the temperature at the top and that at the bottom of the slab. A positive temperature differential indicates that the top of the slab is warmer than the bottom which usually occurs during the daytime. A negative temperature differential indicates that the bottom is warmer than the top, which usually occurs between evening and early morning. During winter in cold regions, positive temperature differentials might be of very short periods of time.

The temperature differential varies continually through a 24-hour daily period and also varies from day to day. There are different ways to consider the temperature differential in design as discussed below.

1. Yearly Average

The simplest way to consider the temperature differential in the fatigue damage analysis is to use a weighted annual average temperature for daytime and one for nighttime. The disadvantage of this alternative is that an analysis with detailed temperature data has to be performed first, say on a monthly basis, and the average annual temperature differential that will give the same fatigue damage obtained and used in futures design.

2. Monthly Average

A more sophisticated option is to use a mean monthly temperature differential for daytime and nighttime throughout the year. To obtain the mean values, a procedure like the one proposed by Darter (15) can be followed. From temperature data from field or by using temperature models like the one developed by Dempsey (41), obtain the temperature differential for every three hours and take the average for day and night for every month.

3. Hourly Annual Distribution

The most comprehensive and accurate means to consider the temperature effect in pavement design is to consider the mean hourly temperature differential distribution throughout the year. Due to the cost of gathering data from the field, it is highly recommended to obtain the data from calibrated climatic models like the CMS model (41). This permits the analysis to consider several years based on climatic data from meteorologic stations. This is the best temperature data available for design. As an example, the result for Urbana, Illinois is given in Figure 10. The only drawback of this procedure is that the relative frequency (that is, the percentage of time at some specific temperature differential) has to be tied to the amount of traffic over that same time. The two proposed options are:

- to consider the traffic uniformly distributed at all times
- permit the designer to input the traffic distribution for every temperature differential interval.

Critical Fatigue Point in the Slab

Field Results

The location at which cracking initiates is of vital importance in the development of a fatigue damage analysis. The first field slab test results dates back to the 1920s when the Bates Test Road was performed (42). The entire road test was laid out on a tangent. The truck traffic consisted of Liberty B trucks driven west along the north side, and east along the south side. Although the results of cracking might be contaminated with the panel size because of the slab length used, yet some important conclusions can be drawn:

- for the sections where almost the entire slab width was destroyed, the distresses always started at the slab edge and progressed across the slab. It is interesting to note that in these sections, the center of the outer rear wheels

travelled at 6 inches from the pavement edge.
- the wheel loads on the north half of the pavement, travelling at a uniform distance of 36 inches from the slab edge, produced very little damage to the test sections.

Later tests like the AASHO and Michigan road tests provided additional evidence upon the critical slab location where cracking initiates. From the AASHO Road Test, it was observed that when transverse cracking first developed, failure usually began at the slab edge at a distance of at least 5 ft from the transverse joint. Longitudinal cracking was formed only in thin slabs (2.5, 3.5 and 5.5 in.) and this was reduced or absent in 6.5 and 8 inches slabs thick. It was also noted that 81 percent of the cases where longitudinal cracking first formed, the crack originated in the outer wheel path within 3 in. of a dowel located between 2.5 and 3.5 ft from the slab edge. This might be a consequence of excessive dowel bearing stress.

Results from the Michigan Road Test (43) show that transverse cracking is the primary mode of failure for concrete slabs of 8-inch thickness and between 15 to 30 ft length.

In summary, results of the experimental tests indicate that transverse cracking is the predominant mode of failure and that it usually initiates in the center third of the slab. Generally, this type of cracking initiates at the slab edge and propagates transversely across the slab. Longitudinal cracking is a less frequent distress and usually develops in thin slabs. This distress initiates at the transverse joint, usually about 24 to 42 inches from the slab edge.

Analytical Results

In the Zero Maintenance study (15), a comprehensive fatigue analysis was conducted using Miner's hypothesis to determine the critical point in the slab where cracking should initiate. Two different locations were studied:

- axle load positioned at the transverse joint where longitudinal cracking initiates, and
- axle load positioned at the slab edge midway transverse joint where transverse cracking initiates.

The fatigue damage analysis conducted was performed by considering different truck axle weights, axle types, and lateral placement across the slab for the two axle load locations mentioned above. The slab thickness considered were 8, 10 and 12 inches with modulus of subgrade reaction of 50, 200, and 500 psi/in.. Thermal curling was not included in the analysis. The accumulated fatigue damage was computed at each 6 in. interval across the slab. A typical result for the edge loading between transverse joint is illustrated in Figure 11 for a 10 in. concrete slab. The results show that when the mean lateral placement (distance between the slab edge and the outer face of the wheels) is less than 36 in., the critical fatigue damage point is at the slab edge. Another important conclusion is that when the distance between the slab edge and the outer wheel is greater than 36 in., the critical fatigue point is no longer at the slab edge but under the wheel path. This is of extreme importance especially for the analysis of widened outer lanes. As an example, consider a concrete pavement in which the mean distance from the slab edge and the wheels is 18 inches. If a widened outer lane is used,

there is no need to extend the slab beyond 18 inches, thus obtaining a total distance of 36 inches. A larger widening will not produce any significant reduction in accumulated fatigue damage and to the contrary it can produce negative effects because the longer side of the slab will be in the transverse direction, thus increasing the curl stress in the transverse direction and also the chances for the development of longitudinal cracking.

Fatigue Damage Analysis

The slab thickness design is based on limiting the amount of slab cracking, which is controlled by limiting the total accumulated fatigue damage. The fatigue damage analysis presented here is based on the following:

- (1) The critical fatigue damage location in the slab is at the longitudinal edge, midway between transverse joints.
- (2) Critical edge stress is caused by combined traffic loads and temperature differentials through the slab.
- (3) The lateral truck distribution is considered by the effect it has on the edge stress.
- (4) Fatigue damage is computed using the Palmgren-Miner hypothesis.
- (5) A correlation between the computed fatigue damage and transverse cracking in field slabs was established.

The model for the number of stress repetitions to PCC slab cracking showed before is the fundamental basis for estimating slab fatigue life. The fatigue damage algorithms using Palmgren-Miner's Hypothesis are as follows:

- Monthly basis:

$$F.D. = \sum_{k=1}^{k=p} \sum_{j=1}^{j=2} \sum_{i=1}^{i=m} \frac{n_{ijk}}{N_{ijk}} \quad (12)$$

where:

F.D. = total accumulated fatigue damage over the design period occurring at the critical fatigue location in the slab

n_{ijk} = number of edge stress repetitions of i^{th} magnitude over day or night for the k^{th} month

N_{ijk} = number of stress repetitions of the i^{th} magnitude over day or night for the k^{th} month determined from PCC field fatigue curve (to 50% cracked slabs)

i = a counter for the magnitude of the axle load

j = a counter for day and night

k = a counter for months over the design period

m = total number of axle load groups

p = total number of months in the design period

The number of traffic loads, n_{ijk} , is computed using traffic data for the design period. It is strongly recommended that the entire axle load distribution be used. It is computed using the following expression:

$n_{ijk} = (\text{ADTT})(\text{MAX})(\text{DISTP}/100)(\text{PC})(\text{VOL}/100)*\text{TF}*30$
where:

ADTT = average annual daily truck traffic at the end of the specific month under consideration

MAX = mean number of axles per truck
 DISTP = percent axles in i^{th} load group
 PC = percent of traffic during day
 VOL = variable (percent) to account for the lateral distribution of truck traffic
 TF = factor to either increase or decrease truck volume for the specific month

If actual traffic load spectra are not available, equivalent single axles may be used as an approximation, then
 ADTT = 18 kip ESAL at the end of the specific month under consideration
 MAX = 1
 DISTP = 100

- Yearly basis with hourly yearly distribution of temperature differentials:

The expression is the same as above but instead of making the analysis for day, night, and by month, it is made for every temperature interval on a yearly basis.

These algorithms have been implemented in a computer program called ILLI-CONC (17).

FIELD CALIBRATION

Correlation Between Fatigue Damage and Slab Cracking

The fatigue analysis that has been developed is quite comprehensive in that it considers directly the effects of traffic loadings, temperature differentials through the slab, joint spacing, and many of the different boundary conditions usually found in concrete pavements. One of the most relevant features incorporated into this design procedure is that the fatigue curves are based on field slabs subjected to traffic and environmental conditions.

A database developed by ERES Consultants Inc. for the Federal Highway Administration (44), herein after referred to as ERES/FHWA, and data from sections of the AASHTO Road Test that were left as part of I-90 were used to calibrate the cumulative fatigue damage with cracking of jointed plain concrete pavements. This database contains numerous projects in-service located in different regions in the U.S., with extensive design, construction, traffic, and performance data. Field data collected and analyzed included distress, drainage, roughness, present serviceability rating (PSR), FWD deflections, destructive testing (coring and boring), and weigh-in-motion (WIM) on selected sites.

The total accumulated fatigue damage for the average slab (i.e. since the mean beam strength was used) from the opening of each project to traffic until the date of survey was computed using the computer program ILLI-CONC. A summary of the field data along with some other information is given in Table 3.

The hourly annual temperature distribution data for different geographic locations in the country were obtained using the CMS Model (41).

The traffic available for all the sections is the 18-kip equivalent single axle load. Therefore, the calibration has to be performed under this basis.

The accumulated fatigue damage was then correlated with the percent of cracked slabs, which is considered to be the most useful cracking index. The result of the calibration is displayed in Figure 12. It is observed that although some scatter of the data are present, a clear trend is observed. For Miner's damage

less than 0.1 there is virtually no slab cracking. As the accumulated fatigue damage increases beyond that point, the slab cracking also increases greatly. According to Miner's hypothesis, the total accumulated fatigue damage of 1 corresponds to failure, which represents a crack. In the case of this analysis when the mean strength and other variables were used, the total accumulated fatigue damage of 1 should correspond to a 50% slab cracking which is indicated by the intersection of the dashed lined in Figure 12. This is based on the fact that all the design inputs considered in the calibration are mean values without any adjustment for any type of variation. This means that 50 percent of values for every input are below the mean and 50 percent are above the mean. This explains the occurrence of having pavement sections with calculated accumulated fatigue damages greater than 1 without 50 percent cracking.

To obtain a better correlation, this should be performed on a slab by slab basis and not over an entire project. This procedure would require all the input data for the fatigue damage analysis for every single slab, which is not feasible. Therefore, the use of mean values are considered the best at this time.

The theoretical functional form of cracking as a function of fatigue damage, follows an S-shape trend. Theoretically, for very low fatigue damage values, no cracking would occur while larger amounts of cracking occur for higher accumulated fatigue damage. Thus an S-shape model is used as given by the following equation:

$$\%Cracks = \frac{1}{0.01 + a(b^{-\log_{10}FD})} \quad (13)$$

Nonlinear regression technique was performed using SAS (12). The results of the analysis yield the following estimates:

$$\begin{aligned} a &= 0.0713 \\ b &= 2.5949 \end{aligned}$$

It should be pointed out that this equation represents the average results and thus corresponds to 50 percent reliability, meaning one-half of the data fall on either side of the best fit curve, as shown in Figure 12.

To design for other levels of reliability, different approaches can be used. As for example, safety factors can be applied to inputs, analysis of the data in terms of failure time theory (that is, perform a survival analysis), or by far the most rational is the traditional probabilistic and reliability approach (45, 46).

In a few sections (MI, MN, and NY), the axle load distribution data from WIM stations were available. In these sections, the analysis was also performed with load spectra, so a direct comparison with the analysis based on ESAL data was possible. This comparison is shown in Figure 13.

From this figure it is observed that in most of the cases the fatigue damage based on ESALs results in slightly higher values of fatigue damage. The explanation for this behavior resides in the equivalency factors used to compute the ESALs, as well as in the case of load spectra where the application of a dual tandem axle is counted as one stress repetition.

Structural Design Verification

The most complete validation of the proposed design procedure would require the construction and performance monitoring of recommended designs in different geographic and climatic regions, and a direct comparison between their performance with the predicted performance. This is recommended for the long-term validation. For the short-term validation an alternate procedure has been used. A reasonable validation can be achieved by comparing the in-service pavement performance of selected sections against the predicted performance given by the procedure. The NCHRP Project 1-19 COPES (47) and ERES/FHWA databases are available. It is desirable to use sections that were not used in the development of the design procedure; thus the COPES database was used.

Data from New Jersey, Utah, California, Georgia, and Michigan are included with different design parameters, environmental conditions, and traffic. The design age was the age at the time of the survey. The predicted results for transverse cracking are compared with the S-Shape curve for design which is shown in Figure 14. From this figure it is observed that some cases fall below the curve, and some above the curve. This indicates that the proposed design procedure is giving average designs. It is very important, however to gather more data for further verification. Furthermore, by using this procedure, new design curves can be developed based on local experience, as for example, for each State.

Sensitivity Analysis for Slab Design

The computer program allows the user to perform several studies. As an example, the effects that different variables have on slab transverse cracking are analyzed. A small factorial was designed and displayed in Table 4. To display the main effects, the bituminous shoulder was selected. Only one case was run for concrete tied shoulder and for widened outer lane, to display the effect of the different shoulder types. The effect of slab length, slab thickness, modulus of subgrade support, modulus of rupture, elastic modulus of the concrete, and coefficient of thermal expansion of the concrete slab are included. The climatic conditions assumed for all the cases correspond to those at Urbana, Il., and the hourly annual distribution option was used.

The results are presented in a series of charts displayed in Figures 15 through 23. An additional run was made for another climate to show the difference. The selected climate corresponds to the one existing at Little Rock, Arkansas, which has a higher number of hours with positive temperature differential.

From the charts, the following conclusions are obtained for the factorial considered:

a) It appears that the most important variables controlling the transverse cracking in slabs with a bituminous shoulder are the slab thickness, slab length, and the modulus of rupture of the concrete as can be observed from Figures 15, 16 and 17. Slab thickness changes from 7 to 8 and 8 to 9 in. have a very large effect on cracking. Joint spacing changes from 15 to 20 ft have a very large effect on cracking. Flexural strength change from 500 to 600 psi has

a very large effect.

b) The coefficient of thermal expansion of the concrete, α , has a significant effect on transverse cracking due to the increase in the curl stress as is observed in Figure 18. At this point it is important to recall that the system is sensitive to the nondimensional variable $\alpha \Delta T$, where ΔT is the temperature differential through the slab thickness. This is also reflected in Figure 19 in which a direct comparison is made between the behavior of the expected slab cracking under the climatic conditions of Little Rock and Urbana. As the number of hours with positive temperature differential increases, the more damage it produces to the pavement.

c) The elastic modulus of the concrete and the slab support represented by the k-value are less important factors in transverse cracking. From Figure 20 it can be seen that as the elastic modulus of the concrete increases, the transverse cracking increases. It should be pointed out that the flexural strength was held constant as the modulus of elasticity varied. With respect to the k-value, the results displayed in Figure 21a show that as the k value increases, the slab cracking also increases. It should be pointed out that this behavior is not unique. This is because the effect of the k-value in slab cracking is associated with the slab length ratio (L/l). Usually, for high (L/l) ratios, an increase in the k-value produces a reduction in the slab edge stress, thus resulting in less accumulated fatigue damage, that is, slab cracking.

An additional case was run to obtain results independent of the slab length effect. The slab length was set to 30 ft, thus approaching the infinite slab condition. The results for this case are displayed in Figure 21b. An important conclusion reached here is that although the k-value increases, the cracking decreases. This is because the combined load and curl stress decreases with the increase of the support, for high (L/l) ratios.

d) The use of a widened outer lane to reduce the development of transverse cracking is a viable alternative, as can be seen in Figure 22. As the slab width increases, a substantial reduction of the slab cracking is experienced. It should be pointed out that little benefit is obtained by widening the lane more than $D=0.5L$. A lane wider than this will not give extra benefit, and, to the contrary, it might give more problems than benefits.

e) The use of tied concrete shoulder also produces a benefit as can be seen in Figure 23. The degree of benefit is directly related to the aggregate interlock factor between the slab and the shoulder, that is, load transfer.

CONCLUSIONS

A comprehensive design procedure for jointed plain concrete pavements has been developed that directly considers the key distress of transverse cracking. A user-friendly personal computer program called ILLI-CONC was also developed to assist in the analysis and/or design of a concrete pavement system as a whole or by considering the independent components separately. The design procedure directly considers the following design components:

- o slab thickness, length and width to prevent repeated load and thermal curling stress related cracking
- o shoulder/lane joint load transfer system
- o base support of the slab
- o transverse joint load transfer system to prevent faulting
- o transverse joint sealant system to prevent spalling
- o subdrainage of the cross section.

The most important findings are summarized as follows:

1. From closed form solutions, algorithms suitable for use on a personal computer were developed to calculate slab edge stress for a series of different boundary conditions, axle loads, and thermal curling based on dimensional analysis. These algorithms proved to be very accurate results and were validated with finite element results found in the literature.
2. The use of dimensional analysis proved to be extremely powerful and effective for developing slab stress analysis models.
3. A new fatigue model from field slab performance data was developed. To achieve slab cracking with a low number of stress repetitions, a stress ratio, σ/M_r , higher than 1.0 is required. This agrees quite well with Meyerhof's conclusions that the theoretical collapse of an edge-loaded slab is roughly 2.5 times the corresponding yield load based on concrete flexural strength. This new field model is significantly different from the laboratory beam equation, but is believed to be much more realistic.
4. From previous analytical studies and from field results, the critical condition for fatigue damage in the slab that leads to cracking is at the longitudinal edge midway between transverse joints for pavements with a bituminous shoulder. For pavements with tied concrete shoulders, the critical location is a function of the load transfer at the slab/shoulder joint. For widened outer lanes, the critical location may shift to the wheel path. For slabs without dowels, another critical point may be at the top of the slab several feet from the corner when the load is at the corner. This would cause a diagonal crack or corner break.
5. A calibration between accumulated fatigue damage and field slab cracking was performed, and a mean design curve derived from the correlation.
6. The design procedure can use either axle load distribution data or ESALs. Most of the traffic data available to designers is ESALs. Detailed axle load distribution data are often not available. Only a few

projects contained these data. Therefore, the design curve was based on ESALs. However, for the sections where axle load distribution data were available from weigh-in-motion studies, a comparison between the accumulated fatigue damage computed with ESALs and axle load distribution was made. The result of this comparison shows that for most of the cases, the total fatigue damage computed with ESALs was higher. This may be explained by the load equivalency factor used in the calculation of the ESALs. Furthermore, the analysis of load spectra in the proposed procedure counts one stress application for a dual or tridem axle load.

7. The most significant variables controlling the transverse cracking in concrete slabs with a bituminous shoulder are the slab thickness, the slab length, the flexural strength of the concrete, and the annual temperature characteristics.

8. The coefficient of thermal expansion, α , also has a significant effect on transverse cracking. The system is sensitive to the nondimensional variable $\alpha \Delta T$, where ΔT is the temperature differential through the slab thickness.

9. The use of a widened outer lane is a viable alternative to reduce the development of transverse cracking. This can be achieved with a D/I ratio of at most 0.52. An excessive widening does not provide any benefit and, on the contrary, the potential for developing longitudinal cracking increases greatly.

10. The use of a tied concrete shoulder also helps in reducing the slab transverse cracking. The degree of benefit is directly related to the load transfer efficiency at the slab/shoulder joint over time.

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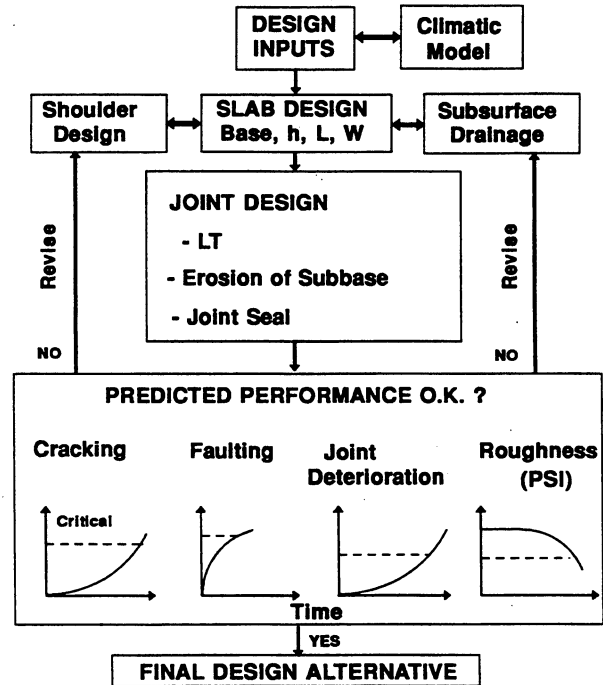


Figure 1. General design procedure.

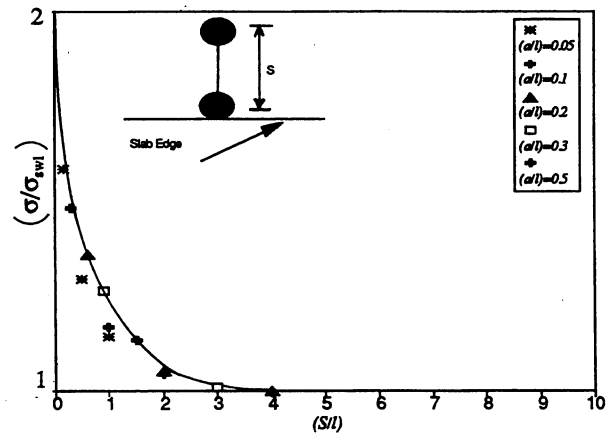


Figure 2. Contribution of inner wheel load on edge stress.

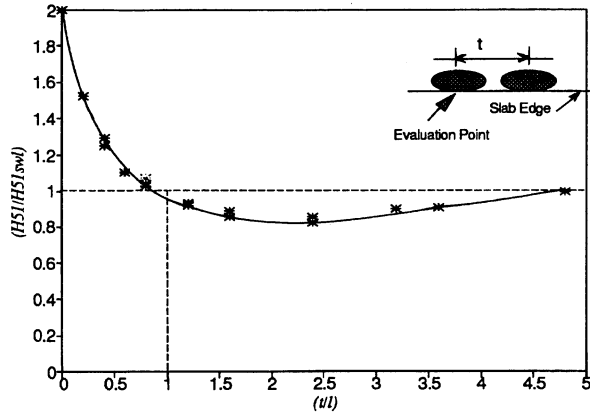


Figure 3. Effect of wheel spacing on edge stress.

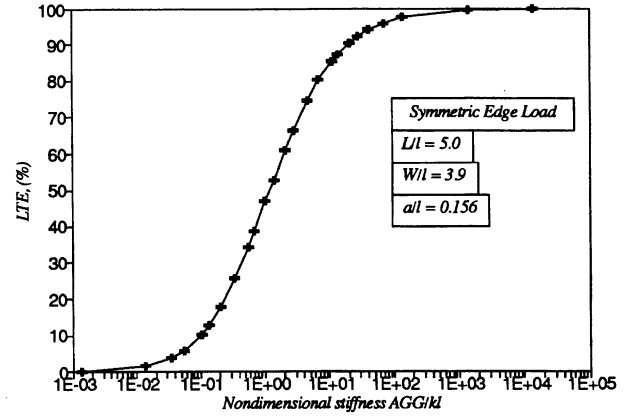


Figure 6. Deflection LTE as a function of dimensionless joint stiffness AGG/kl .

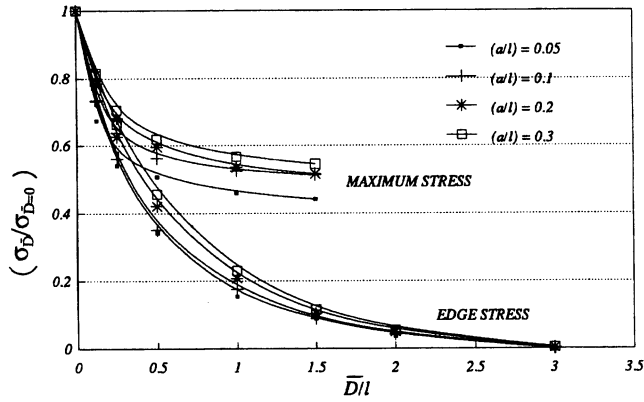


Figure 4. Multiplication factor for edge stress due to widened outer lane.

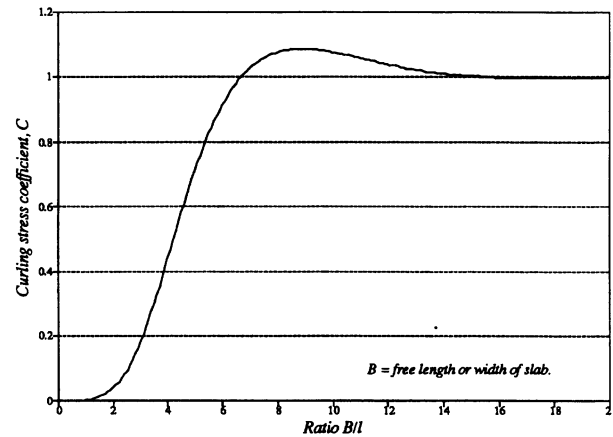


Figure 7. Variation in the differential temperature stress coefficient C with B/l .

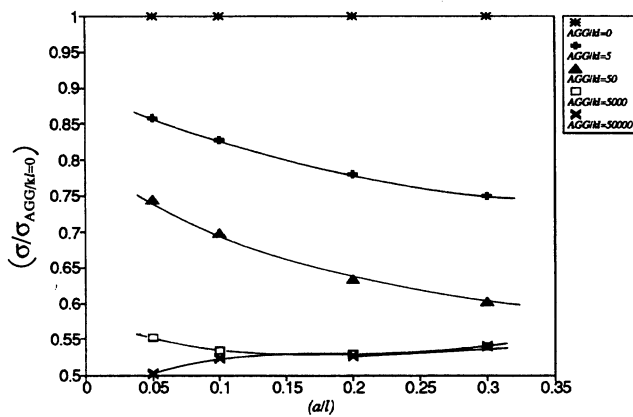


Figure 5. Multiplication factor for edge stress in slab with tied shoulder.

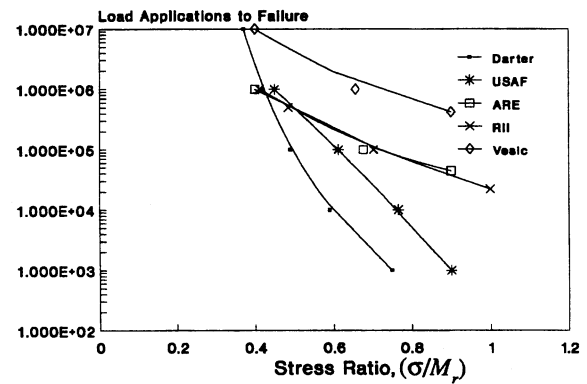


Figure 8. Summary of various PCC fatigue based design models using accelerated full-scale tests.

Table 1. Summary of data from accelerated traffic tests performed by the Corps of Engineers.

TEST SERIES	ID	h in.	k pci	Mr 28 d. psi	Reduced Edge Stress psi	ΔT oF	E Mpsi	L ft	σ Combined psi	Coverages Failure	Number Edge Stress rep.	$\frac{\sigma}{M_r}$
LOCKBOURNE NO. 1	A2	5.7	150	660	911	7.5	3.75	20	997	45	51	1.51
	B1	5.5	75	650	776	4.5	3.44	20	824	187	194	1.27
	B2	5.5	75	660	1144	7.5	3.75	20	1231	35	42	1.86
	C1	5.5	70	650	788	4.5	3.44	20	836	200	207	1.29
	C2	5.5	70	660	1164	7.5	3.75	20	1251	44	51	1.89
	D1	5.5	75	650	776	4.5	3.44	20	824	450	457	1.27
	D2	5.5	75	660	1144	7.5	3.75	20	1231	33	40	1.86
	E2	5.7	104	660	998	7.5	3.75	20	1085	77	84	1.64
	F2	7.8	52	660	803	8	3.75	20	879	110	122	1.33
	K3	9.4	90	650	729	5.5	3.73	20	783	72	79	1.20
	K2	9.4	90	660	547	7	3.44	20	610	700	713	0.92
	N2	8.0	75	625	709	5.8	3.44	20	766	150	159	1.22
	N3	8.1	75	650	935	4	3.73	20	978	9	14	1.50
	O2	9.4	75	625	569	7	3.44	20	629	573	587	1.01
	O3	9.4	75	650	764	5.5	3.73	20	816	72	79	1.26
	P2	7.6	95	625	721	5.8	3.44	20	781	262	270	1.25
	P3	7.6	95	650	937	4	3.73	20	982	6	11	1.51
	Q2	9.4	109	625	524	7	3.44	20	590	1390	1404	0.94
	Q3	9.4	109	650	693	5.5	3.73	20	749	57	63	1.15
	U2	5.8	207	625	818	4.5	3.44	20	865	88	93	1.38
LOCKBOURNE NO. 2	U3	5.8	207	650	995	4.5	3.73	20	1046	2	6	1.61
	AC	9.8	109	645	451	5.7	3.80	25	517	658	671	0.80
	E1	15.0	90	725	796	7.3	3.80	25	857	97	105	1.18
	E2	15.0	150	680	721	7.3	3.80	25	792	942	953	1.16
	E3	15.0	99	711	782	7.3	3.80	25	845	17	23	1.19
LOCKBOURNE 3 SHARONVILLE CHANNELIZED	E4	15.0	164	680	707	7.3	3.80	25	780	203	211	1.15
	E5	18.7	91	695	573	8.4	3.80	25	626	43	51	0.90
	M1	12.0	55	725	719	6.4	3.80	25	770	134	160	1.06
	NO. 3	6.0	62	800	1338	7	3.40	25	1411	18	25	1.76
	57	20.0	27	740	446	8.8	4.00	25	469	34650	36242	0.63
SHARONVILLE HEAVY MWHGL	58	18.0	30	740	519	8.2	4.00	25	546	34650	35867	0.74
	59	16.0	47	729	584	7.6	4.00	25	624	7600	7919	0.86
	60	12.0	335	729	654	7	4.00	25	737	1674	1706	1.01
	61	14.0	300	729	538	7	4.00	25	616	3867	3927	0.84
	62	16.0	361	729	428	7.6	4.00	25	509	10082	10183	0.70
	72	28.0	70	800	466	11.2	3.60	25	491	9680	10396	0.61
	73	24.0	70	800	584	10	3.70	25	614	2115	2350	0.77
	1C5	10.0	60	640	819	5.38	5.07	25	884	221	248	1.38
	2C5	12.0	70	730	632	6.2	5.50	25	701	4230	4356	0.96
	3C5	14.0	74	760	511	7	5.15	25	574	1400	1490	0.75
KLJS	4C5	8.0	74	720	1013	4.38	4.85	25	1075	180	198	1.49
	2DT	12.0	70	600	779	6.2	5.00	25	842	95	123	1.40
	3DT	14.0	74	630	637	7	4.92	25	697	205	251	1.11
	1C5	8.0	251	828	747	4.38	5.80	25	826	54	63	1.00
	2C5	11.0	100	613	641	5.88	4.86	25	714	344	373	1.16
SSPS	3C5	10.0	80	683	763	5.38	5.00	25	832	22	32	1.22
	4C5	10.0	235	695	576	5.38	5.08	25	658	6336	6385	0.95
	4DT	10.0	235	765	709	5.38	5.60	25	796	320	344	1.04
	3200	15.0	120	900	620	7.3	4.50	25	687	3215	3330	0.76
	3240	15.0	120	900	746	7.3	4.50	25	812	350	391	0.90
	4200	15.0	150	580	587	7.3	4.50	25	659	4660	4779	1.14
	4240	15.0	150	870	707	7.3	4.50	25	778	70	89	0.89

Table 2. Summary data from AASHTO Road Test.

Loop	Section	h in.	h2 in.	k pci	Rear Axle kips	p2 psi	σ Rear psi	Cur Stress psi	Mr 28 d.	EDGE S/Mr (28d)	Number Edge Stress rep.	% Slab Cracked
3	220	5	9	83	24	75	446	95	681	0.78	38123	55.5
3	226	5	3	83	24	75	446	95	681	0.78	22172	49.5
3	222	5	6	83	24	75	446	95	681	0.78	46521	37.5
3	246	5	6	83	24	75	446	95	681	0.78	54451	37.5
4	703	6.5	9	73	18	75	474	77	704	0.77	195444	51
4	650	6.5	3	73	32	75	400	77	704	0.67	19037	46.5
4	656	6.5	6	73	32	75	400	77	704	0.67	43259	78
6	370	8	9	105	48	80	397	71	679	0.68	113161	45
6	354	8	3	105	48	80	397	71	679	0.68	136136	67.5
5	507	8	9	77	22.4	75	405	61	680	0.68	211979	63
6	352	9.5	3	105	48	80	309	56	679	0.53	144531	37.5
6	368	9.5	6	105	48	80	309	56	679	0.53	235567	12.5
5	542	9.5	3	77	40	75	274	47	680	0.47	787210	12.5
4	652	8	9	73	32	75	293	60	704	0.50	5213416	25
4	672	8	3	73	32	75	293	60	704	0.50	5213416	12.5

Elastic modulus for concrete, E1 = 4,000,000 psi

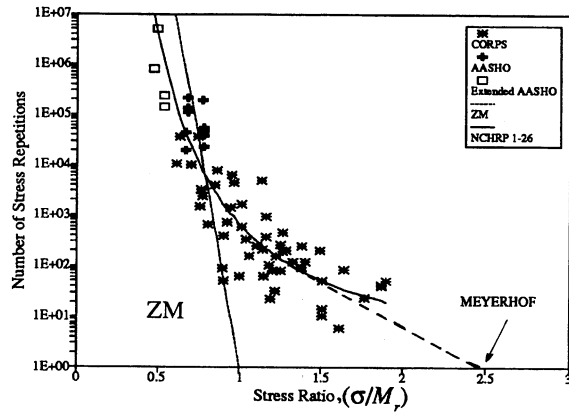


Figure 9. Summary of PCC flexural fatigue models for PCC pavements.

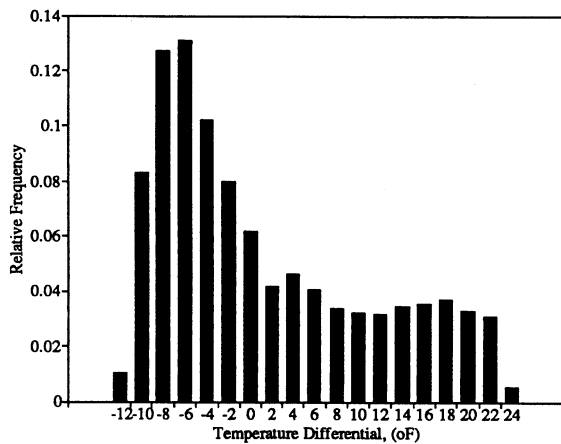


Figure 10. Relative frequency histogram for yearly temperature differential through the slab for Urbana, IL based on 30 years climatic data.

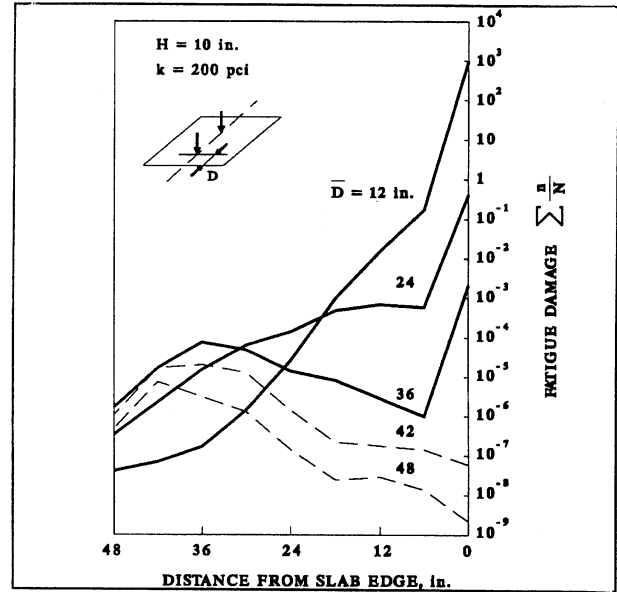


Figure 11. Computed fatigue damage across the slab due to lateral distribution of trucks in lane at midpoint between transverse joints.

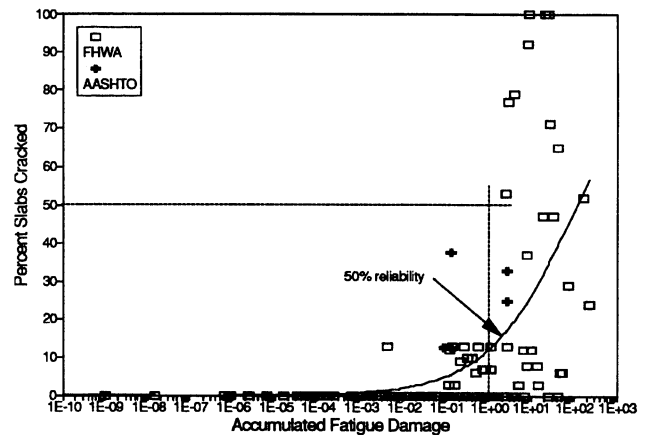


Figure 12. Proposed predictive equation for slab cracking.

Table 3. Summary of transverse cracking for JPCP sections used in the calibration.

Project ID	Slab Length ft	Slab Thick in.	EPCC Mpsi	Mr psi	Base Type	k psi/in.	Cum. ESALs Millions	AGE	Percent Slabs Cracked	Fatigue Damage
MI 1-7a	12	9	6.34	659	AGG	292	0.89	11.7	0	0.0076
	13								0	0.025
	16								0	0.3
	17								0	0.54
MI 1-7b	12	9	6.09	659	AGG	269	0.89	11.7	0	0.0063
	13								0	0.021
	16								0	0.25
	17								0	0.44
MI 1-10a	12	9	6.23	659	ATB	436	0.89	11.7	0	0.013
	13								0	0.039
	18								0	0.88
	19								0	1.2
MI 1-10b	12	9	5.28	659	ATB	502	0.89	11.7	0	0.0075
	13								0	0.032
	18								10	0.42
	19								10	0.32
MN 2-1	13	9	6.78	668	AGG	128	2.79	10.7	0	1.5E-05
	14								0	0.00013
	16								0	0.0027
	19								0	0.075
MN 2-2	13	8	8.01	668	AGG	127	2.79	10.7	0	0.018
	14								0	0.057
	16								0	0.32
	19								0	2.7
FL 2	12	13	5.55	574	SAND	378	2.01	1.4	0	5.3E-14
	13								0	3.8E-11
	18								0	0.0006
	19								0	0.003
FL 3	16	9	4.16	574	LCB	529	5.97	6.3	77	2.8
	17								79	3.8
	22								92	8
	23								100	8.5
NY 1-1	20	9	3.81	687	ATB	549	3.14	19.0	0	0.1
NY 1-6	20	9	4.02	687	AGG	619	3.14	19.0	13	0.14
NY 1-8a	20	9	4.1	687	ATB	638	3.14	19.0	3	0.16
NY 1-8b	20	9	3.88	687	ATB	548	3.14	19.0	0	0.12
NY 2-3	20	9	5.1	687	AGG	341	1.43	12.6	13	0.26
NY 2-9	20	9	5.09	687	AGG	471	1.43	12.6	9	0.21
NY 2-11	26.7	9	6.12	687	AGG	296	1.43	12.6	13	2.6
NY 2-11	30	9	4.63	618	AGG	538	9.14	19.7	3	1.4
NC 1-2	30	9	5.19	618	SC	347	9.14	19.7	6	48
NC 1-3	30	9	3.97	618	SC	494	9.14	19.7	3	5.1
NC 1-4	30	9	4.21	618	AGG	570	9.14	19.7	0	6.9
NC 1-5	30	9	5.5	618	CTB	628	9.14	19.7	0	41
NC 1-6	30	9	5.14	618	ATB	672	9.14	19.7	0	25
NC 1-8	30	9	4.22	618	AGG	513	9.14	19.7	37	7.6
NC 2	18	11	5.89	618	LCB	293	5.76	5.7	0	0.48
	19								0	1
	23								8	7.4
LA5652	25	8	4	704	Gravel	73	11.60	15.3	8	13
LA5676	15	9.5	4	704	Gravel	73	11.60	15.3	25	2.7
LA5690	15	9.5	4	704	Gravel	73	11.60	15.3	0	0.039
LA5672	15	8	4	704	Gravel	73	11.60	15.3	0	0.039
LA5702	15	9.5	4	704	Gravel	73	11.60	15.3	12.5	2.7
LA5512	15	9.5	4	680	Gravel	77	14.10	15.3	0	0.039
LA5498	15	11	4	680	Gravel	77	14.10	15.3	0	0.091
LA5510	15	11	4	680	Gravel	77	14.10	15.3	0	0.00068
LA5542	15	9.5	4	680	Gravel	77	14.10	15.3	0	0.00068
LA5530	15	11	4	680	Gravel	77	14.10	15.3	12.5	0.091
LA5352	15	9.5	4	679	Gravel	105	18.60	15.3	37.5	0.00068
LA5363	15	11	4	679	Gravel	105	18.60	15.3	0	0.0014
LA5350	15	12.5	4	679	Gravel	105	18.60	15.3	0	6.2E-06
LA5368	15	9.5	4	679	Gravel	105	18.60	15.3	12.5	0.13
LA5366	15	11	4	679	Gravel	105	18.60	15.3	0	0.0014
LA5380	15	12.5	4	679	Gravel	105	18.60	15.3	0	6.2E-06
LA5376	15	9.5	4	679	Gravel	105	18.60	15.3	0	0.0014
LA5378	15	11	4	679	Gravel	105	18.60	15.3	0	0.0014
LA5396	15	12.5	4	679	Gravel	105	18.60	15.3	0	6.2E-06
LA5390	15	9.5	4	679	Gravel	105	18.60	15.3	0	0.13
LA5398	15	11	4	679	Gravel	105	18.60	15.3	0	0.0014

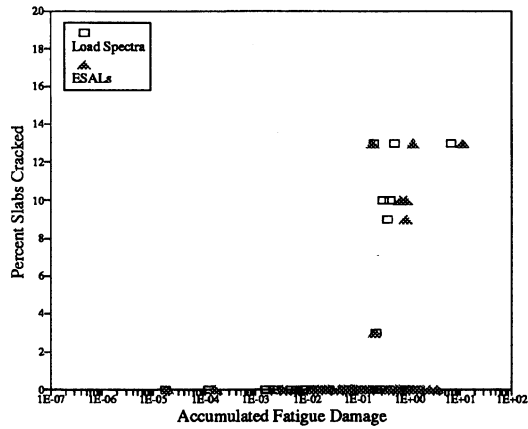


Figure 13. Comparison between fatigue damage computed based on ESALs and Load Spectra.

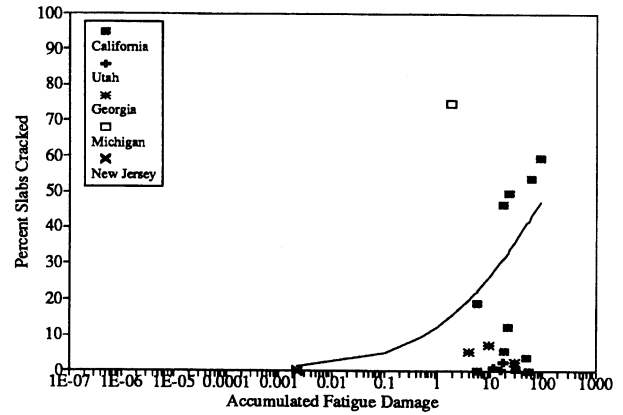


Figure 14. Validation of slab thickness design procedure with data from COPES not used in development.

Table 4. Factorial used for sensitivity analysis.

Shoulder Type	E Mpsi	h in.	Slab Length ft		k value psi/in		Modulus of Rupture psi		Coefficient of Thermal Expansion /F		AIF		WOL in.	
			min	max	min	max	min	max	min	max	min	max	min	max
Bituminous	3	10	15	15	150	150	700	700	5.5	5.5	0	0	0	0
		7	15	15	150	150	700	700	5.5	5.5	0	0	0	0
	4	8	15	15	150	150	700	700	5.5	5.5	0	0	0	0
		9	15	15	150	150	700	700	5.5	5.5	0	0	0	0
		10	10	30	50	500	550	850	4.0	7.0	0	0	0	0
		11	15	15	150	150	700	700	5.5	5.5	0	0	0	0
		12	15	15	150	150	700	700	5.5	5.5	0	0	0	0
	5	10	15	15	150	150	700	700	5.5	5.5	0	0	0	0
	6	10	15	15	150	150	700	700	5.5	5.5	0	0	0	0
Tied PCC	4	9	15	15	150	150	600	600	5.5	5.5	1	100	0	0
WOL	4	9	15	15	150	150	600	600	5.5	5.5	0	0	6	42
Bituminous	4	9	15	15	150	150	700	700	5.5	5.5	0	0	0	0

WOL = Widened Outer Lane
AIF = Aggregate Interlock Factor

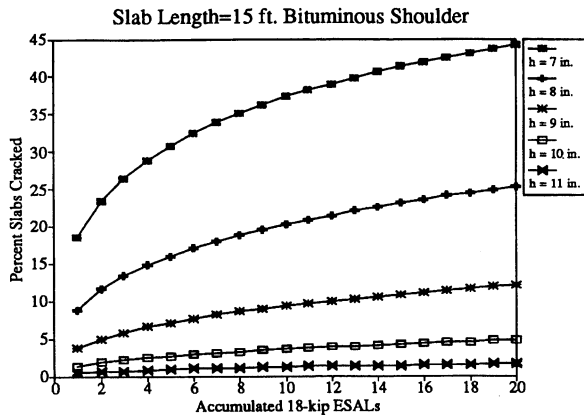


Figure 15. Effect of slab thickness in transverse cracking.

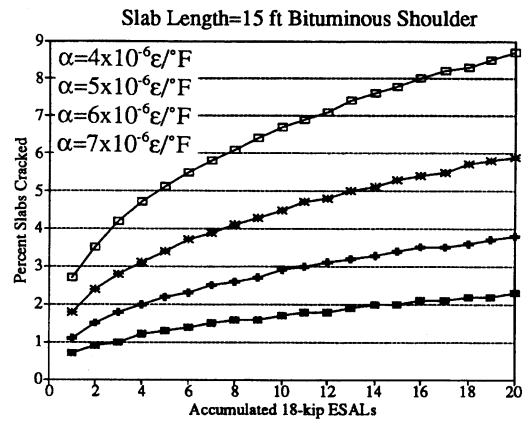


Figure 18. Effect of the coefficient of thermal expansion of the concrete on transverse cracking.

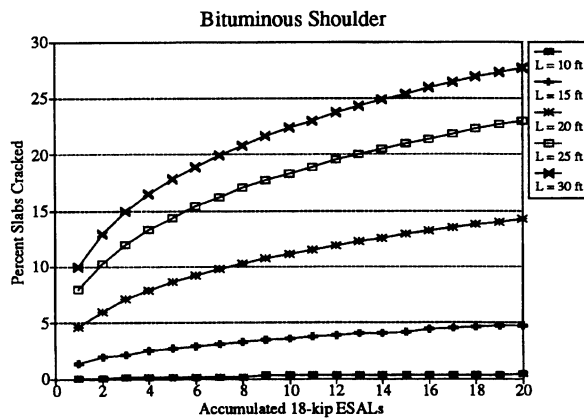


Figure 16. Effect of slab length in transverse cracking for a 10 inch slab.

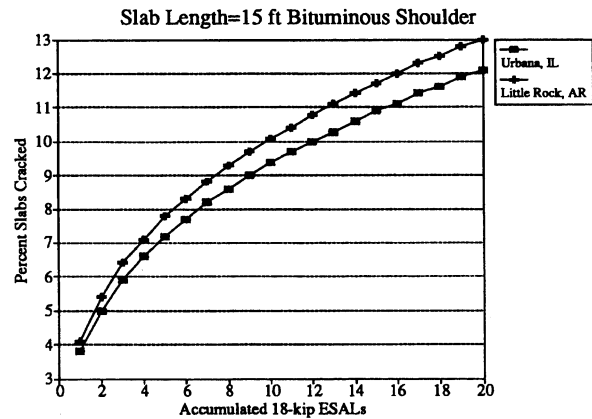


Figure 19. Effect of temperature distribution on transverse cracking.

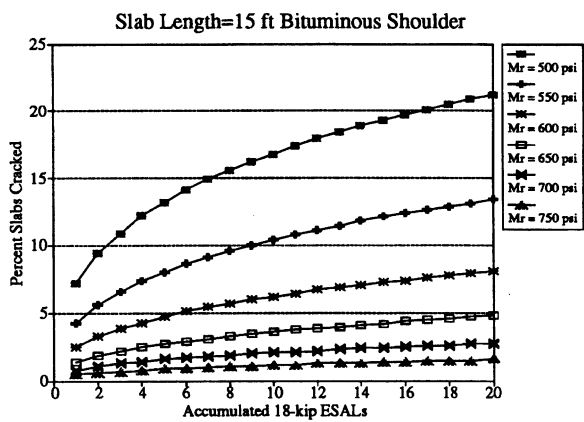


Figure 17. Effect of modulus of rupture on transverse cracking.

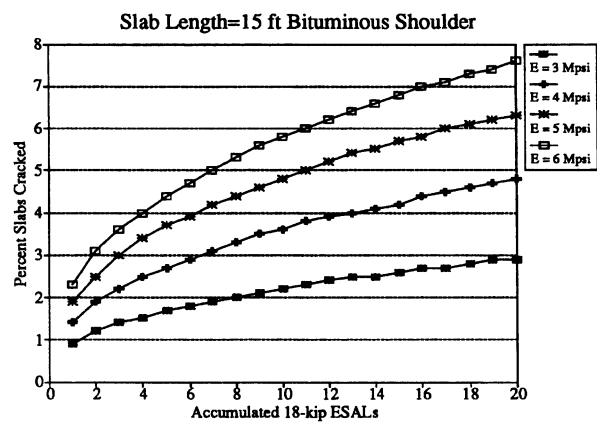


Figure 20. Effect of the modulus of elasticity of the concrete on transverse cracking.

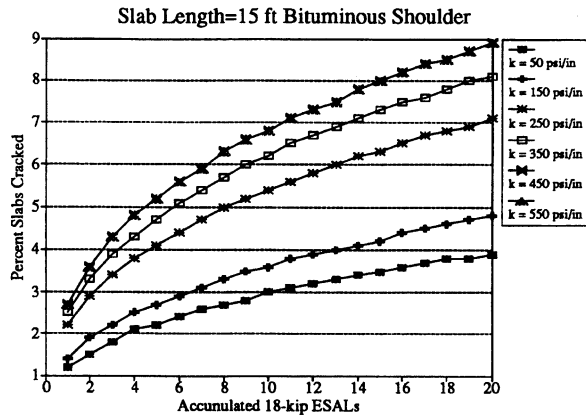


Figure 21a. Effect of the effective k-value on transverse cracking.

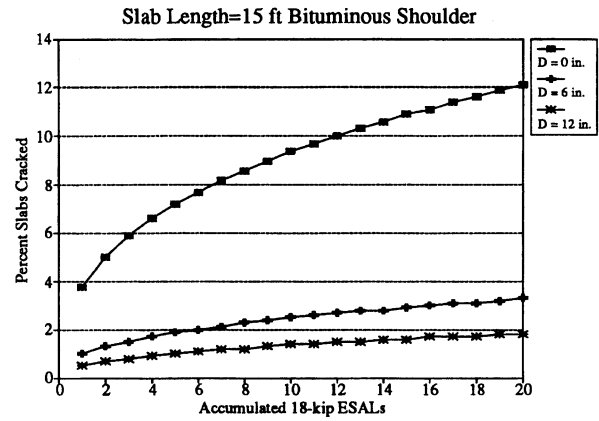


Figure 22. Effect of widened outer lane on transverse cracking.

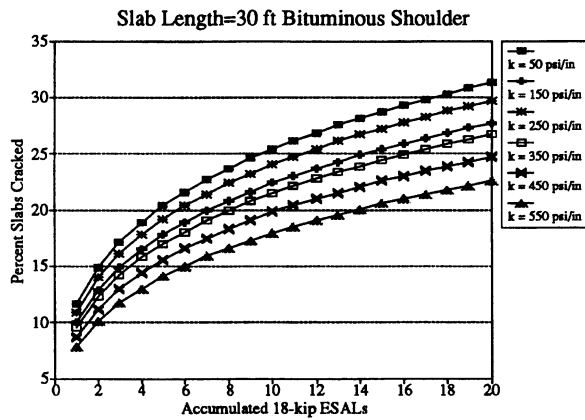


Figure 21b. Effect of the effective k-value on transverse cracking.

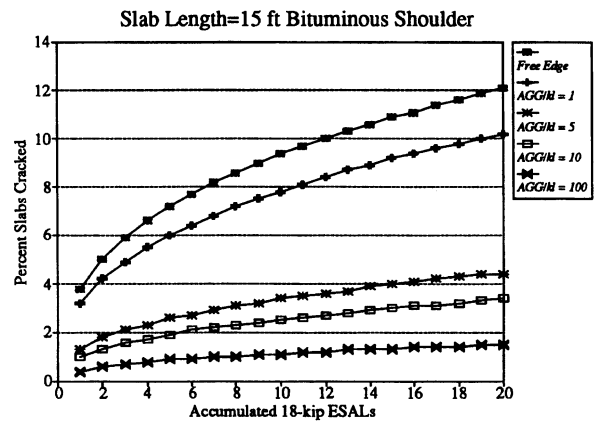


Figure 23. Effect of tied concrete shoulder on transverse cracking.