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Asphalt Concrete Overlay Design Procedure for Portland Cement Concrete Pavements

STEPHEN B. SEEDS, B. F. McCULLOUGH, and R. FRANK CARMICHAEL

ABSTRACT

The development and practical application of a reflection cracking analysis and overlay design procedure, which was developed for the Arkansas State Highway and Transportation Department, are described. The procedure is mechanistically based, but it is calibrated to the performance of experimental overlay sites in Arkansas and Texas. The procedure is incorporated into a computer program (ARKRC-2) for both existing pavement evaluation and overlay design. It considers asphalt concrete overlays and several techniques of reflection cracking control that may accompany overlay placement. These measures include bond breakers, stress-relieving interlayers, undersealing, and increased overlay thickness. The design procedure calls for a program of field measurements of vertical and horizontal slab movements to establish the potential for slab movement after overlay. Differential vertical slab movements are measured at joints (or cracks) by using a light-load deflection device (such as the Dynaflect). Measurements of horizontal slab movement are made over 2 or 3 daily temperature cycles at several existing joints (or cracks) by using a mechanical strain gauge. In the analysis procedure differential vertical slab movements are used to characterize load transfer and predict shear strains that will occur in the overlay under a simulated 18-kip axle load. Horizontal slab movements, on the other hand, are used to predict the maximum daily tensile strains that will be generated in the overlay during different seasons of the year. For both strain criteria, a fatigue-type approach is used to predict how long the overlay will last. A probabilistic distribution is then applied to the horizontal tensile (environmental) strain criteria, such that the overlay design can be based on a minimum tolerable level of reflection cracking over the design life. For joints (or cracked areas) that have problems with poor load transfer and would thus generate excessive overlay shear strains, it is recommended that some type of slab repair or undersealing operation be performed. (The findings of the original study for Arkansas indicated that other control measures such as increased overlay thickness and stress-relieving interlayers are not cost-effective compared with remedying the cause of the poor load transfer problem.) Besides providing a general description of the analytical models and the ARKRC-2 program (which can be adapted to almost any environment in the United States), examples of the overlay design nomographs developed for the specific construction materials and environmental regions found in Arkansas are also presented.

In this paper a design procedure for asphalt concrete overlays of existing rigid pavements is described. The procedure was developed for Arkansas (1) by extending and calibrating the original FHWA-Austin Research Engineers (ARE) procedure (2) based on field measurements and performance observations in Arkansas (by the University of Arkansas and the Arkansas State Highway and Transportation Department) and in Texas (by the Center for Transportation Research at the University of Texas). The primary emphasis of the design procedure is the control of overlay reflection cracking through the examination of the two principle failure mechanisms: temperature-related horizontal slab movements and wheel-load-related differential slab movements at joints.

The primary component of the procedure is a computer program (ARKRC-2) that uses a mechanistic analysis approach to predict the performance of asphalt concrete overlay alternatives that incorporate various crack deterrent measures, including bond breakers, cushion courses or other intermediate layers, undersealing, and increased overlay thickness. A secondary component of the procedure is one in which the computerized portion of the process is replaced by design charts and nomographs. These design charts were developed based on a statistical analysis of the ARKRC-2 program in which (a) a factorial experiment involving the major independent variables was designed, (b) the treatment combinations were generated by using the ARKRC-2 program, and (c) the regression analyses were performed to develop the coefficients for the design equations. These equations and nomographs are capable of accurately considering several of the factors and conditions associated with the design of asphalt concrete overlays in Arkansas. They are also compatible with the AASHTO Pavement Design Guide format, but do have their limitations and constraints (beyond those of the program) that limit their application in other environments.

This paper is organized such that the analysis and design for the two failure mechanisms are considered separately. Within each part a description is provided on field data collection, general input data, the method of analysis, and use of the design charts. Because of the detail that would be required, application of the actual ARKRC-2 program is not described.

OVERLAY DESIGN CONSIDERING TEMPERATURE EFFECTS

In this design procedure the adequacy of a given overlay strategy to withstand reflection cracking is established based on two types of failure criteria: overlay tensile strain and overlay shear strain. Shear strains are basically the result of the potential for differential vertical movements between adjacent slabs underlying the overlay. Tensile strains, on the other hand, are the result of thermal stresses and temperature-related horizontal movements of the underlying slab. Because these two types of distress mechanisms are both associated with the existing concrete pavement, it is possible to estimate the amount of influence they will have on the development of reflection cracking by making some field measurements of concrete movement before placement of the overlay. In this section the design for considering the effects of temperature-related horizontal slab movements on tensile strains and reflection cracking in the overlay is described.

Field Measurements of Slab Movement

In order to predict the effects of cyclic temperature changes, it is necessary to collect measurements of slab movement as a function of air temperature. The recommended procedure for doing this is to install metal reference points on both sides of several joints (or cracks) in the existing portland cement concrete (PCC) pavement and then measure the spacing between these points by using a Berry strain gauge over a range of air temperatures. To avoid some of the other external effects, it is recommended that these measurements be obtained at the rate of five different temperatures per day for a minimum of 2 consecutive days.

The recommended installation procedure to obtain these measurements is to first drill holes on both sides of a joint (crack) and securely glue bolts into these holes to act as reference points. The bolts should have small drilled holes on their heads that function as seats for the Berry strain gauge. Figure 1 shows the placement of these brass bolts. The bolts should be placed out of the wheelpaths (preferably 12 to 18 in. from the pavement edge) to minimize wheel load disturbance.

Although it is important to obtain a good sample of horizontal movement data from several joints (or cracks) in the existing PCC pavement, it is not an easy or safe process because of the need for traffic control. Consequently, it is up to the user or highway engineer to determine the number of joints (or cracks) that should be measured. It should be recognized, however, that the procedure calls for the joint (crack) movement to occur over a drop in air temperature, and the more locations that are measured, the more likely it is that joints (or cracks) with a high reflection cracking potential will be considered. For continuously reinforced concrete pavements (CRCPs), the measurements must be made in areas that exhibit the average crack spacing for the overlay design section.

Figure 2 shows a sample form for collecting the horizontal movement data from a single joint (crack). The grid at the bottom of the figure is provided to allow the user to plot the data after recording it. These plots will be used later as an aid in selecting design movement data.

General Input Data

In addition to the field data needed to characterize temperature-related horizontal slab movements, there are a number of other inputs that would be considered in a complete or comprehensive ARKRC-2 evaluation. These other inputs (which are too numerous to describe in detail) are summarized as follows:

1. Existing pavement characteristics, which include pavement type, joint or average crack spacing, slab thickness, concrete creep modulus (i.e., elastic modulus under creep loading conditions), thermal coefficient, and unit weight;
2. Reinforcement characteristics, which include longitudinal bar diameter and spacing, elastic modulus, thermal coefficient, and bonding stress;
3. Overlay characteristics, which include overlay thickness, creep modulus, thermal coefficient, unit weight, and bonding stress;
4. Characteristics of control methods considered, which include bond-breaker width, intermediate (cushion course) layer thickness, creep modulus, thermal coefficient, and unit weight; and
5. Environment, which refers to the frequency distribution of critical maximum daily temperature drops during the year.

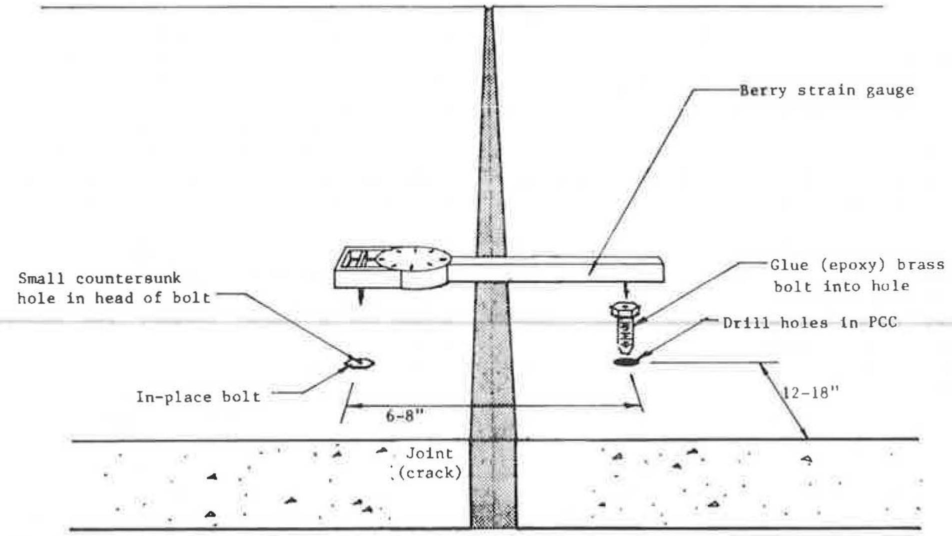
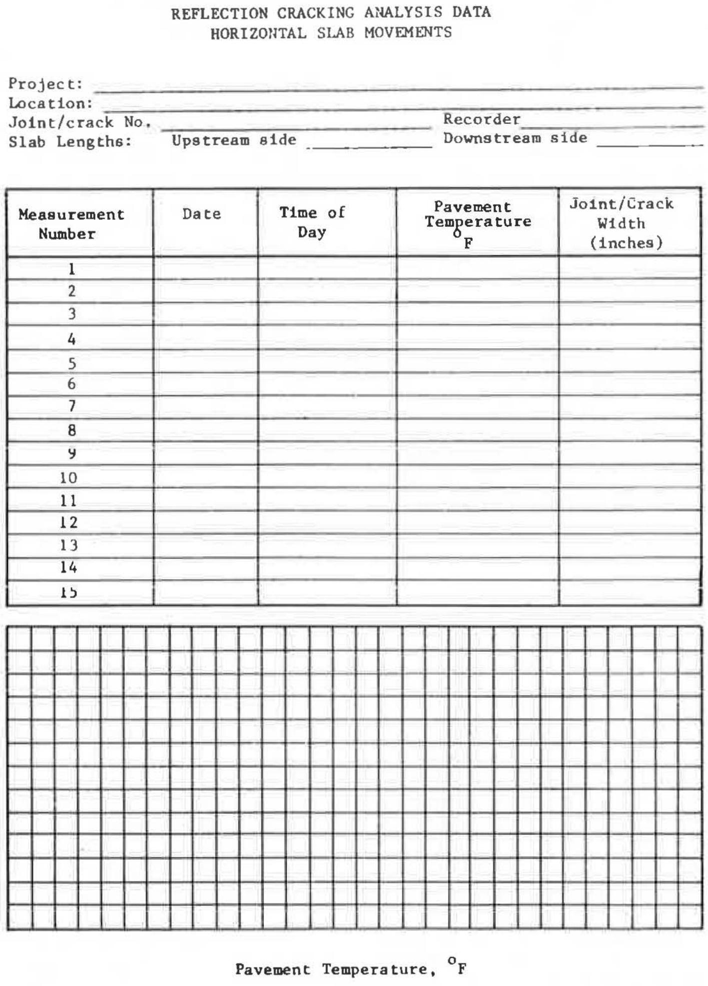


FIGURE 1 Placement of brass bolts for measurement of horizontal slab movement.



REFLECTION CRACKING ANALYSIS DATA
HORIZONTAL SLAB MOVEMENTS

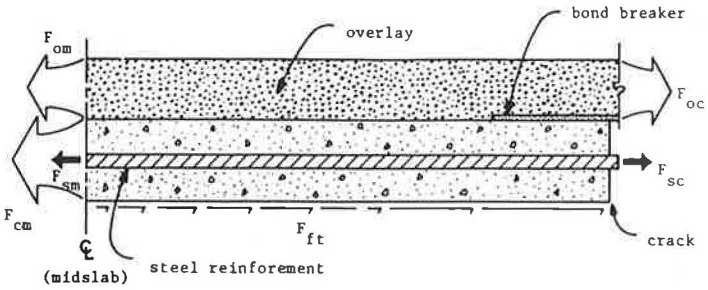
Project: ____________________
Location: ____________________
Joint/crack No. __________ Recorder __________
Slab Lengths: Upstream side __________ Downstream side __________

Measurement Number	Date	Time of Day	Pavement Temperature °F	Joint/Crack Width (inches)
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				

FIGURE 2 Sample form for collecting horizontal movement data.

METHOD OF ANALYSIS

The basic concept behind the method of analysis for temperature-related slab movements is to apply the mechanics of force equilibrium in a pavement structure so that critical tensile strains in the overlay (above a joint or crack) can be computed. These strains are determined for the different critical temperature conditions experienced by the pavement during a yearly cycle. A fatigue model is used to assess damage during each critical period. Miner's linear damage hypothesis is then used to accumulate



F_{om} = Force in overlay at midslab.

F_{sm} = Force in steel at midslab.

F_{cm} = Force in concrete at midslab.

F_{ft} = Slab-base friction.

F_{oc} = Force in overlay at crack (joint).

F_{sc} = Force in steel at crack.

FIGURE 3 Illustration of the force balancing method used to achieve equilibrium in the pavement structure after overlay for the design temperature drop.

the damage during these critical periods and predict the number of years to a certain level of reflective cracking.

More specifically, the field measurements of slab movement are first used to characterize the slab-based friction relationship. This relationship is then adjusted for conditions after overlay, and an iterative process is applied until equilibrium between all forces that act in the pavement structure (at a given design temperature) is achieved. A free body diagram of this is shown in Figure 3. The force in the overlay is ultimately translated into an overlay tensile strain by using its thickness and creep modulus. The bond breaker shown has the effect of increasing the length over which the overlay can absorb slab movements and thus reduces the maximum tensile strain. A cushion course, which is not shown in the figure, can also be considered and has the effect of absorbing a significant amount of strain (due to slab movement) before it reaches the overlay.

Because it is recognized that the tensile strains that induce reflection cracking come about as the result of both direct thermal stresses and the temperature-drop-related movements of the underlying slab, and because the temperature variations are cyclic in nature, the reflection cracking that develops in the overlay must be attributed to fatigue or the accumulation of damage brought about by cyclic loading. Therefore, it was considered essential that the fatigue damage concept be incorporated into the ARKRC-2 analysis and design procedure. The following is the fatigue equation that was developed on the basis of a calibration of observed overlay performance in Arkansas and Texas:

$$N_T = a_1(\varepsilon_T)^{a_2} \qquad (1)$$

where

N_T = average number of fixed strain cycles needed to develop a reflection crack at a given location,
ε_T = asphalt concrete overlay tensile strain for a given critical temperature drop,
a_2 = -3.70,
a_1 = 8.072×10^{-4} $(\mathrm{EOV})^{-1.118}$, and
EOV = asphalt concrete overlay creep modulus (psi).

The consideration of fatigue for a constant cyclic loading condition is basically simple. A small complication is introduced, however, when the effects of a variable cyclic load (such as that resulting from varying low temperature drops) are considered. This consideration of variable load effects requires the assumption that Miner's linear damage hypothesis is applicable to the analysis of fatigue in flexible overlays. This is not a bad assumption and has been used in several other problems that deal with the analysis and design of highway pavements.

Information on the distribution of daily temperature drops for Arkansas was obtained from the National Climatic Center (NCC). This information was collected from a 7-year period (1974 through 1980) for both the maximum daily temperature drop and the difference between 50° F and the minimum daily temperature. The latter data were obtained because a study conducted at the Texas Transportation Institute (3) indicated that the primary temperature-related damage suffered by asphalt concrete occurs when the temperature is less than 50° F. The results of the fatigue equation development verified this observation for conditions in Arkansas; therefore 50° F was selected as a reference temperature for calculating the overlay tensile strains. Inspection of the Arkansas temperature data from NCC indicates that the differences between 50° F and the daily temperature are divided into 10-degree frequency ranges (classes) that identify the average number of days during the year on which the temperature drops a certain magnitude less than 50° F. The total number of days from each range (class) for a given region is never equal to the total number of days in a year (365) because days on which the temperature stays higher than 50° F are not counted. The seven temperature drops and corresponding minimum temperature frequency ranges (classes) considered are given in Table 1.

The average temperature drops (less than 50° F) are used by the program to estimate the corresponding overlay tensile strains. After these tensile

TABLE 1 Temperature Drops and Corresponding Minimum Temperature Frequency Ranges

No.	Range of Temperature Drop (°F)	Range of Minimum Temperature (°F)	Avg Temperature Drop Below 50° F
1	1 to 10	49 to 40	5
2	11 to 20	39 to 30	15
3	21 to 30	29 to 20	25
4	31 to 40	19 to 10	35
5	41 to 50	9 to 0	45
6	51 to 60	−1 to −10	55
7	61 to 70	−11 to −20	65

strains $(\varepsilon_T)_i$ are determined for each average temperature drop, the fatigue equation is used to estimate the allowable number of cycles $[(N_T)_i]$ of a given strain the overlay can carry before it cracks. Next, the incremental damage (d_i) accrued each year by each given strain level is determined by using the following equation:

$$d_i = n_i/(N_T)_i \qquad (2)$$

where n_i is the average number of days during the year on which the overlay is subjected to a given strain level $(\varepsilon_T)_i$. Because each strain level corresponds to a particular average temperature drop, n_i is determined from the temperature distribution data.

Next, the yearly damage due to each individual strain level is accumulated according to Miner's hypothesis:

$$D = \sum_{i=1}^{7} d_i = \sum_{i=1}^{7} n_i/(N_T)_i \qquad (3)$$

where D represents the total damage experienced by the overlay during the course of 1 year.

Because by definition "failure" occurs when D is equal to 1.0, the number of years (Y_T) to failure of the overlay can finally be determined by using the following simple equation:

$$Y_T = 1.0/D \qquad (4)$$

It is important to note that because the fatigue equation represents an average number of cycles to the development of a reflective crack at a given location, Y_T represents the number of years to a reflection cracking level of 50 percent. In the next section on use of the design charts, an explanation is given on how Y_T can be adjusted for a different reflection cracking level.

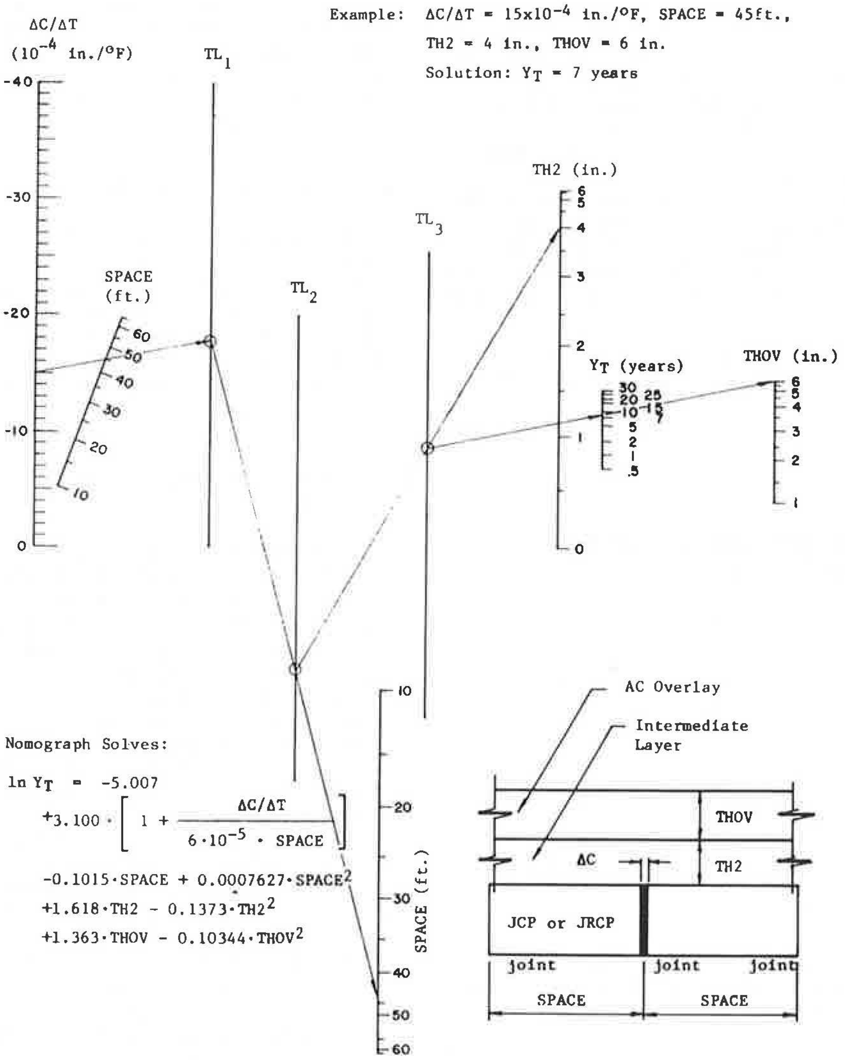
Use of Design Charts (Tensile Strain Criteria)

As mentioned previously, the asphalt concrete overlay design charts for the consideration of tensile strain criteria were developed by using a designed statistical experimental analysis of the ARKRC-2 computer program. Because it was necessary to limit the number of factors considered in the experiment, the resulting design charts do have certain constraints and limitations that pertain to material properties, construction methods, and climate (environment). Figure 4 shows the design chart recommended for overlays on existing jointed concrete pavements in one of Arkansas' five climatic regions. Similar nomographs were developed for other climatic regions and for existing CRCPs as well. (Figure 5 identifies Arkansas' five climatic regions). A discussion of the selection of inputs for the nomographs follows:

1. For each joint (or crack) measured and recorded in the form shown in Figure 2, the user should determine the slope $(\Delta/\Delta T)$ of the best-fit straight line through the data. On the basis of the inspection of the slope values for each line, the user should select a data set or series of data sets for use in analyzing the potential for reflection cracking in the section characterized by the data set(s). This means that, for some overlay projects, it may be necessary to identify and design different overlay sections. In selecting these sections, the user should recognize that those that have the highest slope values will have the greatest potential for reflection cracking (at least from the standpoint of tensile strain). The user should note too that the slope value is the most important characteristic of the data and that it is not necessary to separate sections that have approximately the same slope but different intercepts. Also, because of the inverse relationship between joint (crack) width and temperature, $\Delta C/\Delta T$ should always have a negative value.
2. SPACE defines the spacing between the joints of a jointed pavement or the average spacing between the cracks of a continuous pavement. If the existing pavement is CRCP, then the average crack spacing can be determined by counting the number of cracks in a section of the highway of known length and dividing the section length by the number of cracks. It is important to note that this information is used in conjunction with the horizontal movement data that should have been recorded from areas that exhibited the average joint or crack spacing.
3. THOV defines the thickness (in inches) of the asphalt concrete overlay and represents one of the factors that can be varied in the selection of an adequate design for minimizing reflection cracking. THOV consists of the combined thickness of all binder and surface courses that are considered to increase the load-carrying capacity of the pavement structure. This variable should not include the thickness of any intermediate or strain-absorbing layers (such as an open-graded base course).
4. TH2 is the variable that defines the thickness (in inches) of the intermediate layer that will be placed before the overlay (TH2 equals zero if there is no intermediate layer). An intermediate layer represents a material of certain thickness placed before the overlay to help minimize reflection cracking brought about by underlying slab movements. The layer is different from a bond breaker layer in that it is designed to internally absorb some of the underlying slab movements before they reach the overlay layers. It is not effective in reducing reflection cracking brought about by poor load transfer across joints or cracks.

In this design procedure TH2 can have a large effect on the critical tensile strain developed in the asphalt concrete overlay, particularly if the creep modulus of the layer is low. The strain-absorbing open-graded course used in Arkansas is such a material, but it does have its thickness limits. It can not be less than 3 in. because some of the aggregate particles are as large as 2.5 in. Also, because of possible rutting and compaction problems, the open-graded course thickness should not be greater than 5 or 6 in. Consequently, if the user intends to use some other type of intermediate layer, care should be taken to ensure that its possible thickness limits are considered.

After all the necessary data have been obtained, the following simple design chart procedure may be used to arrive at a suitable asphalt concrete overlay design alternative that considers the temperature effects on critical tensile strains. (It will then be necessary to check this design alternatively



Example: ΔC/ΔT = 15×10^{-4} in./°F, SPACE = 45ft., TH2 = 4 in., THOV = 6 in.

Solution: Y_T = 7 years

Nomograph Solves:

$$\ln Y_T = -5.007 + 3.100\cdot\left[1 + \frac{\Delta C/\Delta T}{6\cdot10^{-5}\cdot \text{SPACE}}\right] - 0.1015\cdot\text{SPACE} + 0.0007627\cdot\text{SPACE}^2 + 1.618\cdot\text{TH2} - 0.1373\cdot\text{TH2}^2 + 1.363\cdot\text{THOV} - 0.10344\cdot\text{THOV}^2$$

FIGURE 4 Asphalt concrete overlay design nomograph for jointed pavements in Arkansas Region B. (Caution: Be aware of restrictions on use of this nomograph.)

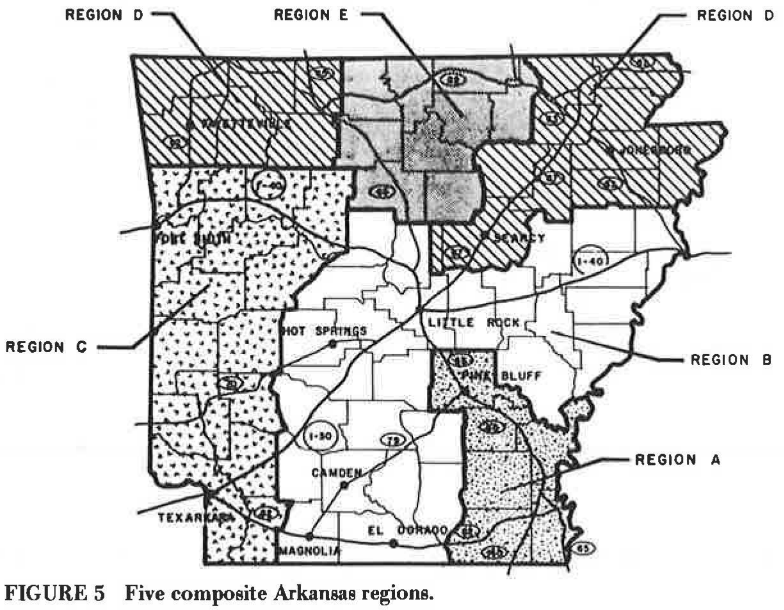


FIGURE 5 Five composite Arkansas regions.

by using the shear strain criteria discussed in the next section).

First, the appropriate nomograph is selected based on the pavement type and region considered. Second, different overlay and intermediate layer thickness combinations (THOV and TH2) are tried until an optimum design alternative for tensile strain criteria is reached.

Finally, if the user is interested in either using a different failure criteria (other than 50 percent reflection cracking) or estimating when different levels of reflection cracking will be reached (based on tensile strain criteria), the following procedure may be applied:

1. Select the level of reflection cracking considered as a limit. This will range anywhere from 1 to 99 percent.
2. Use the data in Table 2 to determine the z-value that corresponds to the selected reflection cracking level.

TABLE 2 z-Values Corresponding to Different Levels of Reflection Cracking

Reflection Cracking (%)	z-Value	Reflection Cracking	z-Value
1	−2.330	55	0.126
5	−1.645	60	0.253
10	−1.282	65	0.385
15	−1.037	70	0.524
20	−0.841	75	0.674
25	−0.674	80	0.841
30	−0.524	85	1.037
35	−0.385	90	1.282
40	−0.253	95	1.645
45	−0.126	99	2.330
50	0.000		

3. Solve for the number of years (Y) that corresponds to the desired level of reflection cracking by using the following formula:

$$Y = (1.585)^{z} \times Y_{50} \qquad (5)$$

where Y_{50} is the number of years before 50 percent reflection cracking is reached (as determined from nomographs), and z is the standard normal variate (from Table 2). It should be noted that the accuracy of this prediction is decreased for very high or very low levels of reflection cracking.

OVERLAY DESIGN CONSIDERING WHEEL LOAD EFFECTS

This part of the asphalt concrete overlay design procedure is used to check the adequacy of the design (developed in the first part) for the effects of wheel load on overlay shear strain. As in the first part, the description of this model is provided in four segments: field data collection, general input data, method of analysis, and use of the design charts.

Field Measurements of Slab Deflection and Load Transfer

Because overlay shear stresses and strains develop primarily as a result of differential vertical movements at joints (or cracks) between adjacent slabs, it is important that some field measurements be made before the overlay is placed to characterize this distress mechanism. The best way to do this is as follows: for a number of joints (or cracks) within a given design section, load one side of the joint and measure the deflection on both the loaded and unloaded sides. A light load is desirable so that the differential deflections measured will approximate those after placing the overlay. The Dynaflect is well suited for this measurement and was recommended for use in Arkansas.

Figure 6 shows the recommended position of the Dynaflect and its geophones within the lane and with respect to the joint or crack. Note that the deflection measurements are taken in the outside wheelpath of the outside lane. Note also that the load wheels and geophone 1 are located on the upstream side of the joint, whereas geophone 2 must be detached from the mounting bar and placed on the downstream side

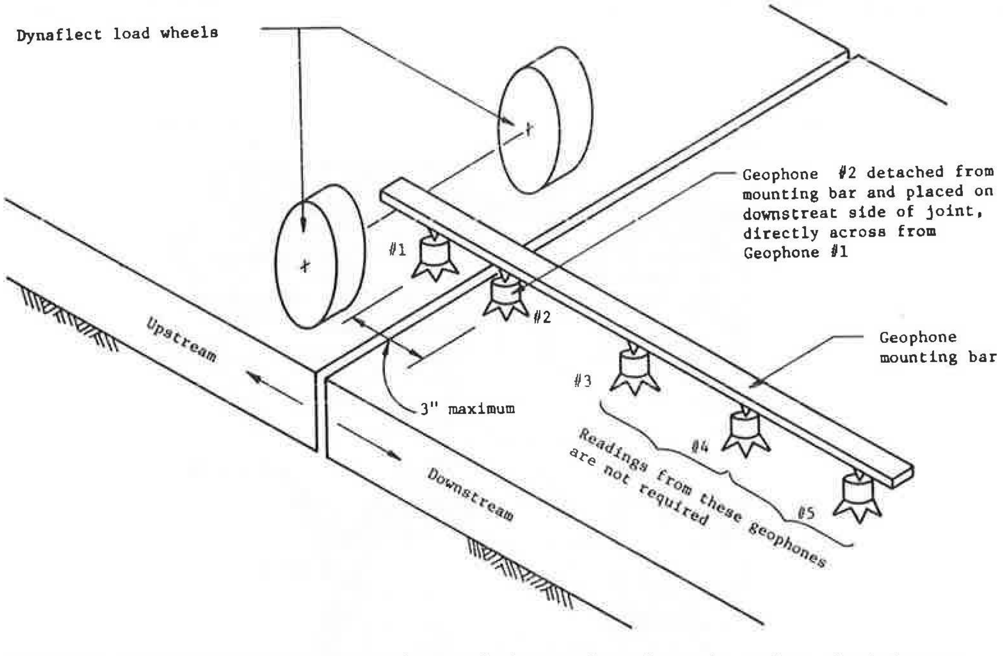


FIGURE 6 Required positioning of Dynaflect load wheels and geophones for load transfer deflection measurements.

of the joint, directly across from geophone 1. Readings from the other geophones may be recorded, but are not required. Henceforth, the deflections from geophones 1 and 2 (when in this configuration) will be designated as w_ℓ (loaded side) and w_u (unloaded side), respectively.

It is recommended that the deflections be obtained during a period representative of the base support conditions after overlay. In other words, measurements should not be made during spring thaw or after a significant rainfall because these saturated conditions are not representative of those after overlay. Late spring, summer, and autumn are probably the best times to obtain representative deflection measurements.

In order to achieve good reliability of the results, it is also important to obtain a good sample of deflection measurements. The number of measurements recommended is dependent on the spacing between the joints (or cracks) and the possibility of the use of some type of undersealant to improve poor load transfer areas.

For the case of jointed pavements [jointed concrete pavement (JCP) and jointed reinforced concrete pavement (JRCP)], it is desirable to obtain measurements at every construction joint. This is especially true if an undersealant is being considered, because certain criteria will be provided later for the selection of which joints to underseal. If an undersealant is not considered and the joint spacing is less than 25 ft, it is probably adequate to obtain measurements at every other joint, so long as there are not any apparent problems with joint pumping.

For the case of CRCP, it is recommended that deflection measurements be obtained for a series of three to five cracks at intervals of approximately 200 ft. Intervals of 100 ft are recommended if an undersealant is to be considered in areas where pumping is observed.

After the data have been recorded, processing should begin by computing the deflection factor (F_W) for each joint (or crack) by using the following equation:

$$F_W = (w_\ell - w_u)/(w_\ell + w_u) \qquad (6)$$

where w_ℓ is the deflection on the loaded side of the joint, and w_u is the deflection on the unloaded side. This data reduction is probably best accomplished with the aid of a computer. After the data are reduced, it is then useful to prepare a longitudinal profile plot of F_W versus distance along the roadway for later analysis.

General Input Data

Besides the field measurements of slab deflection, there are some inputs required for the overlay shear strain analysis:

1. Overlay characteristics, which include dynamic modulus and the combined thickness of any binder and surface (wearing) courses;
2. Intermediate layer characteristics, which include the dynamic modulus and thickness of any type of cushion course or stress-relieving layer placed before the overlay; and
3. Traffic, which refers to the number of 18-kip equivalent single-axle loads that can be expected over the design period.

Method of Analysis

The design for shear strain in an asphalt concrete overlay is based on a theoretical analysis of the Dynaflect deflection measurements made on the slab before placement of the overlay (field measurement program). The difference in deflection across a joint or crack is indicative of the load transfer and therefore the shear forces that will be carried by the overlay. Figure 7 shows the Dynaflect load and geophone configuration used to give the loaded and unloaded deflection values (w_ℓ and w_u). Figure 8 shows how these deflection values make it possible to estimate the amount of shear force (V_O) that will be carried by the overlay layers. The deflections w_ℓ and w_u on either side of a joint due to a load P (Figure 8a) can be simulated by two forces (P_1 and P_2) acting separately (Figure 8b). From slab (or beam) theory, the magnitude of a slab's deflection is directly proportional to the applied load; therefore

$$P_1/P_2 = w_\ell/w_u \qquad (7)$$

Because the total force that causes the deflection on both sides (P) is equal to $P_1 + P_2$, and be-

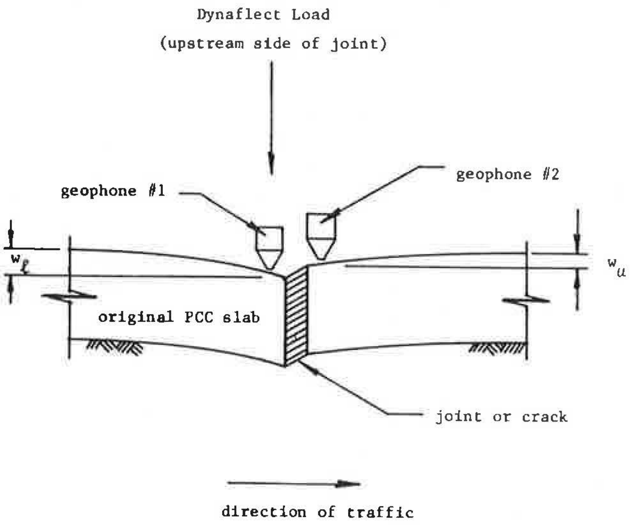
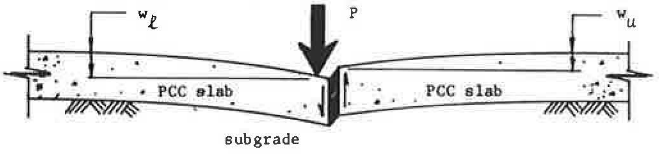
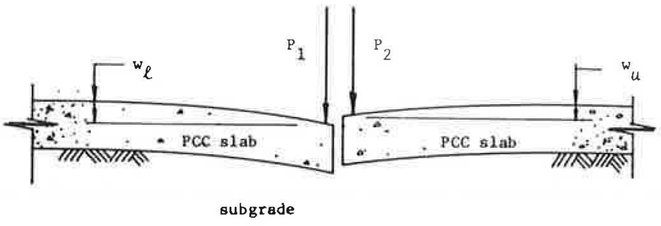


FIGURE 7 Illustration of Dynaflect deflection load and geophone configuration for determining required deflection values.



a) illustration of actual mechanism of load transfer



b) model showing effective forces P_1 and P_2, which result in identical deflections

FIGURE 8 Load transfer diagrams.

cause the shear force after overlay (V_o) is equal to $P_1 - P_2$, the equation can be rearranged to solve for V_o:

$$V_o = P \bullet (w_\ell - w_u)/(w_\ell + w_u) \tag{8}$$

The next step in the determination of the maximum shear strain is to estimate the shear moduli of the overlay layer(s). This is accomplished by using the following equation:

$$G = E/2(1 + \nu) \tag{9}$$

where

G = shear modulus (psi), with G_{OV} for the overlay and G_2 for the intermediate layer;
E = design dynamic modulus of the layer during critical temperature conditions (psi); and
ν = Poisson's ratio for the layer (0.30 recommended for asphalt cement hot-mix overlay, 0.35 for open-graded course intermediate layer).

These shear moduli are then used to determine an effective overlay thickness, D_e (in inches):

$$D_e = THOV + (G_2/G_{OV})\ TH2 \tag{10}$$

where THOV and TH2 are the thicknesses (in.) of overlay and intermediate layers, respectively.

Next, the maximum shear stress in the overlay layers is determined. If a section (A-A) is taken out of the overlay in the region where the shear force acts, then the distribution of shear stress along that section will be as shown in Figure 9. The following general equation defines the shear stress at any location along the face:

$$\tau = VQ/Ib \tag{11}$$

where

τ = shear stress (psi),
V = shear force (lb),
Q = first moment of the area above (or below, depending on the position of the neutral axis) the location where strain is desired ($in.^3$),
I = moment of inertia ($in.^4$), and
b = width of section (in.).

Note that for equilibrium of a small element taken at the top (or bottom) of the section, the shear stress must be zero.

A simplification of this equation can be used to estimate the maximum shear stress at the neutral axis of the cross section:

$$\tau_{max} = 3V/2bh \tag{12}$$

where

V = V_o, i.e., overlay shear force (lb),
b = width of the section (in.); for purposes of the overlay shear calculations, this value should be the width of the region of shear, which is approximately 25 in. for a dual-tired axle; and
h = height of cross section (in.); for the effective overlay thickness (D_e) for overlay shear calculations.

Next, the maximum shear strain in the overlay (γ_{OV}) is determined by using the following equation:

$$\gamma_{OV} = \tau_{OV}/G_{OV} \tag{13}$$

where $\tau_{OV} = \tau_{max}$, the maximum shear stress in the overlay (psi), and G_{OV} is the overlay shear modulus (psi).

Finally, the overlay life for a given shear strain is determined by using a fatigue-type relationship based on asphalt shear strain. Unfortunately, the available literature did not provide a relationship that could be used effectively in the model. Therefore, it was necessary to adapt the overlay tensile strain equation (developed in this study) to consider the effects of shear strain. This was accomplished by using known relationships between tensile and shear stress in the indirect tensile test and between normal and shear moduli [note

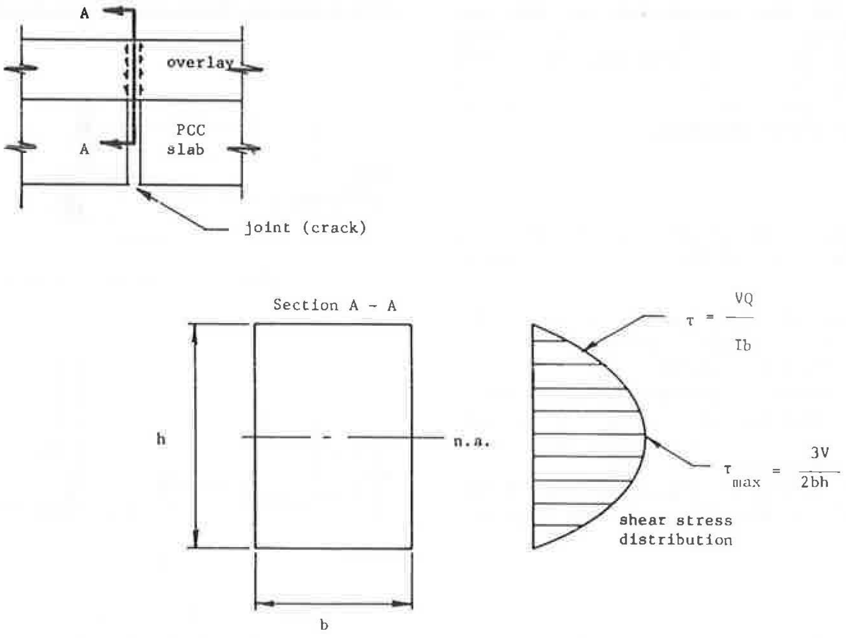


FIGURE 9 Distribution of shear stresses in the overlay.

that Equation 14 is from Anagnos and Kennedy (4) and Equation 15 is from Timoshenko and Gere (5)]:

$$\tau = 2 \cdot \sigma_T = 2 \cdot EDV \cdot \varepsilon_T \qquad (14)$$

$$G_{OV} = EDV/[2 \cdot (1 + \nu_{OV})] \qquad (15)$$

where ν_{OV} is Poisson's ratio for the overlay. Thus, overlay tensile strain can be converted to shear strain by using the following equation:

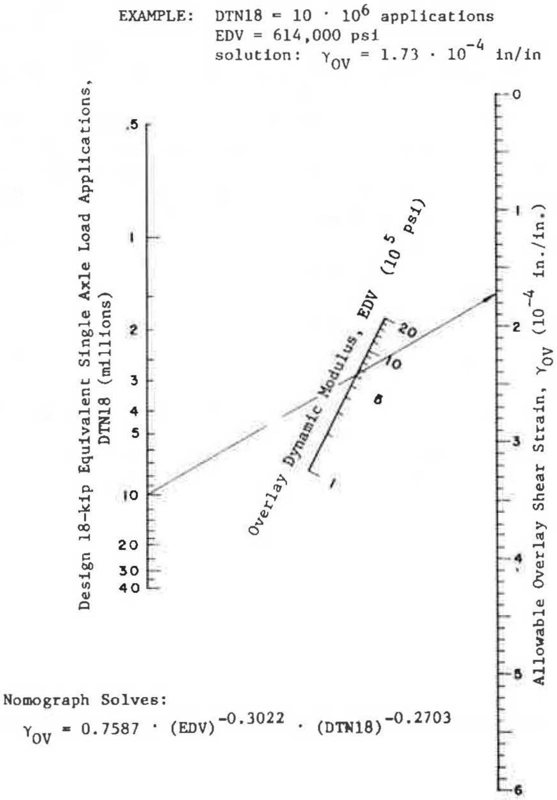
$$\varepsilon_T = \gamma_{OV}/[4(1 + \nu_{OV})] \qquad (16)$$

Then, when this is substituted into the tensile strain fatigue equation and rearranged to solve for allowable overlay shear strain, the result is the following equation (which assumes a value of 0.30 for Poisson's ratio of the overlay material):

$$\gamma_{OV} = 0.7587 \cdot (EDV)^{-0.3002} \cdot (N_T)^{-0.2703} \qquad (17)$$

where N_T = DTN18, the design 18-kip equivalent single-axle load (ESAL) applications that will be carried by the overlay before the development of reflection cracking; and EDV is the dynamic modulus of the overlay material (psi).

This section has thus far described the mechanics of the shear strain model in predicting the allowable 18-kip ESAL traffic. The design model incorporated into the ARKRC-2 program is based on the same concepts, but is formulated in reverse order. The user specifies a design 18-kip ESAL traffic and a possible overlay strategy and the program back-calculates a critical deflection factor (F_w). This factor is then used to single out the joints (or cracks) that are particularly damaging and may require structural maintenance to reduce the potential for generating reflection cracking after overlay.



EXAMPLE: DTN18 = $10 \cdot 10^6$ applications
EDV = 614,000 psi
solution: $\gamma_{OV} = 1.73 \cdot 10^{-4}$ in/in

Nomograph Solves:
$\gamma_{OV} = 0.7587 \cdot (EDV)^{-0.3022} \cdot (DTN18)^{-0.2703}$

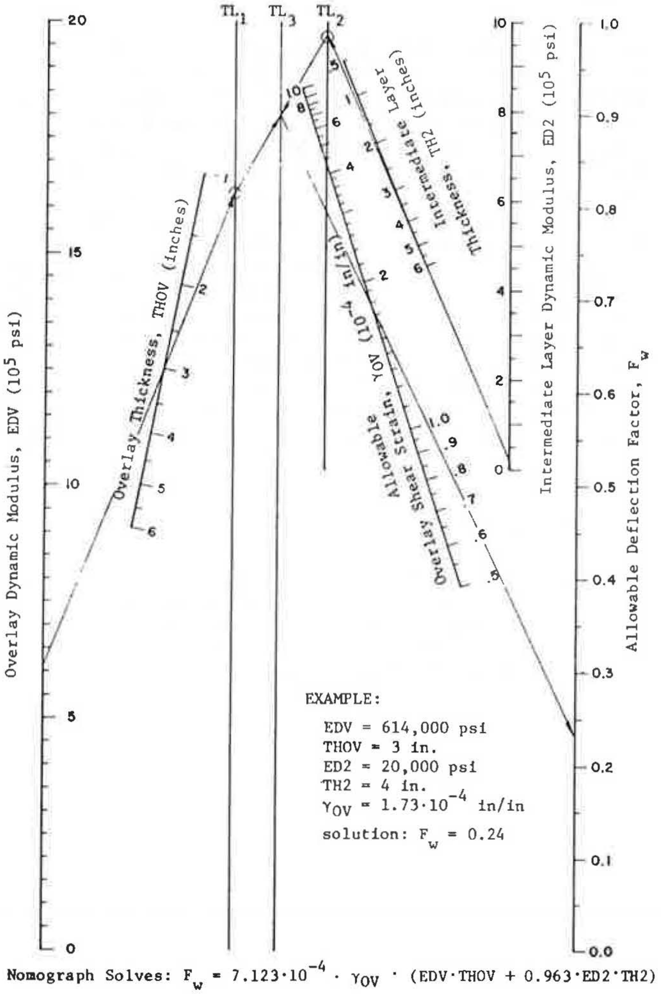
FIGURE 10 Nomograph for estimating allowable overlay shear strain.

Thus if the F_w value for a given joint (crack) calculated by using Equation 6 $[F_w = (w_\ell - w_\mu)/(w_\ell + w_\mu)]$ is greater than the critical F_w, then it is recommended that that joint or crack be undersealed before placing the overlay.

Use of Design Charts (Shear Strain Criteria)

Because of the simple form of the overlay design equation for shear strain criteria, it was possible to develop a series of nomographs in which all of the independent variables (factors) are considered. In this final section of the second part of the design procedure a description of how the design charts should be applied is given.

First, with the overlay dynamic modulus (EDV) and the design traffic (DTN18), use Figure 10 to estimate the allowable overlay shear strain. Then with the trial design (from tensile strain criteria), use the allowable overlay shear strain to determine the allowable deflection factor from Figure 11. Finally, draw a horizontal line on the longitudinal plot of the deflection factor that indicates the level of the allowable deflection factor (as shown in Figure 12). If inspection indicates that a point or series of points exceeds the allowable deflection factor, then it will be necessary to either underseal those joints or use an increased overlay thickness. The design for the latter may be accomplished by reusing



EXAMPLE:
EDV = 614,000 psi
THOV = 3 in.
ED2 = 20,000 psi
TH2 = 4 in.
$\gamma_{OV} = 1.73 \cdot 10^{-4}$ in/in
solution: $F_w = 0.24$

Nomograph Solves: $F_w = 7.123 \cdot 10^{-4} \cdot \gamma_{OV} \cdot (EDV \cdot THOV + 0.963 \cdot ED2 \cdot TH2)$

FIGURE 11 Nomograph for determining allowable deflection factor.

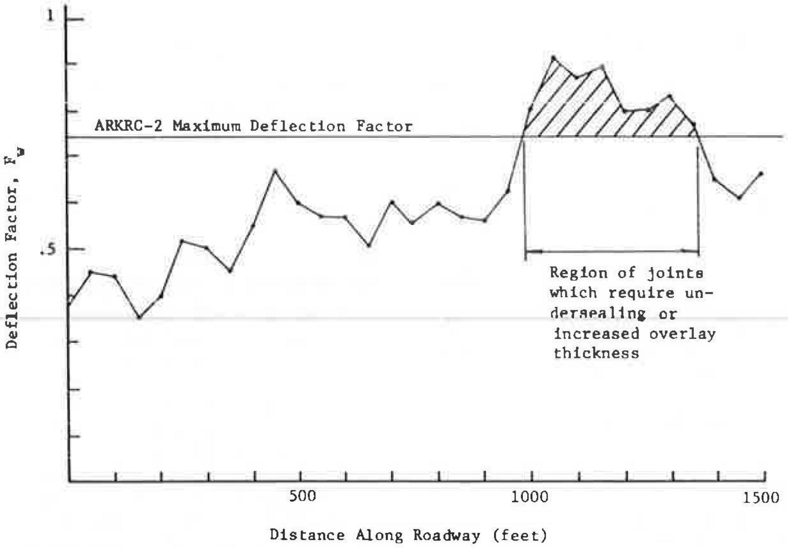


FIGURE 12 Graph of field deflection factors for 50-ft JCP illustrating application of ARKRC-2 maximum deflection factor in detecting joints that will cause premature reflection cracking in the overlay design considered.

Figure 11 with various increased levels of overlay thickness (THOV).

SUMMARY

A new procedure that has been developed for the design of asphalt concrete overlays on existing PCC pavements is described. The procedure uses a mechanistic analysis to evaluate field measurements of slab movement and predict overlay performance in terms of future reflection cracking. The method has been incorporated into both computerized and design chart (nomograph) procedures. The computer-based procedure considers several methods of controlling reflection cracking. Although the inherent fatigue equation is based on environmental conditions in Arkansas and Texas, the procedure is suitable for calibration and adaptation in almost any environment. The design chart procedure described is based solely on environmental conditions and construction practices common to Arkansas, but it is possible to develop similar nomographs for conditions in other states.

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