

A MECHANISTIC BASED DESIGN PROCEDURE FOR JOINTED CONCRETE PAVEMENTS

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A procedure for mechanistic based design of jointed concrete pavements is presented. This includes both jointed plain (JPCP) and jointed reinforced (JRCP) pavements. The procedure was developed specifically for pavements in Illinois, but certain adaptations can be made to the procedure to include all environmental regions of the country.

The basic design procedure is developed around the prevention of transverse cracking due to fatigue. Fatigue damage is calculated from stresses induced by traffic loads and temperature gradients in the slab. An equivalent damage ratio is used to correlate the number of passes to constitute a coverage in terms fatigue damage. Curl stresses are added to or subtracted from the load stresses depending on the time of day. Fatigue damage is calculated as a function of the shoulder type, traffic distribution, environmental factors and axle load. Material properties, joint spacing, and design reliability are also design inputs. Three shoulder types bituminous, PCC tied shoulders, and extended width traffic lanes are included in the design procedure.

The basic failure criteria used in the design procedure is the number of slabs with transverse cracks. Since the joint system was modified to prevent the random cracking of the slabs with long joint spacing, any cracking of the slabs is considered to decrease the overall pavement condition. The procedure allows the design engineer to specify the percent of slabs with transverse cracks which constitutes failure. Pavements with a lower level of traffic, can tolerate a higher percentage of cracked slabs than pavements with high level of traffic.

The analysis model used in the procedure is the finite element model

ILLI-SLAB. Since the model will permit the analysis of two layers either bonded or unbonded, the subbase is considered to be a part of the pavement system rather than a part of the support. Using this approach, the effect of stabilized subbases on the stress and deflections in the pavement system can be evaluated directly.

Introduction

Jointed concrete pavements can be either plain (JPC) or reinforced (JRC). JPC pavements normally have dowelled transverse joints spaced 20 feet or less apart, while JRC pavements have joint spacing of greater than 20 but less than 100 feet apart. Slabs with a joint spacing in feet, greater than approximately twice the thickness in inches (nominally about 20 feet for highway pavements) will usually crack at intervals between 12 and 20 feet, forming a natural hinge which relieves the curling stresses. Reinforcing steel is normally placed at the middepth in longer slabs for which cracking is anticipated to hold these cracks tightly closed, thereby, promoting load transfer across the cracks through aggregate interlock. In the case of JRC pavement designs with contraction joints at 40 foot intervals, it can be assumed that one or two transverse cracks will form between the dowelled joints. However, since curling stresses are a major factor in the thickness design, and since cracking relieves tension restraint to the extent governed by the crack spacing, the thickness design procedure should be based on the length of slab between cracks rather than on the spacing between joints. Thus, all thickness designs will be based on slab lengths of 20 feet or less even though some pavements will have a greater joint spacing. Improved performance has been noted for pavements with joint spacing under 20 feet. (19)

One method of controlling the intermediate cracking in JRC pavements is to create a hinge at specified locations. This can be accomplished with a transverse saw cut to a depth of approximately one-fourth or one-third the slab thickness. By so doing, the hinge will form as a straight line which can be sealed in the same manner as the dowelled joints. Since it is known where the hinge(s) will form, additional reinforcing steel can be placed at

these locations to aid in holding the hinge joint tightly closed, and reduce the probability of rupture of the steel through lower load transfer stresses. Problems due to corrosion effects may be mitigated by using epoxy-coated reinforcement.

For both JPC and JRC pavements a number of options must be considered to obtain economical designs. These options include:

- 1) shoulder type
- 2) strength of the concrete
- 3) design life and reliability of design
- 4) joint spacing and type of load transfer

Type and thickness of subbase and subsurface drainage are also design options which are indirectly considered. Although subbase material and thickness are important design considerations, they have only nominal impact on the design thickness.

Mechanistic design will permit the simultaneous consideration of several design options in a rational manner, which permits the designer to weigh the effects of one variable against the effects of others. By weighing one variable against others it is possible to determine the cost effectiveness of each design option and thus develop an optimum system design.

The degree of pavement performance is defined in terms of the service level. The service level of the pavement is a function of several factors including the percent slabs which have developed a transverse crack. The design procedure presented herein is structurally based to limit fatigue cracking and factors such as joint raveling and spalling must be considered separately. Correlation between the incidence cracking and the calculated fatigue damage have been developed from "in-service" pavements in Illinois but may be extended to other areas of the country. The procedure is not directly correlated with the Pavement Serviceability Index (PSI) nor does it account for the loss in performance due to other factors such as joint deterioration or "d" cracking of the slabs. Problems such as these are believed to be more material and construction oriented and therefore, are not included in this thickness design procedure.

This design procedure for jointed plain concrete pavements is mechanistic based and allows for the consideration of all of the options listed above. The design procedure is based on the elimination of transverse fatigue cracking of the slabs caused by repeated load applications and simultaneous slab curling. This was the predominant mode of failure of the PCC slabs in the AASHO Road Test and other experimental pavements (9,19). However, fatigue data from the AASHO Road Test may be biased because of the severe pumping problem which developed during the test.

Design Framework

In a mechanistic based design procedure, the pavement system is analyzed to determine its structural response (stresses, strains and deflections) of the pavement to vehicle loads. The concrete slab thickness is determined for a specified level of slab cracking, which is based on

cumulated fatigue damage determined using the calculated pavement responses.

A. Pavement Structural Response

The two-dimensional finite-element slab analysis model called ILLI-SLAB developed especially for the analysis of jointed concrete pavements was used in this study. This model is based on the classical theory for a medium thick plate on a Winkler type foundation, and was formulated to evaluate the structural response of slabs with joint and/or cracks with or without load transfer. It employs the 4-noded plate bending element (ACM or RPB12) with three degrees of freedom per node. The Winkler type foundation is modeled as a uniform support, distributed over the entire element using an equivalent mass formulation. Loads are applied uniformly over the specified elements using a work equivalent load vector (1).

The ILLI-SLAB program can be used to effectively analyze pavements which contain either one or two layers (slab and stabilized subbase) with either uniform or non-uniform stiffness, pavements with transverse and longitudinal (shoulder) joints, with any specified degree of load transfer across both the transverse and longitudinal joints. The program can also evaluate the effect of warping and curling stresses which are significant when combined with load stresses especially under non-uniform (erosion) support conditions. The ILLI-SLAB program which has been documented elsewhere (1,2,3), has been thoroughly validated by extensive use and comparison of results obtained from the program with results from other analyses and theoretical treatments where appropriate. It has also been validated using results from instrumented pavements, both as a part of this study (23) and from other studies.

The computerized ILLI-SLAB program was used to develop stress algorithms which are used in the design procedure. Certain parameters such as pavement thickness, foundation modulus, and magnitude of wheel load were found to have significant effect on the load stress. Slab length can also be a factor in the stresses caused by load, whenever the slab length is less than about 6 times the radius of relative stiffness of the slab. Ioannides (5) indicated for pavement thicknesses of 8 to 11 inches, the change in wheel load edge stress is less than 10% for joint spacing greater than 8 feet. Based on these findings, it is reasonable to ignore effects of joint spacing greater than 15 feet when considering wheel load stresses in the slab. With slab lengths less than about 8 feet there is a significant decrease in slab stresses due to load.

The load stress algorithms are based on the maximum stress at the critical location in the slab. The critical location was found to be at the pavement edge midway between joints. The magnitude of the maximum stress was also found to be a function of the type of shoulder and level of load transfer between the shoulder and the main line slab. Three shoulder types included in the design procedure are:

- 1) conventional bituminous;

- 2) portland cement concrete tied shoulders with either medium or high load transfer with the main pavement slab; and
- 3) extended width traffic lanes with either concrete or bituminous shoulders beyond the lane extensions.

The effect of the shoulder type on load stress is illustrated in Figure 1 for an 18^k axle load. The curves show a similar trends with decreases in stress with increase in thickness. The highest stresses occur with asphalt shoulder and the lowest stresses with the 10 foot tied shoulder with high load transfer (and high aggregate interlock) along the outer longitudinal joint. It should be noted, however, the critical stresses with 10 foot tied shoulders and high load transfer will rapidly increase if load transfer effectiveness is lost due to loss in ties or loss of aggregate interlock. The critical stresses will increase to a level greater than the critical stresses which occur with the 2 foot extended lane if the load transfer diminishes to any appreciable degree during the life of the pavement. As load transfer with tied shoulder decreases to a level approaching zero, stresses will approach those for the asphalt shoulder.

The load algorithm develop is presented in the form of a nomograph for the design procedure (Figure 13). The algorithm is also included in a computer program (ILLJOINT) for jointed pavement thickness design (6). The load algorithm was established from single layer analysis in which the strength and the support of the subgrade is represented by the foundation modulus (k value). The increased slab support due to a stabilized subbase layer is taken into account in ILLI-SLAB analysis by considering the subbase layer as a part of pavement system. This is accomplished by modeling the thickness and elastic properties of the stabilized subbases as a composite system with no bond between the layers. The results indicated little difference in the stresses in the pavement when the subbase layer is taken into account as opposed to when the subbase is ignored. This is particularly true for the case for a 4 inch subbase for the range of slab thicknesses greater than 8 inches. Figure 2 illustrates a comparison of subbase types and thickness requirements based on a 10 inch slab thickness under an 18^k axle load. The results for the 4 inch BAM (bituminous aggregate mixture) and the lime modified subbases are similar and can be approximated with single layer analysis. The greatest change in stress occurs with an 8 inch CAM II (lean concrete) subbase. Effects on curl stresses are not included in this analysis.

B. CURLING STRESS CONSIDERATIONS

It has long been recognized that temperature gradients through concrete slabs cause stresses which should be included in design calculations (7). Most current design procedures for rigid pavements do not take curl stress directly into account when determining slab thickness. The design philosophy, regarding curl stress considerations, has been to create joints in the slab to compensate for the curl stresses and to base the design stress upon load alone. The problem with this philosophy is that earlier investigators

have noted that curl stresses are a function of the joint spacing. This means that the optimum joint spacing cannot be rationally selected unless curl stresses are included in the total stress calculation.

The effect of the thermal gradient on the slab is to cause a convex or a concave curl in the slab. Convex curl occurs under a positive temperature gradient, shown in Figure 3, and concave curl under a negative temperature gradient(8). The tensile stress, at the pavement bottom due to the weight of the slab, is additive to the tensile stress due to wheel loads. This is noted as the most critical load condition (which occurs under a positive temperature gradient). Conversely, when the slab curls under a negative temperature gradient, the curl stress is subtractive from the load stress, which effectively reduces the total stress. Cracking related to slab curling normally occurs in the transverse direction across the traffic lane.

Westergaard (7) developed a closed form solution for calculating curl stresses for jointed concrete pavements. Bradbury (8) developed the Westergaard curl stress equations into a useful format for application to pavement analysis. The intensity of induced curl stress is equal to that part of the restrained unit strain multiplied by the modulus of elasticity of the concrete (8). The if slab were fully restrained (Figure 4), i.e., the slab remains flat, the level of extreme fiber stress would be equal to E times the thermal strain in an unrestrained slab (ignoring the Poisson's ratio affect). The fully restrained state tends to exist in the central portion of long slabs while in shorter slabs the stress will be relieved by the slab being free to curl due to the presence of transverse joints (8). As Westergaard (7) indicated in his analysis of temperature stresses in concrete pavements, the nature of the curling in a slab is governed by slab length and the relative stiffness of slab to subgrade (l) where:

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

E = modulus of elasticity of concrete
h = pavement thickness
μ = poisson's ratio
k = foundation modulus

For slab lengths 9l or less, the maximum curl stress occurs at the midslab location as illustrated. For slab lengths of greater than 12l (approximately 40 feet for typical highway pavements), the maximum curl stress occurs at approximately the 1/3 points or at 4.44l from the end of the slab.

Darter (4) also indicated the effect of joint spacing on total load stress. His study indicates a significant increase in total slab stress with increasing joint spacing up to about 25 feet (Figure 5). This is also verified by Bradbury (8). Darter (4) developed an algorithm using regression techniques to determine the curl stress for jointed pavements. The algorithm which is included in the design computer program is the basis for the curl stress nomograph used in the design procedure (Figure 14). The nomograph indicates the curl

stress is mainly a function of the joint spacing and the modulus of foundation support. As the k -value and the slab length increase, so does the curl stress. This is an important consideration when considering the type and thickness of subbase and the joint spacing. Since under curled conditions the pavement slab can act independently of the subbase, a stiff subbase may cause unnecessary increase in the curl stress. Also, curl of the slab may allow debris to accumulate between the subbase and the slab so that voids will form between the two layers. If the subbase is very stiff and cannot yield, the voids may cause stress concentrations in the slab when loaded. Because curl stresses are sometimes additive and sometimes subtractive from load stresses, it is necessary to have a good understanding of the temperature related behavior in a pavement to properly calculate the stress in a slab.

It should also be noted that the curl stress is a function of the temperature gradient. Darter's regression equation is limited to the range in the temperature gradient from $+3.0^{\circ}\text{F/in}$ to a -1.5°F/in . To simplify the computations for curling stress, weighted thermal gradients (to represent the range of variation in the gradients) were determined for the State of Illinois for both daytime and nighttime hours (discussed later). It was found that a weighted gradient of $+1.65$ degrees per inch should be applied during 25 percent of the time for the daytime gradient, a gradient of -0.65 applied for 35 percent of the time to represent the nighttime gradient and the remaining 40 percent of the time, the pavement would be analyzed as having no thermal gradient. The amount of time in which the above gradients are applicable is also an important consideration. Bradbury (8) recommended that positive (daytime), zero and negative (nighttime) gradients be applied 25, 40, and 35 percent of the time, respectively. This means the traffic distribution should also be proportioned accordingly. Upon review of records of cyclic slab temperature data from the AASHTO Road Test (9) and other field studies, the above ratios appear reasonable.

C. FATIGUE DAMAGE ANALYSIS

The primary distress component for JPC and JRC pavements is fatigue damage. The fatigue damage analysis is based on the following:

- (i) The critical location for fatigue damage is at the longitudinal edge (depending on the type of shoulder) midway between transverse joints.
- (ii) Mid-slab pavement cracking is due to critical edge stresses resulting from the combination load and slab curling related stresses.
- (iii) Load and curl stresses can be computed using the methods described previously.
- (iv) Because of the lateral traffic distribution, only a limited portion of the traffic is assumed to cause

critical stresses at the critical fatigue location.

- (v) Fatigue damage is computed using a linear damage accumulation model normally attributed to the Miner hypothesis (10).

Result from a number of laboratory and field studies on the fatigue of concrete have been reported (4,11,12). A summary and conclusions regarding fatigue characteristics of PCC are:

- (1) Failure in fatigue occurs in plain concrete members subjected to repeated flexural loads.
- (2) No fatigue limit for the material is evident up to ten million repetitions or reversals of load.
- (3) The number of cycles to failure that PCC can sustain in flexure is a function of the ratio of applied stress to the modulus of rupture (stress ratio).
- (4) There is a large variability in the strength of concrete.
- (5) Due to the variability of concrete strength, a linear cumulative damage theory for plain concrete in a reasonable failure criterion.
- (6) Increasing the modulus of rupture and thickness of the concrete will increase its fatigue life.

Result from 140 beam tests yielded a least square regression equation (4), relating the stress ratio to the number of allowable load applications prior to fatigue failure as:

$$\log N_d = 16.61 - 17.61(R)$$

where:

N_d = number of allowable load applications
 R = ratio of applied stress to modulus of rupture (stress ratio)

This equation is the fundamental basis (as a function of stress ratio) for estimating pavement life to accumulated stress applications, and hence to pavement life. Conclusions regarding the repetitive nature of loads for the concrete specimens must be based upon statistical analysis since there is a substantial variability associated with the strength of PC concrete. However, some questions do arise as the applicability of the laboratory results from beam specimens to the remaining life of actual pavement slabs under field conditions. Such factors as pavement thickness, loss of support, moisture gradient warp, rate of loading, rest periods and minimum stress periods due curl only are not accounted for in the laboratory fatigue results. Because of these uncertainties, allowable load repetitions based on stress levels of in-service slabs must be

calibrated with surveyed cracking levels.

Fatigue damage analysis using the Miner hypothesis (10) can be applied to pavement design is as follows:

$$TD = \sum_{k=1}^p \sum_{j=1}^3 \sum_{i=1}^m \frac{n_{ijk}}{N_{ijk}}$$

where:

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

n_{ijk} = number of stress replications of i th magnitude over the design period.

N_{ijk} = number of allowable axle load applications of the i th magnitude over the identical cases for N_{ijk}

i = a counter for the magnitude of axle load

j = a counter for day, night and zero temperature gradient

k = a counter for the particular case of soil k value

m = total number of single-axle load groups

p = total number of soil k value cases occurring over the course of the design period

The number of traffic loads, n_{ijk} , is computed using traffic data for the design period. If actual traffic load ranges are not available, 18-kip single loads (ESALs) can be used.

A data base developed from Illinois pavement performance data using the Concrete Pavement Evaluation System (COPES) (13) was used to calibrate the cumulative fatigue damage with PCC pavement cracking distress. The data base consists of 142 survey sections of Illinois jointed pavements from which information on pavement properties, traffic history, and pavement distresses were extracted and used into the calibration process. For each survey section, the accumulated fatigue damage was calculated and correlated with the pavement cracking. Since the calculations were based on past performance of existing pavement systems, a plot of the calculated fatigue damage against the cracking index defines the pavement cracking-cumulated fatigue curve shown in Figure 6. The regression equation for the 50% reliability curve is shown in the figure. Although the curve fit is not particularly good, the general shape of this curve is similar to those suggested by others (4,20). Pumping and similar factors may have contributed to the scatter in the data. The actual curve shape is expected to be similar in shape to an accumulation plot for normally distributed data. However, the range of data available is not broad enough to substantiate this or any other distribution. In any case cracking levels as high as 80 to 100 percent are of practically no interest. Future data should establish the distribution and the trend of the shape of the curves in this region. The calibration process has served to position the curve with respect to the

fatigue damage axis.

Medium and high severity of cracking was used to develop the fatigue damage curve. Some disparities obviously exist between these data and fatigue results from laboratory specimens since the failure of the laboratory concrete specimens failed by complete collapse whereas the field cracking was probably progressive and the cracks were allowed to deteriorate to the medium to high severity distress range before it was considered to have failed. The calibration curve in Figure 6 is the best available at this time since the performance data are not given in terms of when cracking first initiated the pavement. As more data are made available, the regression curve can be updated. Design reliability curves for 50, 85, and 95 percent were established using regression techniques.

The thickness design procedure accounts for curl stress in the concrete slab by including it in the total stress. Experience shows that the thermal gradients through a slab and the concomitant curling stresses will vary through the year. Therefore it was found beneficial to determine an effective temperature gradient in the pavement to simplify the fatigue damage calculation. Weighted (or effective) temperature gradients for the State of Illinois were determined which, if applicable year around, would produce the same fatigue damage as the actual temperature gradients calculated using monthly temperature gradient data.

D. LATERAL TRUCK TRAFFIC DISTRIBUTION DAMAGE EFFECTS

Previous studies indicate the lateral distribution of the wheel load placement has approximately a normal distribution (4). The mean distance D from the outside edge of the wheel to the pavement edge is affected by many factors (Figure 7). Such factors are location and type of edge strip, type and condition of shoulders, edge restraints such as retaining walls or guard rails, width of paving lanes, etc. Thus, only general guidelines can be given for the distance D .

The fatigue damage accumulation at the critical location is affected by the width of the lateral distribution pattern of the truck traffic. Since the critical fatigue location in the pavement is at the pavement edge, the distance of the truck tire from the pavement edge has a significant impact on the accumulation of fatigue damage at that location. This is also a function of the shoulder type. A study was made of the proportion of truck passes which should be applied at the design wheel load position on the slab to produce the same level of fatigue damage as all loads comprising the lateral distribution. The details of the study involve the determination of wheel load stresses and fatigue damage across the width of the traffic distribution at 6 inch intervals. The lateral distribution of fatigue damage calculated at 6 inch intervals (Figure 8) provided a determination of the fatigue damage at any point across the wheel path distribution. The equivalent number of load applications required to produce the same amount of fatigue damage if all loads were applied at the design load position could then be

back calculated. This number of equivalent number of wheel loads is referred to as an "Equivalent Damage Ratio (EDR)." It is essentially a pass to coverage ratio based on fatigue at the critical location.

Figure 8 illustrates the change in accumulated fatigue damage with the change in traffic distribution for the case of bituminous shoulder. The largest amount of fatigue damage occurs at the longitudinal edge. Traffic distributions with $\bar{D} = 18$ and $\bar{D} = 12$ inches are shown. The results from this analysis indicate for a $\bar{D} = 18$ ", damage ratio of approximately 0.05 is appropriate. That is, approximately 5 percent of the traffic if applied at the design wheel load location would cause the same amount of fatigue damage as accumulated under the full traffic distribution. Other investigations (4) have indicated the critical percent of the truck traffic within 6 inches of the slab edge to be around 6 percent. This is reasonably close to the 5 percent (EDR) found in this study. Table A contains a listing for various cases of shoulder types of equivalent damage ratio versus mean distance, $\bar{D}$.

E. JOINTS

From a design standpoint, joints are placed in concrete pavements to control cracking and limit the magnitude of curl stress. A major distress associated with transverse joints is faulting which can be severe enough to make driving difficult.

Causes of faulting has been well documented(21,22). Basically, faulting occurs when free water under the slab removes erodible material from underneath the leave slab and redeposits it under the approach slab thus causing the approach slab to be raised. Insufficient load transfer allows the slabs on the approach and leave slabs to deflect independently, thereby causing the free water to move very rapidly from one side of the joint to the other carrying erodible materials with it. If sufficient load transfer across the joint could be maintained, the differential joint deflection between slabs would be minimized, which would significantly reduce the backward pumping of fines as a load passes over the joint.

There are several factors which affect the quality of the load transfer: These include:

- (1) Joint opening (aggregate interlock)
- (2) Foundation support
- (3) Mechanical load transfer devices

Controlling the above factors gives the designer some control on the joint movement, and have a significant impact on the performance of the joint.

Load transfer by aggregate interlock has been shown to be a function of the joint opening (14). Load transfer efficiency is drastically reduced with increasing joint opening. For this reason, joint opening should be minimized as much as possible.

Use of mechanical load transfer devices has proven to be an effective deterrent to joint faulting. Results from several field studies have shown that doweled transverse joints have a significantly lower tendency toward faulting than similar joints without dowels. Use of tied PCC

shoulders will also provide additional support and aid in reducing joint faulting.

As discussed previously, temperature gradients through the slab cause curl stress in the concrete and curl stress increases with increasing joint spacing. If the joint spacing is sufficiently long (approaching 9 l) these stresses will be great enough to produce transverse cracks especially when combined with load stresses. In general it can be expected that whenever the joint spacing is greater than about 20 feet an intermediate crack will form between joints. For design purposes it is assumed that at least one and maybe two cracks will form between joints in pavements with joint spacing of 40 feet. Figure 9a shows a typical layout of a 40 foot slab in with the locations of the most likely transverse cracking in Figure 9b. Figure 10 shows two methods to control transverse slab cracking. The joint types are shown in Figure 11.

Two hinge joints (Type B) are shown in Figure 11. Hinge joints can be formed by placing #6 deformed bars 36 inches long on 18 inch centers and sawing the slab to a depth of approximately one forth to one third the slab thickness, but making sure the reinforcing steel is not cut in the process. The sawcut will cause the slab to crack along a straight line and will relieve the curl stresses in the slab. The straight joint will also allow easy maintenance of the joint. The joint transfers the load by aggregate interlock, with the joint held tightly closed by the deformed bars.

Some pavements have been constructed using welded wire mesh only to hold the joint or crack closed. This is not recommended for heavily trafficked pavements since the mechanism for load transfers of this joint is by aggregate interlock. As demonstrated in the AASHO Road Test, if the mesh fails the joint open and faulting will develop (4,9).

Joint type A shown in Figure 11 is a dowelled joint. Joint type C, also shown in Figure 11, is normally used for the center line joint for which the load transfer is maintained by aggregate interlock and the joint held closed by the tie bars. The longitudinal shoulder joint can be either joint type D or E shown in Figure 11. These joints are designed for the tied shoulder option and can be considered as either a high or medium load transfer as indicated.

Figure 10b shows a 40 foot jointed pavement similar to Figure 10a except it has only one hinge joint placed at the slab midpoint. A variation of Figure 10b is a 20 - foot joint spacing layout, where each transverse joint is doweled. This joint layout is recommended for heavily trafficked pavements.

F. THICKNESS DESIGN PRINCIPLES RELATED TO JOINTED CONCRETE PAVEMENT

The usefulness of this design procedure are the tradeoffs and flexibility afforded to the designer between design parameters, level of cracking and design reliability. The reliability curves provide the designer a probabilistic approach to the design process. Since fatigue failure does not occur in all slabs simultaneously, the design reliability is related to the

probability that a given percentage of cracked slabs will not be exceeded during the pavement design life. The design procedure provides for the designer to select a design reliability level. The level of service is a combination of cracking level and the design reliability. The design reliability levels as stated previously, range from 50, 85 to 95 percent. Design reliability and cracking level are components of service level.

The terminal service level for a pavement is related to the type of facility being designed. Obviously, the final service level for a freeway or interstate pavement will be higher than for a two lane rural road. The selection of the terminal service level for the pavement will establish the percent of slabs which are cracked at the time the pavement reaches the final service level. Based on the input data taken from the COPES data base, a designer can reasonably design pavement thickness between cracking levels 8% and 50%. Through the use of the calibration curve, the designer is afforded the flexibility of choosing tradeoffs between cracking level and reliability of the design life for a given fatigue damage. A cracking level of 50% is considered to be total failure. Most designs should be designed for cracking levels between 8 and 20 percent. This cracking level is expected to correlate with a high level of PSI (4.5 to 5.0) prior to significant loss in pavement performance. The PSI will decrease as slab cracking continues to develop. Actual correlation between PSI and slab cracking can be achieved as more pavement performance data are made available for this correlation.

The type of subbase is not a direct input into the design procedure. Highly erodible subbase materials are not acceptable. If subbase erosion is permitted to occur, it is nearly impossible to develop a design procedure based on mechanistic principles. This requires a highly erosion resistance subbase which is stabilized using cement, cement and flyash, lime and flyash with a compressive strength of 1000 psi or greater, an asphaltic concrete subbase, or an open-graded granular material placed on either a lime treated subgrade or on the natural subgrade with a granular filter layer between the open-graded layer and the subbase soil. Experience in Illinois has indicated that some stabilized subbases will erode, however, field tests on these subbases showed the compressive strength to be well below 1000 psi.

The functions of the subbase in this design procedure are:

1. to provide a stable construction platform,
2. to control the depth of frost penetration,
3. prevent erosion of the pavement support,
4. provide uniform slab support,
5. facilitate drainage,
6. provide increased slab support.

If rigid, thick subbases are used, it is recommended that the system be analyzed as a two layer system rather than assigning a pseudo "k" value to the top of the subbase layer. When non-stabilized subbases are used without any stabilized layers, then the "k" value on the top of the subbase is taken to be the same as the "k" for the natural soil.

Current IDOT design policy specifies a 4 inch thick stabilized subbase. It was determined that the stiffness of this subbase is not sufficient to warrant two layer analysis using the ILLI-SLAB program. Therefore, the wheel load and curl stresses for this procedure are based on single layer analysis and provide accuracy of at least as great as some of the other inputs which must be used in the design process. If stiffer subbases are incorporated into the pavement section, special consideration must be made for curl related stresses.

The design procedure is developed to consider three types of shoulders: 1) bituminous surfaced with granular or stabilized bases; 2) tied concrete; and 3) extended width driving lane with bituminous or concrete shoulders beyond the lane extensions. Both the tied concrete and the extended width driving lane systems will require less pavement thickness than when using conventional PCC pavement with bituminous shoulders.

In terms of material properties, the elastic constraints E and μ have little impact on a design. Subgrade support (k value) has a relative insignificant effect on the design thickness, within a realistic range the range of "k" is between 50 pci to 200 pci. The design procedure can except a wide range of "k", however the range in "k" pointed out above is reasonable for Illinois. The modulus of rupture of the concrete does have a significant affect on the design thickness. Consideration may be given to design of using the IDOT minimum acceptable 14-day MR of 650 psi to determine a design value. It is recommended a coefficient of variation of 15 percent for the strength evaluation be implemented.

Previously, it was pointed out for pavements with the range of joint spacing greater than 8 ft, the wheel load stress did not vary significantly. However, this is not the case in terms of fatigue damage. Increased curl stress resulting from the increased joint spacing, will cause the fatigue damage to increase significantly with the longer joint spacing. Joint spacing in the range of 15 to 20 feet have proved to perform effectively and are recommended.

DISCUSSION OF DESIGN INPUTS AND OPTIONS

A brief review of the design procedure is provided in this paper. A more thorough review is given in reference 6. Basically, the procedure consists of the selection of the design inputs and design options to be considered in the design, analyzing the pavement systems for fatigue and comparing the design life and reliability with the level of service desired. If the level of service is other than desired, a new combination of options is selected and the pavement reanalyzed as before. Iterations of selection of options and analysis are continued until acceptable pavement designs are developed.

The procedure inputs required for thickness design are:

1. subgrade support in terms of the modulus of subgrade reaction "k" which is representative of the soil strength throughout the year;

2. estimated strength of the concrete at 90 days, or when pavement is expected to be opened to traffic, if less than 90 days;
3. assumed values for the elastic constants E and μ for the concrete; (Default values of 4,000,000 and .15 are preprogrammed into the design nomographs and the computerized version of the design procedure.)
4. traffic data expressed in terms of the number of loads expected in specified load ranges or equivalent 18 kip single axle loads anticipated over the life of the pavement;
5. weighted averages of thermal gradients to be used for calculation of the curling stresses. (Default values are preprogrammed into the design nomographs and the computerized version of the design procedure.)

The design options which can be varied during the design iterations include:

1. slab thickness;
2. the type of subbase (assumed thickness of 4");
3. type shoulder (asphalt, tied PCC, or extended paving lane);
4. joint spacing and type of load transfer;
5. design reliability level (usually 50, 85 or 95 percent);
6. final service level for the pavement.

The design procedure is implemented through the use of the design worksheet (Figure 12) or a computerized version (ILLJOINT-reference 6) written specifically to calculate the fatigue damage. The service level curve for correlating fatigue damage and cracking is shown in Figure 15.

A. Modulus of Subgrade Reaction

The modulus of subgrade reaction "k" is not a constant value over time, but changes throughout the year. However, since the design thickness is relatively insensitive to the "k" value, each year can be broken down into two increments, namely, the late winter when the ground is partially frozen, and spring when the frost leaves the ground as one increment, and the remaining summer, fall and winter months when the "k" value is likely to be somewhat higher than the other increment. Typical values are assigned for each increment and the fatigue damage determined for each of these increments. If changes in "k" during these two increments are small, then a single value can be assigned for the entire period.

B. Concrete Strength

Three factors are taken into account in determining the strength of the concrete: 1) the concrete's tendency to increase in strength with age; 2) the natural variability in the concrete strength; and 3) the method of testing. These three factors are considered simultaneously when selecting the design strength of the concrete.

Most agencies use the strength of concrete at 90 days for the design purposes. By 90 days, most

pavements will begin to experience design loads, and most of the ultimate strength will have been achieved.

Strength of the concrete is usually expressed in terms of its modulus of rupture. The IDOT normally specifies the strength of the concrete by beams strength at 14 days tested under center point loading. It has been determined, however, that the flexural strength of a beam under third point loading (a more realistic test for design strength) is approximately 85 percent of its flexural strength under center point loading. Thus, the equivalent flexural strength for third point loading can be determined by multiplying the strength determined using center point loading by 0.85.

With Type I cement, IDOT construction data indicate the average 90 day strength of concrete is 1.19 times the 14 day strength. Therefore, the 90 day strength for beams under third point loading is approximately equal to the 14 day strength tested under center point loading ($1.19 \times 0.85 = 1.01$). Thus, if Type I cement is used without accelerators or retardants, the mean 14 day strength of beams tested under center point loading can be used as the mean strength for the concrete for design.

Variations in the strength of concrete can have a significant effect on the performance of the pavement. Under normal construction practices, concrete has a coefficient of variation of from 10 to 12 percent for very high quality control, which can go up to 20 percent or higher for poor quality control. For this design procedure a coefficient of variation of 15 percent is assumed, which represents fair to good quality control. Other values for the coefficient of variation can be used at the option of the designer but such values should be justified.

Variations in the concrete strength are taken into account by selecting a design modulus of rupture as one standard deviation below the mean. Thus with a coefficient of variation of 15 percent, the design modulus of rupture (DMR) can be determined by the equation,

$$DMR_{90} = \text{mean } MR_{90} (1 - CV) = \text{mean } MR_{14-cp} (1 - CV)$$

where MR_{14-cp} is the modulus of rupture at 14 days under center point loading and CV is the coefficient of variation of concrete.

C. Traffic

Since traffic input is one of the primary inputs in the design procedure, the more accurately one can predict the traffic, the more reliable the design procedure. It would be ideal if the traffic could be estimated in terms of the number of loads within narrow load ranges. This would permit a much more reliable design procedure. The computerized and worksheet versions of the design procedure have been formulated to accept traffic in load ranges or as 18-kip equivalent single axle loads (18-kip ESAL).

It is recognized, however, that most traffic data will be available in a format so that the best estimate will probably be the number of equivalent 18-kip single axle loads. Thus the design procedure has been set up in such a way that the

designs can be achieved with only these data for traffic input. It should be noted that using the 18-kip ESAL data the equivalency factor varies with the magnitude of load and is not the same for calculating fatigue damage as was determined from the AASHTO Road Test, which is based on the relative changes in PSI of the pavement per load application.

As does the level of traffic, the lateral distribution of the traffic plays an important role in the calculation of fatigue damage. If a truck were centered on a 12 foot paving lane, the distance D would be approximately 24 inches. When there is a paved shoulder and no lateral obstructions, typical values for D range from 12 to 21 inches, with the most frequent distances ranging from 16 to 18 inches (13). Mean D values of 12 and 18 inches are used for this design procedure.

The equivalent damage ratio (EDR) is selected as a function of the shoulder type and mean D location of the traffic distribution. Each pavement-shoulder arrangements will require different equivalent damage ratios be used for design. The equivalent damage ratios are given in Table A. The design traffic should also be broken down according to the temperature gradients discussed previously. The thermal gradients are automatically included as a part of the curling stress determination in the design procedure.

DESIGN PROCEDURE

The general steps to determine pavement thickness are as follows:

1. Determine the design traffic level and choose the mean distance D (either 12 or 18);
2. Determine the subgrade support;
3. Select the terminal service level, i.e. cracking level, design reliability and the corresponding limiting fatigue damage. This should be related to the level of pavement facility being designed;
4. Select a trial slab thickness;
5. Determine the traffic load stresses and calculate total slab stresses (total slab stresses is the sum of the traffic load stress and curling stress);
6. Calculate the number of allowable load applications;
7. Compute the accumulated fatigue damage;
8. Compare the trial fatigue damage with limiting fatigue damage (Figure 15) and return to step 4 if necessary. (This is best accomplished by preparing service level vs. thickness plots - Figure 16).

A. DESIGN EXAMPLE

The following is provided to show the designer how the nomographs are utilized in design calculations. As an example, a design is illustrated for a PCC two-lane highway with asphalt shoulders. Results are shown for slab thickness from 8, 10 and 12 inches. These thicknesses will provide a range of accumulated fatigue results such that a required slab thickness can be chosen. The slab length is 15 feet. A 90-day PCC design

modulus of rupture of 704 psi is used. This is calculated from a 14-day center point loading strength of 820 psi with a coefficient of variation of 15%. The hand calculation sheet for a thickness 8 inches is shown in Figures 12.

A traffic level of 8.8×10^6 18 kip ESAL is assumed for the entire design period. For the asphalt shoulder case, edge load factors are taken from Table A, a portion of which is reproduced below:

EQUIVALENT DAMAGE RATIOS

D	H	K	Damage Ratio
18"	8	100	.05
		200	.05
		100	.05
	12	200	.05
		100	.06
		200	.06

The EDR's are listed for two k-values (representing different soil conditions during the year) and three temperature gradients (representing different times of the day when the slab is in a curled position). A foundation modulus of 100 psi/in. is used for 4 months of the year and a value of 200 psi/in. is used for the remaining 8 months. The lower value represents spring and early summer when foundation support is weak. The upper value represents fall and winter whose k-values are generally stronger.

To follow the design process using the worksheet, refer to the top line of Figure 12. The traffic is broken down into different times of the year and different times of the day, using the equivalent damage ratios given in the preceding tables:

TRAFFIC BREAKDOWN

Total traffic	Temp gradient	k value	8"	10"	12"
8.8×10^6	Daytime (25%)	100	36667 ¹	36667	44000
		200	73333	73333	88000
		100	58667	58667	70400
	Nighttime (35%)	200	117333	117333	140800
		100	61600	61600	71887
		200	123200	123200	143733

$$^1 (8.8 \times 10^6) (.25) (.33) (.05) = 36667$$

Wheel load stress in a concrete slab with bituminous shoulders is determined from Figure 13. Assuming a thickness "H" of 8 inches and a "k" value of 100 psi/in results in a wheel load stress of 370 psi. (If using a 10 foot PCC tied shoulder with a high load transfer, a reduction in the medium load transfer stress is allowed using the factors in Table B.)

The appropriate R-value for a 15 foot slab length is read from Table C. If a slab length greater than 15 feet is desired, an additional factor, R', is obtained from Table C. This R' factor is added to the 15 foot R factor to get a total R value. For this case, with slab thickness of 8 in. and k value of 100 psi/in., an R value of

1.09 is read. Note that for a slab with zero temperature gradient, curling stresses are zero and the R value is not required. (The R value is an adjustment factor made to the curling stress to properly combine it with the load stress to arrive at the correct total stress.)

Curling stresses produced by temperature gradients are read from Figure 14. The daytime curling stress for an 8 in. slab with a 15 foot slab length with a subgrade k of 100 psi/in. is 114 psi. Note that two scales exist for curling stress. One corresponds to a positive daytime gradient and the other to a negative nighttime gradient.

The total stress that exists in the pavement is calculated by adding the wheel load stress and the curling stress multiplied by the R value. In this case,

$$512.0 \text{ psi} = (388.0 \text{ psi}) + (1.087) \times (114.0 \text{ psi})$$

The number of allowable load applications can be determined by using Equation 1 based on the total stress and the modulus of rupture of the concrete. With a PCC design strength of 533 psi and a total stress value of 512 psi, the number of allowable load applications before cracking is 2.

The cumulative fatigue damage can now be computed using the Minor hypothesis. A cumulative fatigue damage of 1.66×10^4 is determined by dividing the expected load applications (33667) by the allowable load applications (2).

This type of calculation is made for each load range, k-value, and temperature gradient. These values are then summed up to get a total cumulative fatigue. This is shown as 1.83×10^4 at the bottom of the sheet in Figure 12. The same procedure is repeated for each trial thickness. (The computer program ILIJOINT written for the PC computer was used to perform these calculations.)

The fatigue damage for each slab thickness (8,10,12 inches) was plotted against the corresponding service level (from Figure 15) for D = 18 inches at 95% reliability. These are shown in Figure 16. This figure is a plot of service level and pavement thickness showing the benefit in terms of slab thickness of using a 10 foot concrete shoulder with a high degree of load transfer, or a 2 foot extended width driving lane against a bituminous shoulder. On the average the slab thickness can be reduced approximately 2 inches over that required with a bituminous shoulder. The 10 foot concrete shoulder with a medium load transfer across the longitudinal shoulder joint will not offer as much reduction in thickness as the 2 foot extended width driving lane. Due consideration should be given to the type of longitudinal joint, i.e., joint transfer, to be used when designing pavements with the 10 foot tied shoulder.

Figure 17 shows the effect of joint spacing on the service level. The effect is significant in this case.

Conclusions

As a result of recent development of mechanistic analysis tools for rigid pavements, a mechanistic design procedure is suggested. This

procedure allows the effect of shoulder type, concrete strength, and joint spacing on pavement thickness to be considered in a rational manner by the means of calculated fatigue damage. The design thickness can be associated with the level of pavement performance since the fatigue damage is calibrated to actual performance of in service pavements. Environmental factors, which are reflected in the curl stress and the design joint spacing, are also included along with material properties of concrete. A variety of shoulder types are available since shoulder design will have a significant effect upon the design thickness.

REFERENCES

1. Tabatabaie, A.M., and E.J. Barenberg, "Finite-Element Analysis of Jointed or Cracked Concrete Pavements," Transportation Research Record, No. 671, Transportation Research Board, 1978.
2. Thompson, M.R., E.J. Barenberg, A.M. Ioannides, and J.A. Fisher, Development of a Stress-Dependent Finite Element Slab Model," Report to AFOSR on Contract AFOSR-82-0143, May 1983.
3. Ioannides, A.M., M.R. Thompson, and E.J. Barenberg, "The Westergaard Solutions Reconsidered," Transportation Research Record No. 1043, Transportation Research Board, 1985.
4. Darter, M.I., "Design of Zero-Maintenance plain Jointed Concrete Pavement: Vol I-Development of Design Procedures," University of Illinois, Technical Report to Federal Highway Administration, June 1977.
5. Ioannides, M. A., "Analysis of Slabs-on-Grade for a Variety of Loading and Support Conditions," Ph.D. thesis, Department of Civil Engineering, University of Illinois at Urbana-Champaign, 1973.
6. Zollinger, D. G., E. J. Barenberg, Kuberski, M. A., and Risser, R. J., "Manual for the Mechanistic Based Design Procedure of Jointed Concrete Pavements," Illinois Cooperative Highway Research Program, Transportation Research Laboratory, University of Illinois at Urbana-Champaign, March 1988.
7. Westergaard, H. M., "Analysis of Stresses in concrete Pavements due to Variations in Temperature," Proceedings of the Sixth Annual Meeting, Highway Research Board, 1927; also Public Roads, Vol. 8, No 3, May 1927.
8. Bradbury, R. D., Reinforced Concrete Pavements. Wire Reinforcement Institute, Washington, D.C., 1938.
9. "The AASHO Road Test - Report 5 - Pavement Research," Special Report 61E, Highway Research Board, 1962.

10. Miner, M. A. "Cumulative Damage in Fatigue," Transactions, American Society of Mechanical Engineers, Vol. 67, 1945, pp. A159-A164.
11. Ballinger, Craig A., "Effect of Load Variations on the Flexural Fatigue strength of Plain Concrete." Federal Highway Administration Final Report, September 1972.
12. Lloyd, J. P., Lott, J. E., and Kesler, L. E., Fatigue of Concrete, Engineering Experiment Station Bulletin 499, University of Illinois, 1968.
13. Becker, J.M., M.I. Darter, M.B. Snyder, and R.E. Smith, "Concrete Pavement Evaluation System (COPES)," NCHRP Report No. 227, Final Report, NCHRP Project 1-19, 1984.
14. Colley, B. E. and H. A. Humphrey. "Aggregate Interlock at Joints in Concrete Pavements," Highway Research Record No. 189, Highway Research Board, Washington, D.C. 1967.
15. Van Breeman, W., "Experimental Dowell Installations in New Jersey," Proceedings HRB, Vol 34, 1955.
16. Woodstrom, James H., "Concrete Pavement Drainage in California," Presented at the International Seminar on Drainage and Erodibility at the Concrete Slab-Subbase-Shoulder Interface, Paris, March 24-25, 1983.
17. Van Wijk, Adrian, J., "Rigid Pavement Pumping: (1) Subbase Erosion (2) Economic Modeling," Ph.D. thesis, School of Civil Engineering, Purdue University of 1985.
18. Barenberg, E. J., "Effects of Moisture and Drainage on Behavior and Performance of NJDOT Rigid Pavement Systems," Prepared for New Jersey Department of Transportation, Dept. of Civil Engineering, University of Illinois, Urbana, IL, July 1982.

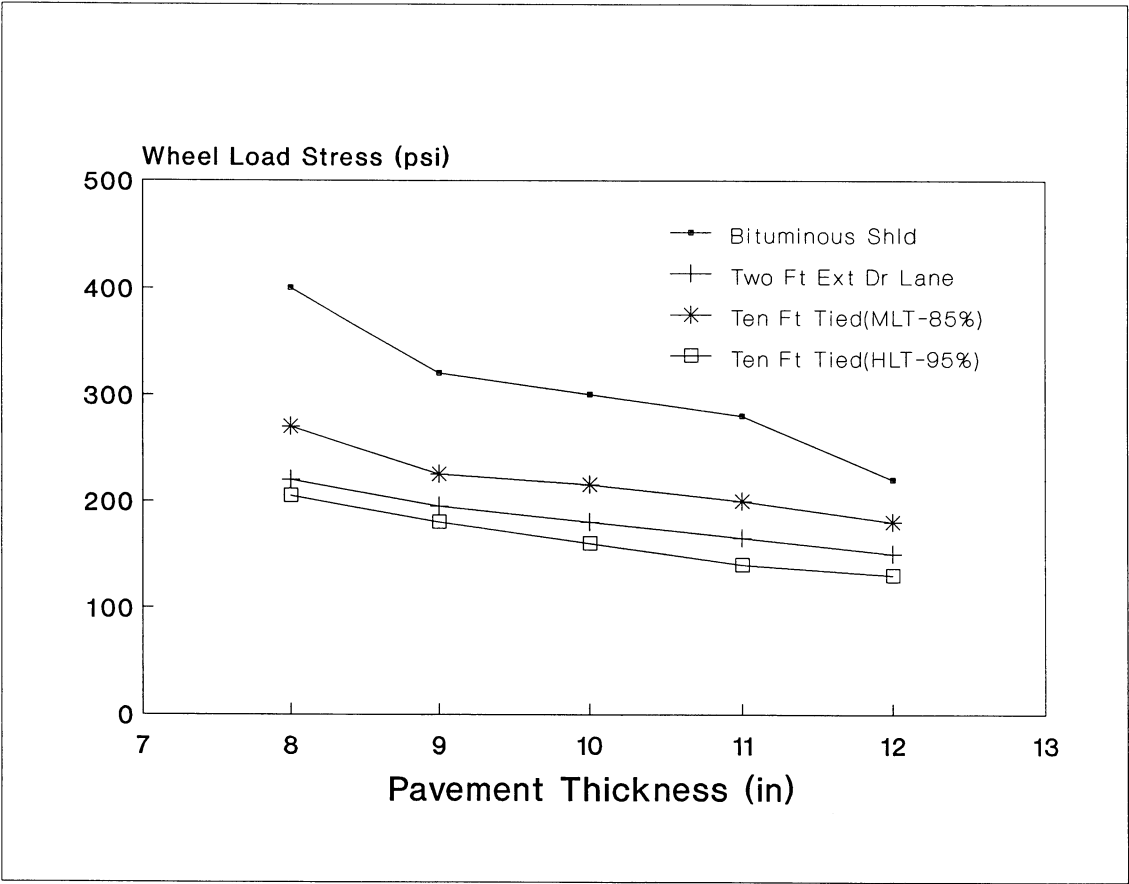


Figure 1 Wheel Load Stress (18^k SAL) vs. Slab Thickness

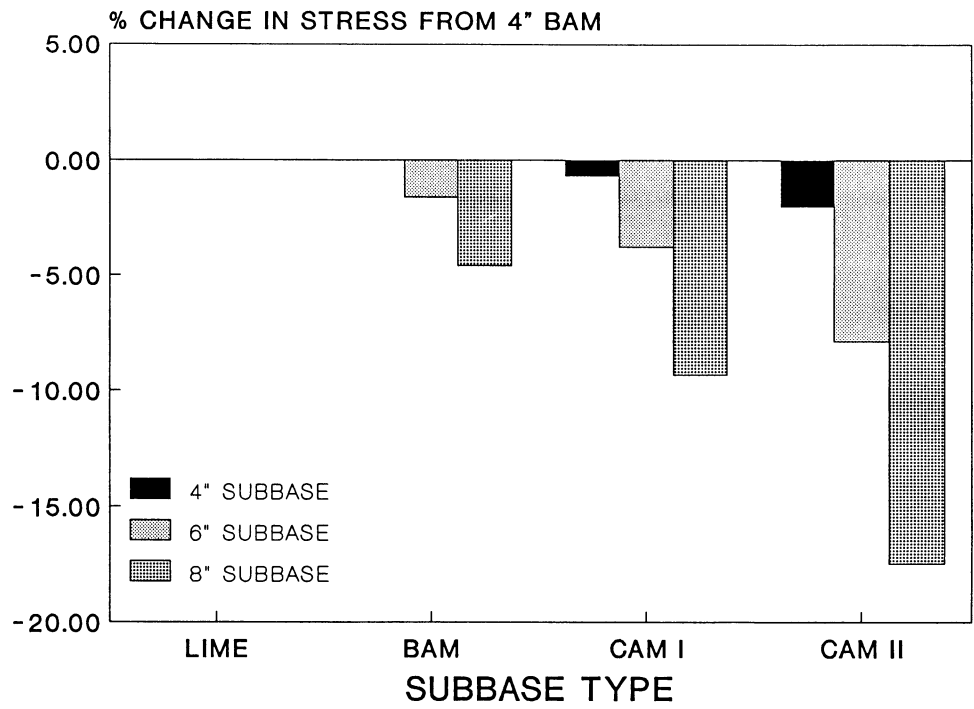


Figure 2 Percent Change in Wheel Load Stress Relative to a 4 in. BAM Subbase (10 in. Slab Constant)

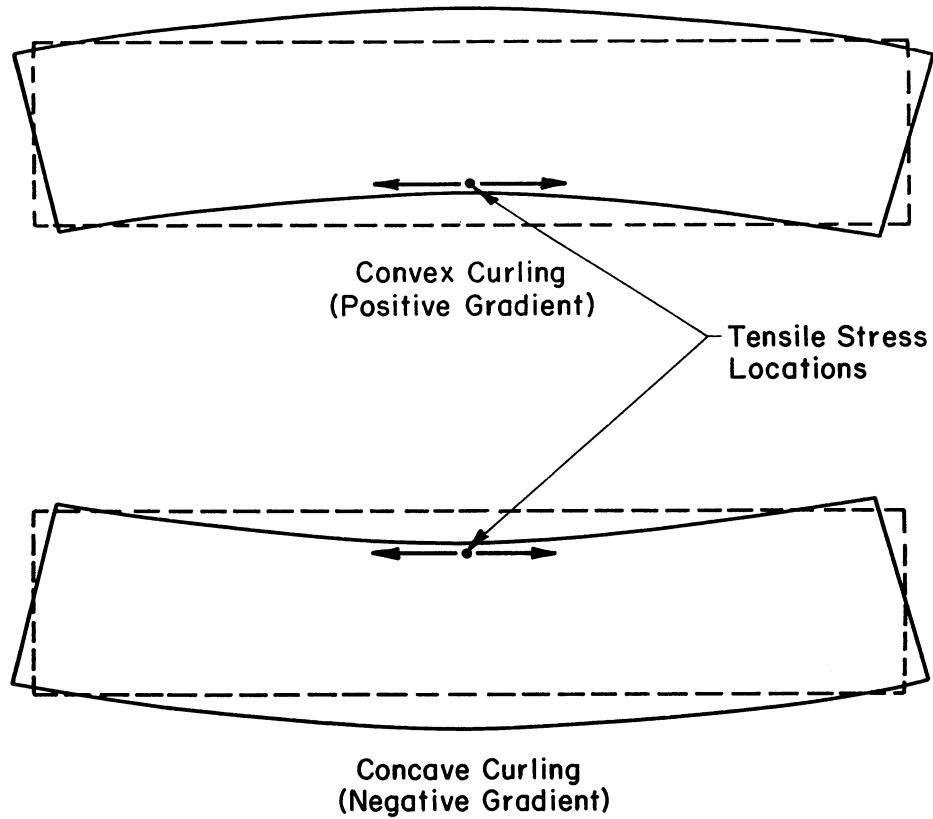


Figure 3 Deflected Slab Shape Under Temperature Gradient

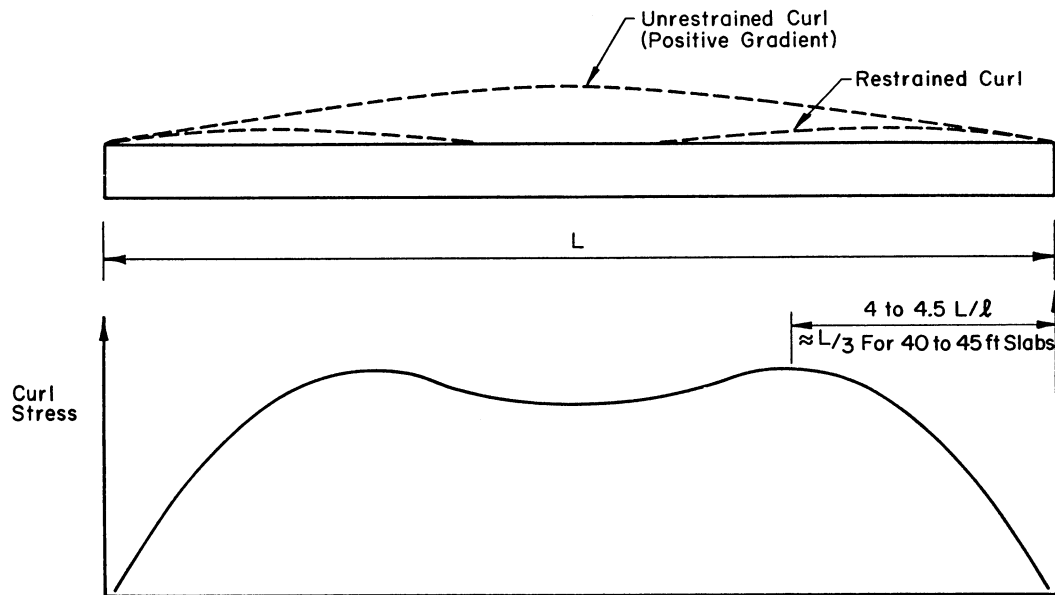


Figure 4 Slab Weight will Prevent Deformation at the Center of the Slab

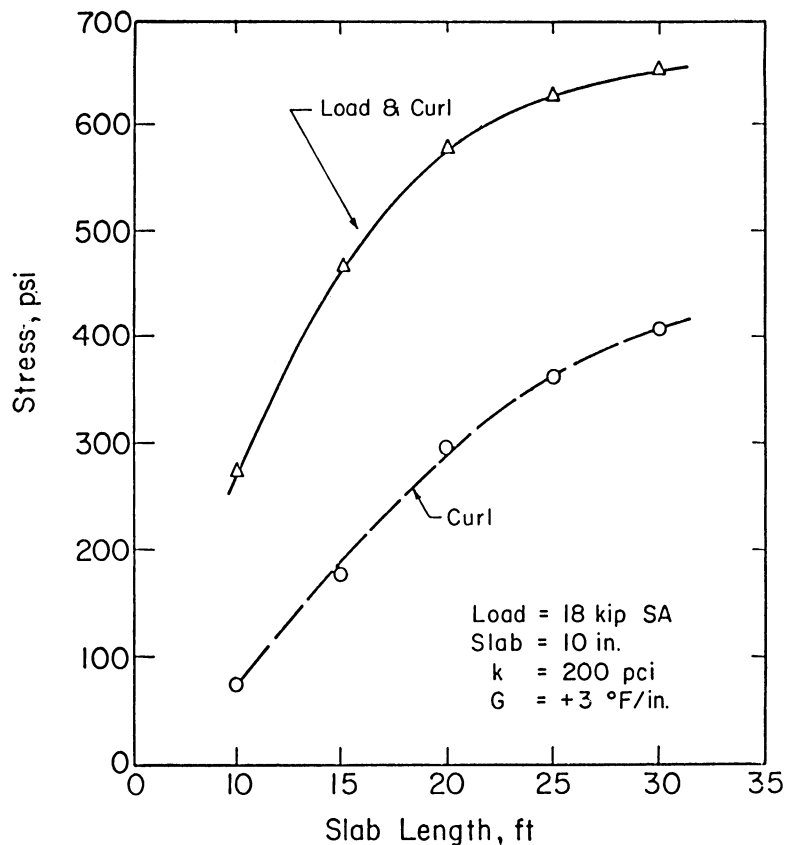
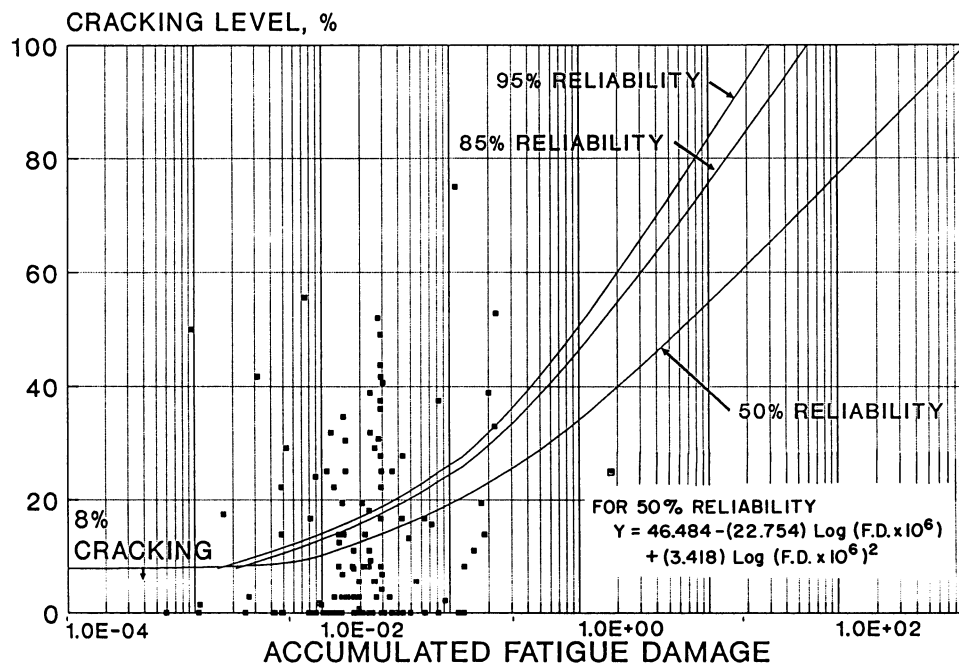


Figure 5 Edge Stress Due to Load and Thermal Gradient for Varying Joint Spacing. (4)

Figure 6 Calibrated Performance Curves Based on Illinois COPES Data ($\bar{D} = 18$ in.)

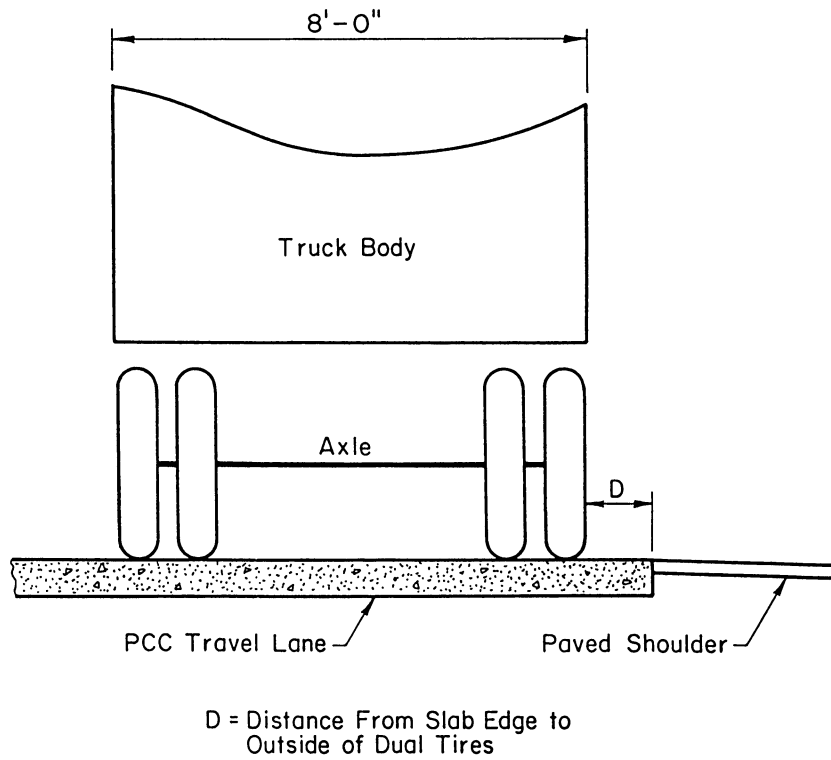


FIGURE 7 ILLUSTRATION OF THE MEAN DISTANCE FROM SLAB EDGE TO OUTSIDE OF DUAL TIRES

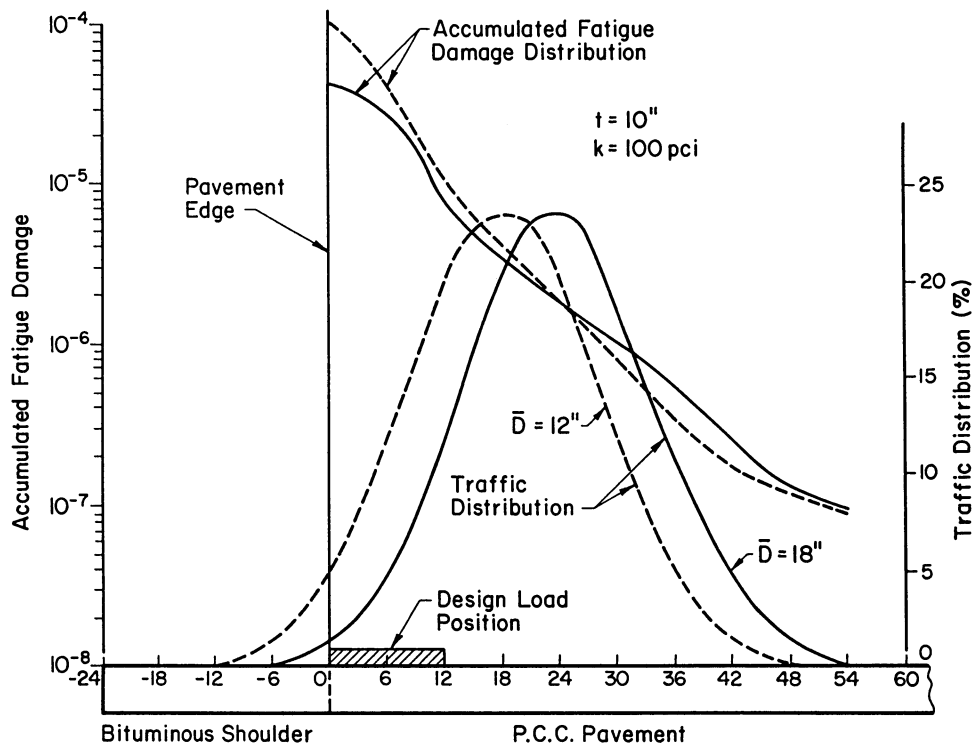
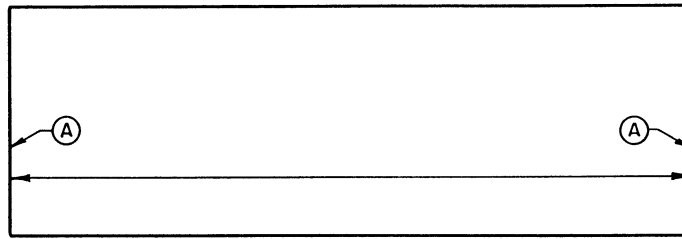
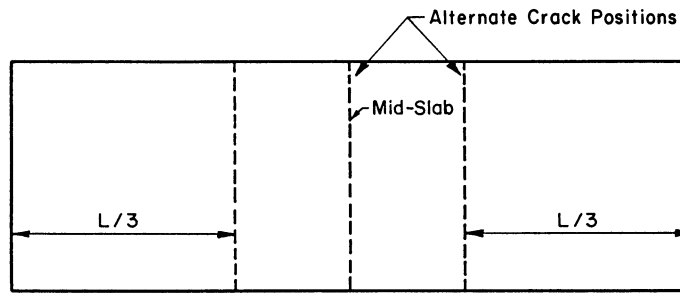


Figure 8 Lateral Distribution Fatigue Damage Effects

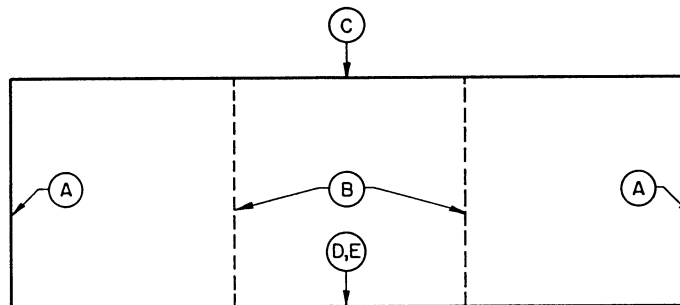


(a) 40' Jointed Slab With No Joints

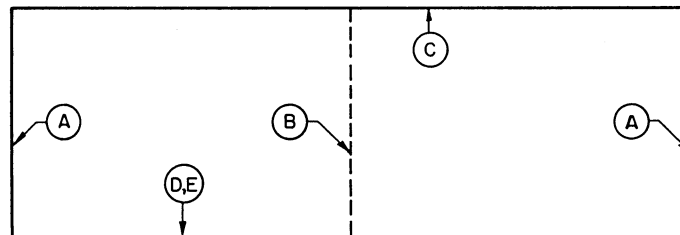


(b) 40' Joint Slab With Most Likely Crack Locations Shown as Dashed Lines

Figure 9 Jointed Slab Layout



(a) 40' Jointed Slab With Two Hinge Joints



(b) 40' Jointed Slab With One Hinge Joint

Figure 10 Joint Spacing Layout with Hinge Joints

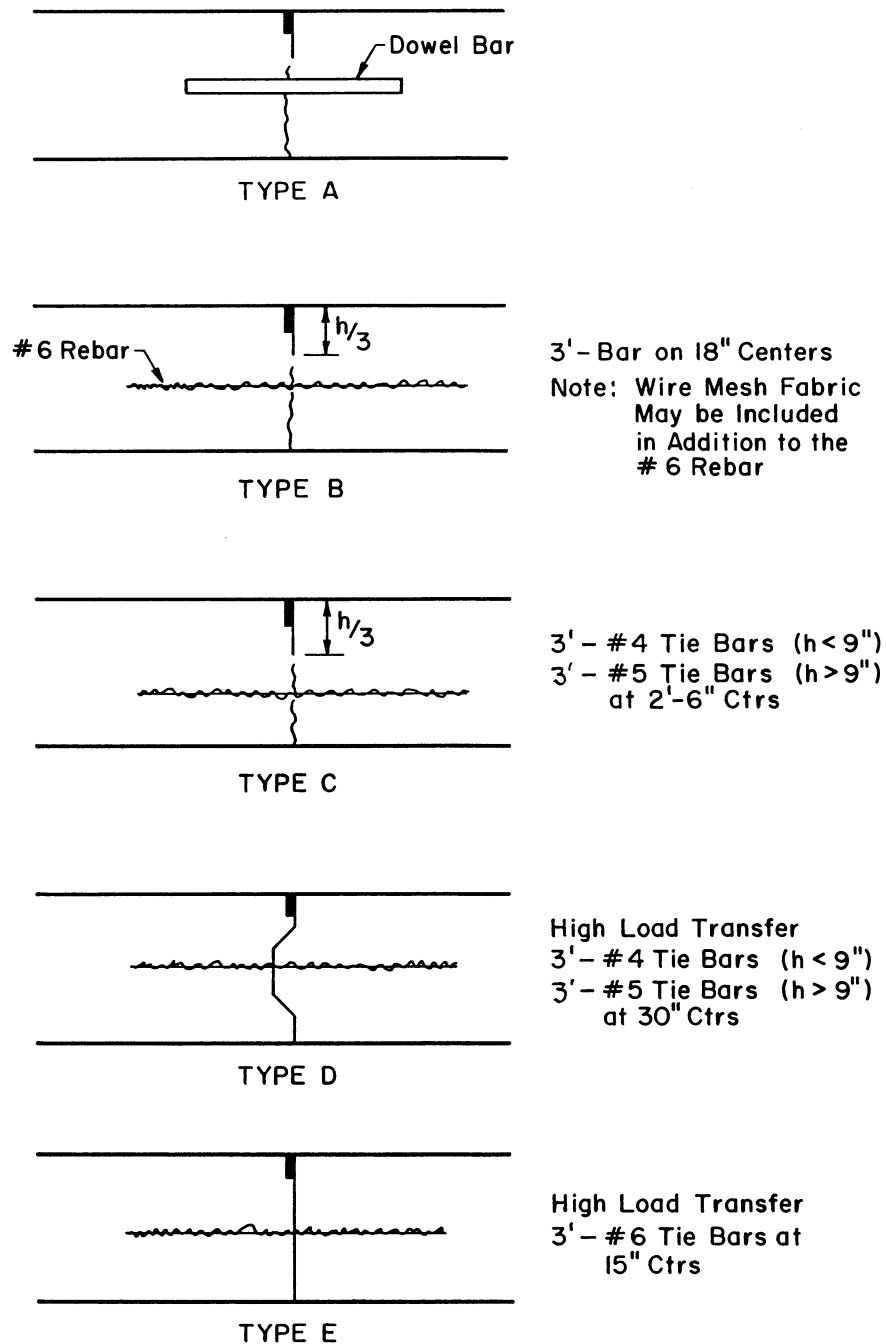


Figure 11 Joint Details

PAVEMENT THICKNESS FATIGUE CALCULATION

PROJECT: FA 404 Galesburg

THICKNESS: 8"

MR₉₀: 553 psiTYPE: Bituminous Shoulder; $\bar{D} = 18"$

	AXLE LOAD GROUP (kips)	k VALUE	EXPECTED LOAD APPLICATIONS	WHEEL LOAD STRESS (psi)	R	CURLING STRESS (psi)	TOTAL STRESS (psi)	ALLOWABLE LOAD APPLICATIONS	FATIGUE DAMAGE
DAYTIME GRADIENT	18	100	33667	388	1.087	114.0	512	2.	1.66×10^4
	18	200	73333	346	1.087	114	470	44	1.65×10^3
ZERO GRADIENT	18	100	58667	388	-	0	388	17961	32.7
	18	200	117333	346	-	0	346	390668	.30
NIGHTTIME GRADIENT	18	100	61600	388	.985	-46	342	523825	.11
	18	200	123200	346	.985	-46	300	1.14×10^7	.010
TOTAL TRAFFIC = 8.8×10^6							TOTAL FATIGUE = 1.83×10^4 DAMAGE		

Figure 12 Pavement Thickness Fatigue Calculation - 8 Inch Thickness

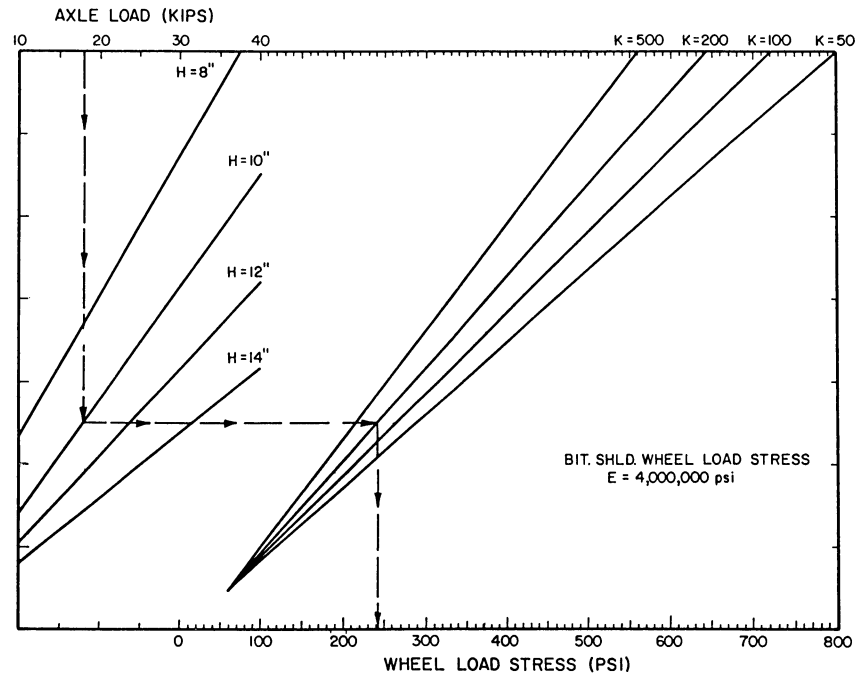


Figure 13 Bituminous Shoulder - Wheel Load Stress - E = 4,000,000 psi

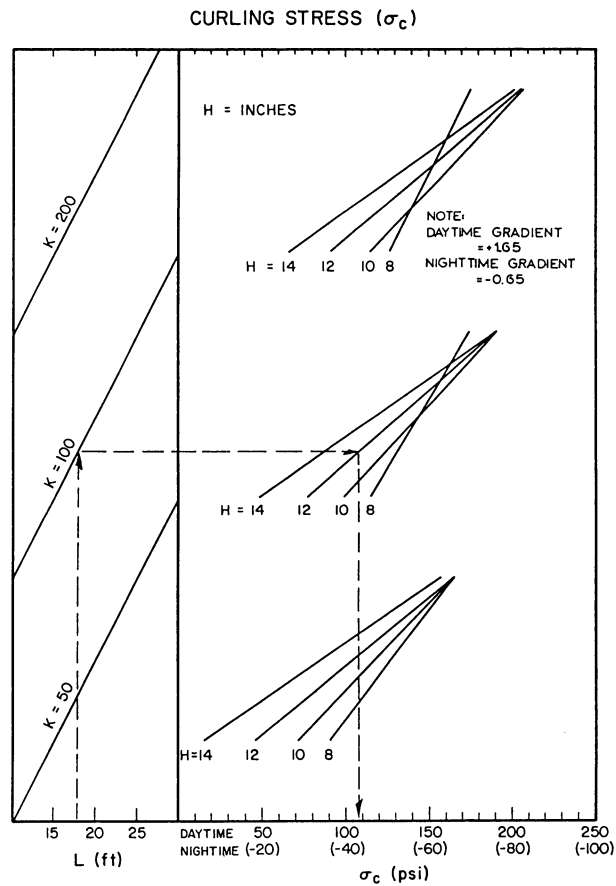


Figure 14 Curling Stress - K = 50, 100, 200

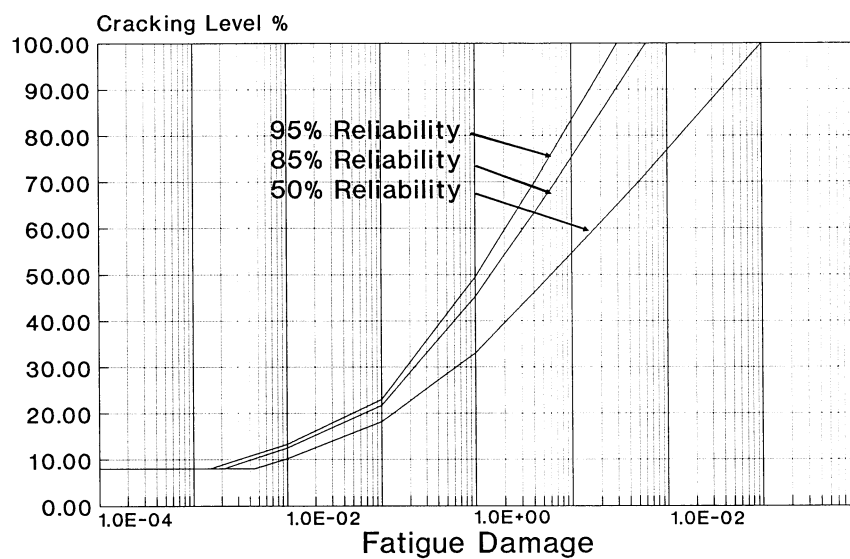


Figure 15 Service Level vs. Fatigue Damage

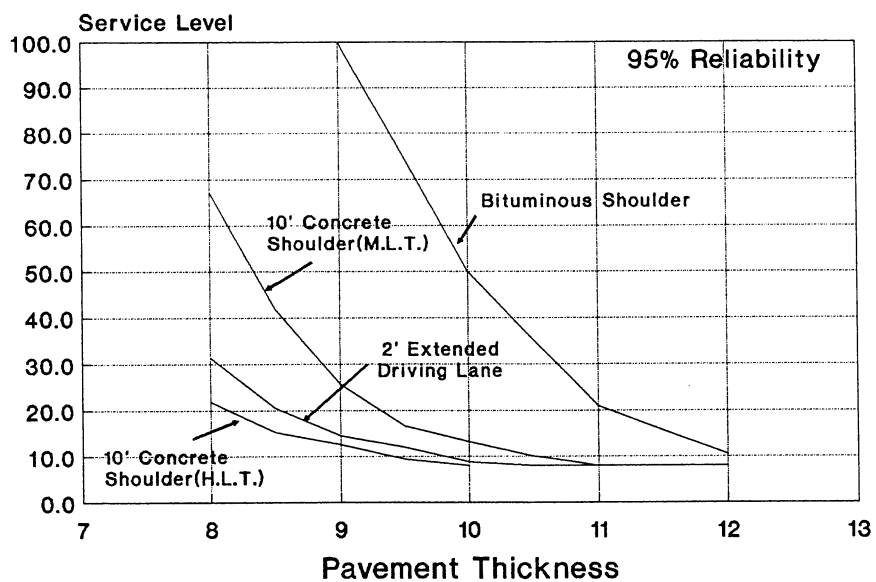


Figure 16 Service Level vs. Pavement Thickness

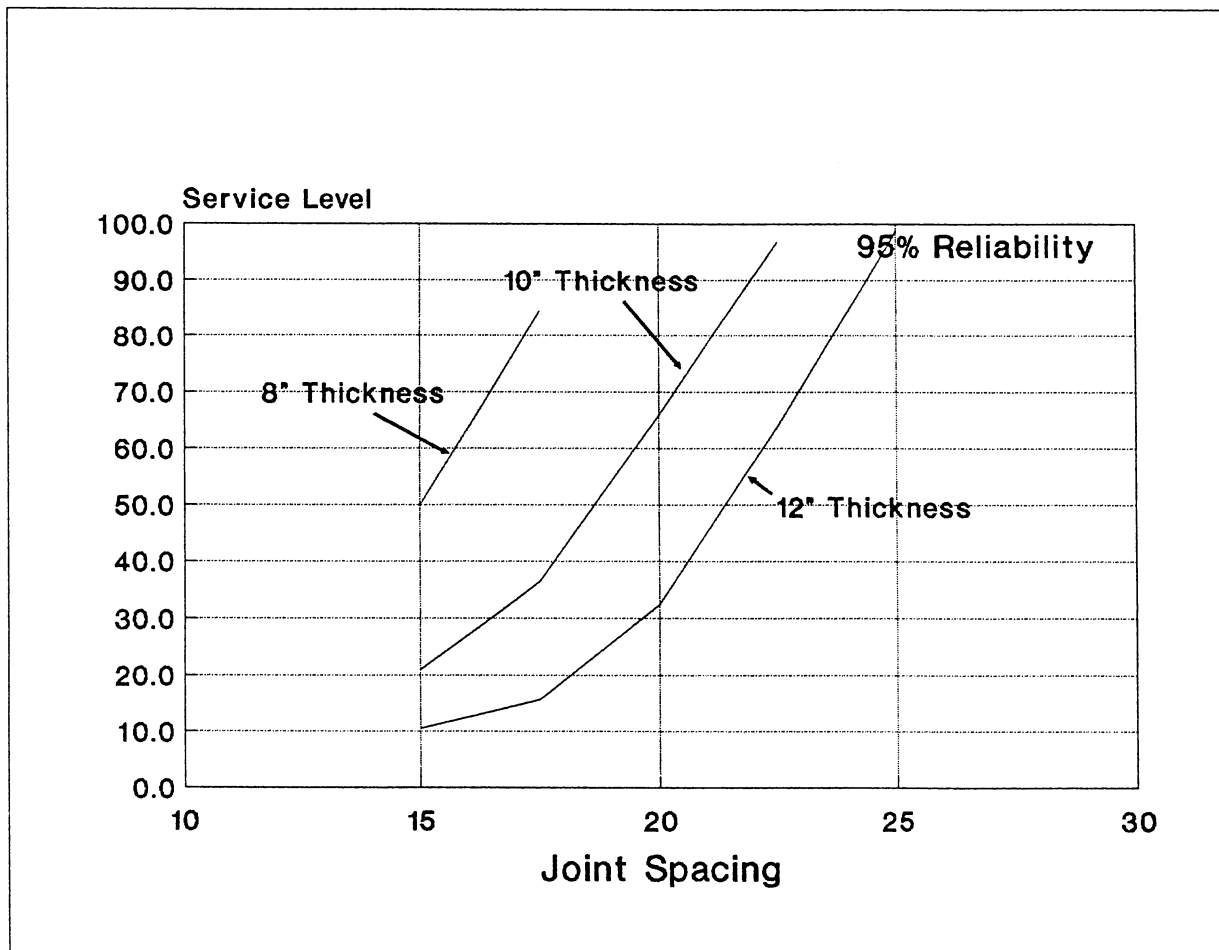


Figure 17 Service Level vs. Joint Spacing

TABLE A
EQUIVALENT DAMAGE RATIOS

Bituminous Shoulders

THICKNESS	$\bar{D}=18"$		$\bar{D}=12"$	
	K=100	K=200	K=100	K=200
8	.05	.05	.12	.12
10	.05	.05	.12	.12
12	.06	.06	.14	.14

10' Concrete Shoulder (MLT)

THICKNESS	$\bar{D}=18"$		$\bar{D}=12"$	
	K=100	K=200	K=100	K=200
8	.07	.06	.18	.17
10	.09	.09	.19	.19
12	.17	.17	.22	.22

10' Concrete Shoulder (HLT)

THICKNESS	$\bar{D}=18"$		$\bar{D}=12"$	
	K=100	K=200	K=100	K=200
8	.12	.13	.24	.18
10	.21	.22	.27	.23
12	.34	.33	.36	.32

Two Foot Extended Driving Lane

THICKNESS	$\bar{D}=18"$		$\bar{D}=12"$	
	K=100	K=200	K=100	K=200
8	.11	.14	.17	.19
10	.17	.20	.24	.25
12	.25	.28	.31	.34

TABLE B

10' CONCRETE SHOULDER ADJUSTMENT FOR HLT E=4,000,000

FOUNDATION MODULUS	THICKNESS			
	8	10	12	14
100	.77	.76	.74	.72
200	.76	.73	.72	.71

TABLE C

R- FACTOR

DAYTIME GRADIENT

FOUNDATION MODULUS	THICKNESS			
	8	10	12	14
100	1.09	1.13	1.16	1.18
200	1.12	1.16	1.21	1.24

NIGHTTIME GRADIENT

FOUNDATION MODULUS	THICKNESS			
	8	10	12	14
100	.98	1.00	1.01	1.02
200	.99	1.03	1.05	1.06

 R^1 - FACTOR
 $L=20'$ ($R=R_{15}+R^1$)

GRADIENT	K=100	K=200
NIGHTTIME	.02	.01
DAYTIME	.02	.04