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PAVEMENT DESIGNS BASED ON WORK

by

Herbert F. Southgate, P.E.  
Chief Research Engineer

and

Robert C. Deen, P.E.  
Director

Kentucky Transportation Research Program  
College of Engineering  
University of Kentucky  
Lexington, Kentucky 40506-0043

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16. Abstract Thickness design curves presented in the report provide a systematic methodology for the selection of equivalent pavement designs for a broad range of layered systems. This is a unified system since the failure criteria are founded on the same concept of work strain and work. The analyses of stress-strain fields in the layered systems are based on elastic layered theory. This theory is represented by the Chevron N-layered computer program. The report also summarizes the historical development and evolution of pavement design in Kentucky.					
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## INTRODUCTION

The 1949 (1) and 1959 (2) Kentucky flexible pavement thickness design curves were based on empirical experience. Revisions made in 1968 (3), 1971 (4), and 1981 (5) combined elastic theory with empirical experience. The latter three revisions utilized a failure criterion of tensile strain at the bottom of the asphaltic concrete based on laboratory testing as well as a criterion developed from theoretical analyses of vertical compressive strains at the top of the subgrade.

## BACKGROUND

### 1959 KENTUCKY THICKNESS DESIGN CURVES

The 1949 Kentucky thickness design curves (1) were updated in 1959 (2) after a testing program consisting of pavement deflection tests and opening of test pits on selected pavements to perform plate bearing and in situ CBR tests, sampling of materials from each layer, and measuring surface ruts and/or distortion of layer boundaries. Traffic analyses provided estimates of accumulated fatigue, and the pavements were assigned a satisfactory or unsatisfactory performance rating. Pavements were sorted into groups having essentially the same accumulated traffic fatigue (equivalent 18-kip axleloads) and a curve was drawn through the thickness-CBR plots to separate satisfactory from unsatisfactory performances. Only two groups of "traffic fatigue" had sufficient data to permit a complete analysis. Those designated Traffic Curves IV and VI (3-6 million and 10-20 million EWL, respectively) represented two levels of equivalent wheelloads (EWLs). Other traffic curves (accumulated fatigue) were obtained by interpolation and extrapolation. Further analyses indicated that pavement thicknesses associated with Traffic Curve X (160-320 million EWL) should have been greater. The 1968 curves (3) did provide such adjustments, making pavements for Curve X (160-320 million EWL) thicker than before. The 1959 Traffic Curves were based on a 5-kip EWL and were converted to an equivalent 18-kip EAL for the 1968 Kentucky design curves. Pavements associated with a given traffic curve were thought at that time to manifest approximately the same surface deflections.

## ELASTIC THEORY APPLIED TO 1959 CURVES

In 1968, the Chevron N-layer computer program (based on elastic theory) (6) was obtained and used to analyze the 1959 Kentucky design curves (2). Those analyses indicated that pavements associated with the 1959 Traffic Curve X resulted in the same vertical compressive strains rather than the same surface deflections.

## ELASTIC MODULI

The elastic modulus for the asphaltic concrete pavements on the AASHO Road Test was determined to be approximately 600 ksi (7). This corresponded also to a mean annual temperature of 60 degrees F. The mean annual temperature for Kentucky is 70 degrees F, resulting in an equivalent modulus of approximately 480 ksi (3). Subgrades of pavements tested in 1957 had an average CBR of 7. Mitchell and Shen (8) had determined that the elastic modulus of clay could be estimated by multiplying the CBR by 1500.

## CRUSHED STONE BASE

In the 1968 analyses, a value of 25 ksi was assigned as the modulus for crushed stone base. Subsequent analyses showed that this created a weak layer between two stronger layers for subgrade CBRs greater than 17, leading to unrealistic results.

Matching theory to Traffic Curve X also permitted development of a relationship of moduli for the crushed stone layer as a function of the modulus of the asphaltic concrete and the CBR of the subgrade. Analyses indicated that the modulus of the crushed stone layer increased as the CBR of the subgrade increased. The modulus of the crushed stone base was 2.8 times the modulus of the subgrade at CBR 7. A theoretical Bousinesq solution could be estimated as the CBR equivalent to the asphaltic modulus (480 ksi) divided by 1500 — a CBR of 320. These two data points were used to define

$$\log(F) = 0.674797 - 0.269364 * \log(\text{CBR}) \quad 1$$

in which F = factor used to multiply 1500 \* CBR to obtain the modulus for crushed stone base, and

CBR = California Bearing Ratio.

## FATIGUE CRITERIA

Dorman and Metcalf (9) determined from laboratory tests that fatigue of asphaltic concrete could be described by

$$\log(\epsilon_a) = -2.69897 - 0.163382 * \log(\text{EAL}) \quad 2$$

in which  $\epsilon_a$  = tensile strain at the bottom of the asphaltic concrete.

The above relationship (Figure 1) was used in the development of the 1971 (4) and 1981 (5) thickness design methods.

While developing the 1968 Kentucky thickness design curves (3), a vertical compressive strain of  $2.4 \times 10^{-4}$  at the top of a CBR 7 subgrade was determined to correspond to a fatigue of  $8 \times 10^6$  18-kip EALs under the center of one 9-kip circularly loaded area. Damage factors, sometimes called load equivalency factors, were calculated using

$$\text{DF} = (1.2504)^{(P-18)} \quad 3$$

in which P = axleload in kips and

DF = damage factor

(EAL assigned to  $\epsilon_a$  caused by one 18-kip axleload)

= -----

(EAL assigned to  $\epsilon_a$  caused by a given axleload)

Several pavement structures ranging from relatively thin to thick were subjected to a range of loads and analyzed using the Chevron N-layer program. An arithmetic mean of the vertical compressive strains (for different pavement thicknesses) was calculated for each of the various loads. Each mean strain value was plotted versus the associated EAL calculated using Equation 2 to produce Figure 2. This procedure resulted in a prominent downward bend of the criterion relationship for high EALs that was not an extension (extrapolation) of trends for EALs less than  $8 \times 10^6$ , resulting in thicker designs for large EALs.

## 1971 KENTUCKY THICKNESS DESIGN METHOD

A series of nomographs (4) were developed that accurately described the theoretical behavioral of pavements. Use of those nomographs by a group of engineering students resulted in such a range of pavement thickness designs that the nomographs were assessed as inappropriate for general use.

Several attributes and characteristics of the nomographs, however, became an integral part of subsequent research efforts. Results from the 1959 test pits (2) indicated that distortion at the top of the subgrade diminished as the pavement thickness increased, which in turn was related to an increase in design traffic. Thus, for designs greater than 4 million EALs, the subgrade should be fully protected from distortion. In general, geometrics of farm-to-market roads would preclude speeds necessary for hydroplaning because of water standing in the ruts on the pavement surface. Thus, the criterion for such roads would allow the subgrade to distort (rut) provided the asphaltic concrete is protected from fatigue cracking during the design life. Curve IA of the 1959 Kentucky thickness design method was associated with farm-to-market roads subjected to the equivalent of one application of an 18-kip axleload per day for 20 years. Criteria between 7,300 and 4 million EALs were established so the allowable distortion of the top of the subgrade decreased as EALs increased. That was accomplished by determining the required thicknesses associated with both the asphaltic concrete and subgrade strain criteria and adding a proportion of the difference in thicknesses to the thickness required by the asphaltic concrete criterion (Figure 3).

An attempt was made to assess the compatibility of both a strain-controlled and stress-controlled criterion for a range of moduli of asphaltic concrete (3). For a strain-control criterion, results of laboratory tests reported in the literature indicated that strain-fatigue relationships for a range of asphaltic concrete moduli could be expressed as parallel lines by most investigators or as a series of lines converging at very low tensile strains and a large number of repetitions by others. Analysis of Kentucky experience indicated that the most appropriate relationship could be expressed as a family of strain-fatigue lines associated with a range of moduli of asphaltic concrete that converge at one repetition and a large (catastrophic) value of tensile strain. Thus, for a given level of fatigue, values of tensile strain could be determined for any desired modulus of asphaltic concrete. However, the same approach was not applicable for the development of a single stress-controlled criterion. One repetition of a catastrophic

load resulted in a different critical stress value and a separate stress-fatigue relationship for each modulus of asphaltic concrete. Thus, it was not possible to develop a design method incorporating both stress and strain criteria.

#### REVISIONS TO CHEVRON N-LAYER COMPUTER PROGRAM

The original version of the Chevron N-layer computer program (6) utilized a single circular loaded area to obtain stresses, strains, and deflections at radii from the center of that load. The program was revised to input multiple loads at specified X-Y coordinates on the surface and to obtain stresses, strains, and deflections using superposition principles at any designated location within the X-Y-Z coordinate system, the Z coordinate being the depth below the surface. Documentation and example problems are contained in Reference 11. This development made it possible to investigate various loads, tire spacings, axle spacings, number of axles/tires within a group, uneven loads within the same group of tires/axles, tire contact pressures, etc. Another revision was the addition of an equation (10, 11) to calculate strain energy density.

With the revised program, all 100 combinations of layer thicknesses used at the AASHO Road Test, of which 67 were constructed (12), were analyzed. A matrix of tire loads ranging from 2,000 to 8,000 pounds on 500-pound increments per tire were applied to each pavement for a two-tired single axle with a spacing to represent a steering axle, a four-tired single axle, an eight-tired tandem axle, and a twelve-tired tridem (5). Additional studies were initiated to quantify the effects of different axleloads within the same group of axles (tandem and tridem) (13), additional axles in a group, additional tires per axle, and various tire pressures (14). These analyses permitted development of load equivalency factors, or damage factors, for axle and tire arrangements used at the AASHO Road Test (15) as well as new arrangements now seen on highways.

Results for the four-tired single axle were in very close agreement with results using the AASHTO Equation C-16 (16) at a level of serviceability of 2.5 and within the range of axleloads employed at the Road Test. However, the factors for tandem axles were different. Adjustment factors were developed to account for uneven axleloads within a tandem or tridem group for axles weighed on static weigh scales (14). Adjustment factors also were developed to account for increased tire contact pressures currently being used. The

effects of increased tire contact pressures vary widely, depending on the thickness of the asphaltic concrete layer (14). Results of those analyses were applied to a traffic stream on a particular interstate pavement and comparisons made. Accounting for tire/axle arrangements, uneven loading among axles in the same group, and increased tire contact pressures, the Kentucky load equivalencies resulted in an accumulated fatigue that was 1.27 times greater than that obtained using AASHTO factors (16, 17).

A computer program was written to calculate an average EAL for each vehicle type based on truck traffic weighed at a particular weigh station and recorded on a loadometer computer tape. The EAL was calculated for each vehicle, summed for that truck classification, and an average value calculated for each truck classification.

Using the Kentucky load equivalency relationships to calculate the average load equivalency per trip for each vehicle classification results in a calculated 18-kip EAL that is approximately 73 percent of that using the AASHTO load equivalency relationships. Combining the effects of uneven loading and the Kentucky load equivalency relationships results in an 18-kip EAL that is approximately 88 percent of the AASHTO EAL. Combining the Kentucky load equivalency relationships, uneven loading, and effects of tire contact pressure results in an 18-kip EAL that is approximately 127 percent of the AASHTO EAL. Comparing results within the Kentucky procedures only, uneven loading between the axles within that group of axles increases the calculated EAL by approximately 20 percent. Increased tire contact pressure resulted in approximately an additional 55 percent increase in EAL. Combining the additional effects of uneven load distribution and increased tire contact pressure increased the calculated EAL approximately 75 percent compared to the EAL using only Kentucky load equivalency relationships.

#### 1981 KENTUCKY THICKNESS DESIGN CURVES

The same matrix of pavement layer thicknesses used in the 1968 (3) and 1971 (4) studies was used in this revision. The modulus of the asphaltic concrete was selected to be 480 ksi, based on results discussed previously. The modulus of the crushed stone base was varied according to Equation 1. An 18-kip four-tired single axleload for tire spacings in use on current trucks was applied to the surface of each pavement. The 80-psi tire contact pressure was used in all cases. Poisson's ratio was 0.40 for asphaltic concrete and dense-graded aggregate bases and 0.45 for the subgrade.

For a given subgrade modulus, the vertical compressive strain at the top of the subgrade was plotted as a function of thickness of asphaltic concrete by visual fit. Curves of equal thickness of dense-graded aggregate were drawn. The same methodology was used to plot relationships of surface deflection and tensile strain at the bottom of the asphaltic concrete.

The same criteria used in the 1971 and 1981 Kentucky thickness design methods (4, 5) were used in the development of thickness design curves for pavements where the thickness of the asphaltic concrete was the same percentage of the total thickness. Thickness design curves were developed for four percentages (33, 50, 75, and 100) of asphaltic concrete (5). Subsequently, curves were developed for pavements consisting of asphaltic concrete on 4 inches of dense-graded aggregate base and asphaltic concrete on stabilized base materials (18) such as pozzolanic materials, cement-treated materials, fly-ash mixtures, etc.

#### RUTTING INVESTIGATIONS

With the influx of heavy trucks into the coal fields of Kentucky came increased rutting of heavy duty pavements. Most severe rutting was located on fairly steep upgrades and at intersections having traffic control devices. To investigate this phenomenon, 4-foot wide trenches were dug to a depth of approximately 3 feet so various layers could be observed carefully. Two of those pavements consisted of 17 and 18 inches of full-depth asphaltic concrete. For both those pavements, there was no distortion from the bottom of the asphaltic concrete up to 6 inches below the surface of the pavement. From that depth to the surface, the distortion increased. Distortion in the top 6 inches was noted in three ways:

1. In the wheelpaths, the construction interface for each succeeding lift of pavement was displaced vertically more severely than the interface below.
2. The top layer of surface mix was much thinner in the wheelpaths and much thicker between wheelpaths, indicating shear flow within the surface mix.
3. Random orientation of aggregate particles in the lower layers changed gradually from the 6-inch depth to a parallel orientation at the surface. Aggregate particles very near the

surface appeared to be separated by, and sliding on, a layer of asphalt cement.

To verify shear flow was occurring, two 0.25-inch wide by 0.25-inch deep cuts were made in the surface of the pavement. The first cut was made at an angle to the centerline of the road, and the second cut was perpendicular to the centerline. These cuts were filled with glass beads used in traffic paint to prevent closure and to provide an identification of the cuts later. After three weeks, the cuts in the wheelpaths were displaced downhill approximately 0.6 inch. No measurements were made to determine if the cuts had been displaced laterally.

#### FUNDAMENTAL RELATIONSHIPS

##### WORK

The following equation (10, 11, 19) was added to the Chevron N-layer computer program to calculate the strain energy density at a given point within the pavement structure:

$$W = (1/2)*\lambda v^2 + \mu*(e_{11}^2 + e_{22}^2 + e_{33}^2 + 2*e_{12}^2 + 2*e_{23}^2 + 2*e_{13}^2) \quad 4$$

in which W = strain energy density, or energy of deformation per unit volume, or the volume density of strain energy,

$e_{ij}$  = i, jth component of the strain tensor,

$\mu$  =  $E/(2(1 + \sigma))$ , the modulus of rigidity or the shear modulus,

E = Young's modulus,

$\sigma$  = Poisson's ratio

$\lambda$  =  $E\sigma/((1 + \sigma)(1 - 2\sigma))$ , and

$v = e_{11} + e_{22} + e_{33}$ .

Strain energy density takes into account all nine components of strain, or stress, four of which will have no resultant value because one shear component balances another component for two situations. However, all components are calculated and printed. Work is the three-dimensional summation of strain energy density for the volume of material involved. Thus, it was assumed that, for a unit volume at a given point in the pavement

structure, work also was equal to the calculated strain energy density (Work =  $\text{in.}^3 \times \text{psi} = \text{in.-lb}$ ).

#### WORK STRAIN

"Work strain" as used in this study is not a pure strain. The equation to calculate strain energy density contains two different terms, each involving Poisson's ratio. Each term involves either the square of the sum of the principle strains or the sum of the square of each strain component. Dividing the strain energy density by  $0.5 * E$  and taking the square root yields a numerical value of the same order of magnitude as any individual strain component. This value has been called work strain and is taken as the net effect of all strain components. Similar calculations may be made using stress components and yielding work stress, equal to the product of Young's modulus of elasticity and work strain.

Laboratory fatigue testing typically utilizes strain gages placed at the bottom of rectangular beams of asphaltic concrete. Dimensions of the sample beam generally are such that the strain gage is functional only in the direction of the longest dimension. Thus, the tensile strain at the bottom of the asphaltic concrete has been used to develop theoretical thickness design curves.

It should be noted that, directly under the center of the load, the radial strain equals the tangential tensile strain. However, for any other location, the tangential tensile strain is the larger of the two, but in most cases is only slightly larger than the radial component. Why, then, should the radial component be ignored, and how should the radial component be included? Likewise, the shear component also may be significant, particularly at any location other than under the center of the loaded area. The 1981 Kentucky thickness design method, The Asphalt Institute design method (20), and the Shell design method (21) have utilized the tensile strain component at the bottom of the asphaltic concrete and the vertical compressive strain at the top of the subgrade and ignored all other components. No known thickness design procedure incorporates all strain, or stress, components into its criteria. Work strain incorporates the effects of all strain components and resolves the conflict discussed above.

## WORK VERSUS WORK STRAIN

The TOTAL amount of work done by the pavement structure due to an applied load involves a three-dimensional summation of strain energy density that becomes extremely expensive in terms of money and computer time. The area of greatest range of work would be within a volumetric slice of unit width passing through the center of the loaded areas along a given axle that is either a single axle or one of a group of axles. Locations of greatest unit work were identified in a previous study (22), and maximum unit work was noted to be under the edge of the tire closest to the other dual tire. The stress/strain component having the greatest variation from point to point under the tire contact area was shear. Shear has its greatest influence at the outer edge of the tire contact area. The vertical component of stress/strain is greatest at the center of the tire contact area.

Theoretically, the total amount of work caused by a given axleload is the same without regard to pavement structure. The work calculated at specific locations within a pavement structure varies according to layer thicknesses and subgrade moduli. This variation is relatively small, and a small change in work requires a large change in thickness. However, work strain provides a much wider range of values and permits easier usage in developing thickness design curves. Thus, for convenience and ease of use, work strain has been chosen as the variable upon which to base the thickness design curves.

## LOCATION OF GREATEST REACTION

An investigation was made to determine the location of the greatest reaction to applied loads within two- and three-layered pavement structures. For normally spaced dual tires, this location was beneath the edge of the tire closest to the adjacent tire (5, 13, 14, 22). Figure 4 illustrates the relationship between work at the bottom of the asphaltic concrete with respect to location along the axis of the axle. The numerical value of work was slightly larger beneath the edge of the inside tire than under the edge of the outside tire; but the difference was negligible, particularly when compared to the value of work under the center of the loaded area.

For two-layered pavement structures (22), the primary contributor to work was the shear component -- stress or strain. Also, the distribution of work from the center to the edge of the tire varied greatly, and the variation was predominantly due to the shear component. Maximum shear was reached at a depth from the surface equal to approximately 35 to 40 percent of the

thickness of the asphaltic concrete layer. Thus, for an 7.5-inch thickness of asphaltic concrete, the maximum shear would be at approximately 2.5 inches below the surface. Considering that Kentucky often uses a 1-inch surface over a 1.5-inch binder course, the maximum shear is very close to a construction interface -- the point of least aggregate interlock. At an interface, compaction equipment tends to orient the longest axis of an aggregate particle parallel to the surface. Thus, resistance to shearing is minimal, and particles may move laterally if friction between aggregate particles is overcome by loading forces and/or the tensile properties of the asphalt cement is exceeded, which may occur at higher pavement temperatures. Lateral movement of lower particles allows upper particles to move vertically downward, resulting in surface rutting within the wheelpath.

#### CHEVRON N-LAYER ANALYSES

A matrix of layer thicknesses, moduli of asphaltic concrete, subgrade moduli, and crushed stone moduli (as a function of subgrade modulus) were analyzed using the Chevron N-layer computer program as modified by Kentucky (11). The LaGrange interpolation method was used to fit third degree polynomial equations for various combinations within that matrix of thicknesses and then used to interpolate for other required pavement thicknesses.

#### COMPARISON OF WORK STRAIN AND STRAIN COMPONENTS

Figures 5 and 6 illustrate the relationship between work strain and tensile strain at the bottom of the asphaltic concrete and vertical compressive strain at the top of the subgrade, respectively. The correlation in Figure 5 is not as good as shown in Figure 6 because the relationship between radial strain and layer thicknesses results in a larger scatter of data than for the relationship between tensile strain and layer thickness. Since work strain is the net effect of all components, the correlation between work strain and radial strain is not as good as for work strain and tensile strain. The radial strain at the top of the subgrade is not nearly as influential as at the bottom of the asphaltic concrete layer. Thus, the relationship between work strain and vertical compressive strain at the top of the subgrade has a very narrow band of data scatter.

Many laboratory fatigue tests measure the tensile strain component. Thus, the relatively close correlation between work strain and tensile strain

of the bottom fiber has the advantage of utilizing all previous laboratory test results and permits utilization of previous experience. Confidence in the newer approach is increased because previous experience is utilized.

#### DAMAGE FACTORS

Fatigue is defined as

$$DF = \frac{\text{EAL associated with the work strain caused by 18-kip single axleload}}{\text{EAL associated with work strain caused by axleload P}} \quad 5$$

The 1959 Kentucky design curves were based on the following fatigue equation for single and tandem axles (2, 3):

$$DF_{59} = (A)^B \quad 6$$

in which  $DF_{59}$  = damage factor used in the 1959 Kentucky design system;

A = constant that is the slope of the semilogarithmic relationship between axleload and repetitions and has a value of 1.2504 for single axles with four tires and a value of 1.1254 for tandem axleloads;

B = (P - 10) to correspond to the EWL system used in the 1959 Kentucky design system,  
= (P - 18) for four-tired single axles (1968), and  
= (P - 32) for eight-tired tandem axle in the 1968 Kentucky design system; and

P = axleload in kips.

The 1981 Kentucky thickness design curves were based on the general equation expressed as

$$\log(DF_{81}) = C + D(P) + E(P)^2 \quad 7$$

in which  $DF_{81}$  = damage factor used as a part of the 1981 Kentucky design curves and

C, D, E = coefficients of correlation depending upon tire and axle configuration (see Table 1 of Reference 14).

#### COMPARISON OF 1959 AND 1981 KENTUCKY THICKNESS DESIGN CURVES

Pavement thicknesses represented by the 1959 Kentucky design curves were to be comprised of 1/3 asphaltic concrete and 2/3 crushed stone base. A table of layer design thicknesses had been prepared for convenience. As a result, the 1/3 - 2/3 ratio of asphaltic concrete to crushed stone base was not maintained. For example, the thickness of crushed stone base might vary as much as 3 to 4 inches for the same thickness of asphaltic concrete, resulting in percentages of asphaltic concrete thicknesses ranging from 28 to 52, rather than the design value of 33.

Thickness designs were determined for given values of CBR and 18-kip EALs from both the 1959 and 1981 curves. Values of work strain at the bottom of the asphaltic concrete and at the top of the subgrade were determined for those designs. Figure 7 shows the relationship between EAL and work strain at the bottom of the asphaltic concrete. The correlation is not good. Figure 8 shows the relationship between EAL and work strain at the top of the subgrade, and the correlation is much better. The larger scatter for the 1959 design curves is associated with the range in the percentages of the asphaltic concrete of the total thickness described above. Some of the scatter for the 1981 curves might be attributable to the use of one component of strain rather than work strain. However, best fit curves through their respective sets of data intersect each other as shown in Figure 8. For relatively high values of work strain, the 1959 curve is associated with a higher design EALs (thicker pavement) than that for the 1981 curve. Conversely, for lower values of work strain, the 1959 curve is associated with a lesser design EAL (thinner pavement) than for the 1981 curve.

#### FATIGUE RELATIONSHIPS

The 1959 Kentucky thickness design curves were empirically based; the 1981 Kentucky thickness design curves established a theoretical basis to merge with those empirical data. However, Figure 8 displays two intersecting curves (one for 1959 design curves and one for 1981 curves) for data that had been assumed to be essentially the same. Kentucky W-4 tables for 1959 through 1984 were used to describe a traffic stream. Using the count and weight data, it was possible to estimate the 20-year EAL based on each year's traffic data as

shown in Figure 9. Thus, a relationship was obtained that permitted adjusting an EAL calculated using the 1959 damage factors to an equivalent value based upon the 1981 damage factors given by

$$DF = 10^{(a + b \cdot P + c \cdot P^2)}$$

8

in which  $P$  = axleload in kips and

$a, b, c$  = constants given in Table 1 for respective axle configurations.

These correlations show that the empirical experience matches the theoretical work strain at the top of the subgrade while the limiting tensile capacity of the asphaltic concrete is not exceeded.

#### STRESS- OR STRAIN-CONTROLLED FATIGUE RELATIONSHIPS

Reference has been made earlier to the procedure used to proportion the design thickness according to the difference in thicknesses required by the tensile strain at the bottom of the asphaltic concrete and the vertical compressive strain at the top of the subgrade. Final design thicknesses from the 1981 Kentucky curves (5) for EALs less than  $4 \times 10^6$  has an equivalent vertical compressive strain that may be equated to an equivalent work strain and/or work stress. Therefore, one design procedure is possible for both stress-controlled and strain-controlled fatigue criteria as shown in Figure 10.

The "hooks" at either end of the 1981 Kentucky design curves (5) were the result of averaging large variances beyond the range in axleloads normally encountered on the highways. In the range of typical 18-kip design EALs, the relationship is almost a straight line on a log-log plot, as shown in Figure 11.

A regression equation was fitted to the relationship between work strain at the top of the subgrade and 18-kip EALs to determine the criterion for positioning thickness design curves. One equation produced close agreement with the 1981 Kentucky thickness designs for the range of low EALs. However, large discrepancies occurred for low CBR values in the range of high EALs. Thus, regression equations for criteria were obtained for three ranges of CBRs: 3 to 5, 5 to 7, and 7 and greater. These three curves became the basis for the development of all thickness design curves presented in this report.

The "hook" at each end of the criterion line (5) has been straightened for each of the three criterion lines in Figure 11. The result is that designs for low volume roads are slightly thicker than those required by the 1981 Kentucky design curves. For very high 18-kip EALs ( $10^7$  EALs), design thicknesses are slightly thinner than those required by the 1981 curves (5).

## 1987 KENTUCKY THICKNESS DESIGN CURVES

### CONVENTIONAL ASPHALTIC CONCRETE/DENSE-GRADED AGGREGATE DESIGNS

Total thickness design curves have been constructed for structures where the thicknesses of the asphaltic concrete are 33, 50, and 75 percent of the total pavement thickness as shown in Figures 12-14. Figure 15 is for pavement structures having a 4-inch thickness of dense-graded aggregate below the asphaltic concrete. These design curves were based on work strain at the top of the subgrade as the failure criterion. Design thicknesses shown on the vertical axes in Figures 12-15 are the total thickness of all layers above the subgrade.

### ASPHALTIC CONCRETE OVER BROKEN PORTLAND CEMENT CONCRETE

Analyses of dynamic deflections indicate the modulus of fragmented portland cement concrete is a function of the size of broken pieces. Preliminary analyses indicated the modulus is approximately 10 ksi for totally crushed material and increased to approximately 1,000 ksi for 30- to 36-inch pieces of portland cement concrete. Overlay thickness design curves using asphaltic concrete are presented for three moduli of broken and seated portland cement concrete pavements (Figures 16-18). Overlay thickness designs shown in Figures 16-18 may be calculated using the equation presented in Table 2.

### ASPHALTIC CONCRETE OVER PORTLAND CEMENT CONCRETE

Analyses (24) of temperature gradients in asphaltic and portland cement concrete pavements indicated that at least 5 inches of asphaltic concrete overlay is required so the temperature at the asphaltic-portland cement concrete interface will be no higher than if the portland cement concrete were exposed directly to the sun. The coefficient of heat transfer for the two materials is almost identical, but the coefficient of heat absorption for

black materials is almost twice that for white materials. Therefore, a significant thickness of black (asphaltic) material is required to dissipate the absorbed heat. Without the proper thickness of asphaltic concrete, the portland cement concrete will tend to expand with rising temperatures. If expansion is not possible, compressive forces will increase and may result in crushed concrete at the faces of sawed joints, or blowups may occur.

Any overlay over a joint that has differential vertical movement will exhibit a reflection crack and no thickness of overlay will prevent the reflection of that joint. If there is no vertical movement, the required overlay thickness is not a function of fatigue but is a function of the annual range of temperatures. Thicknesses in excess of 5 inches (for Kentucky conditions) will minimize horizontal (expansion/contraction) movements of the portland cement slab.

It does not appear that fatigue is the controlling factor for this type of pavement structure. With heavier axleloads and increased tire contact pressures, rutting has been observed in other states and provinces as a major problem. This phenomenon may be a result of shear flow within the asphaltic concrete layer.

Neither a critical magnitude of shear stress/strain nor a fatigue-shear relationship has been identified in literature. However, current research involves subjecting an asphaltic concrete tubular specimen to torsional shear (25). Test results indicate that the critical torsional shear stress is relatively low. Efforts are underway to compare torsional shear results for hollow and solid specimens. Therefore, it is premature to develop thickness design curves based on shear criterion.

#### PORTLAND CEMENT CONCRETE OVER ASPHALTIC CONCRETE

An existing asphaltic concrete pavement on a crushed stone base has a total thickness such that even a 4-inch portland cement concrete pavement on asphaltic concrete results in a very low value of work strain at the top of the subgrade. Thus, the work strain-fatigue relationship presented in Figure 8 is not appropriate. The required fatigue criterion must be appropriate to portland cement concrete.

Development of thickness design curves for portland cement concrete over a crushed stone base has been reported previously (26). The design criterion depended almost entirely upon the work strain at the bottom of the portland cement concrete slab. The fatigue criterion was a merger of empirical

criteria used by the Portland Cement Association and by the American Association of State Highway and Transportation Officials. The PCA design system was based on empirical data from highways having relatively low truck volumes and lesser gross loads in the 1940's as compared to much higher truck traffic with larger gross loads at the AASHO Road Test. The concept of work strain provided the necessary key to merge the two criteria into one. Thickness design curves presented in Figure 19 were developed on the assumption that the existing asphaltic concrete pavement has deteriorated until the modulus is 200 ksi instead of the 480 ksi of new material. The same fatigue criteria for portland cement concrete (26) is used herein.

#### PORTLAND CEMENT CONCRETE OVER PORTLAND CEMENT CONCRETE

Kentucky's thickness design curves (26) should be used for portland cement concrete overlays that are to be fully bonded to existing portland cement concrete. Unbonded portland cement concrete overlays are not recommended at this time. Warping and curling stresses cause the unbonded slab to be seated or unseated, depending on the time of day and traffic. It is conceivable that corners might break off and cracks develop at mid-slab prematurely. Locating new sawed joints directly over old joints and cracks is a difficult and tedious task. Some pavements overlaid with portland cement concrete were seen in Iowa where the sawed joint was only 0.25 inch from a crack reflected from the underlying slab. Reflection cracks are a severe problem for either bonded or unbonded concrete overlays. Joint sealing becomes more expensive when two closely spaced joints occur because the new sawed joint does not coincide with the old joint.

#### SUMMARY

A matching of empirical experience with elastic theory adequately defines fatigue. Fatigue failure is described by work strain at the top of the subgrade.

Shear is a material problem, at least partially related to mix design. However, the location of maximum shear is much nearer the surface than had been thought and the maximum tolerable value of shear strain, or stress, is much less than had been thought. Maximum shear occurs at depths that very often coincide with construction interfaces between layers of asphaltic

concrete. Consideration should be given to changing the construction thicknesses of layers near the surface of the pavement.

Calculated strains and stresses for multiple-tired loads usually are less than those due to a single loaded area.

The point of maximum work strain is located beneath the edge of the tire closest to the adjacent tire.

Concepts of work and work strain combine all components of strain into one net value. The work strain at the top of the subgrade appears to closely correspond with over 40 years of empirical experience in Kentucky.

Figure 10 shows that the concept of work strain and work stress provide the means to base a thickness design method on both strain-controlled and stress-controlled criteria.

The fatigue criterion for portland cement concrete pavements also applies to portland cement concrete overlays. Fatigue criteria for asphaltic concrete overlays are applicable to existing asphaltic concrete pavements and for broken and seated portland cement concrete pavements. Shear may overshadow fatigue criteria for asphaltic concrete overlays on existing non-broken portland cement concrete pavements.

The following is a listing of charts appropriate for various pavement designs:

<u>DESCRIPTION</u>	<u>FIGURE NO.</u>
New pavements, 33 percent asphaltic concrete, 67 percent dense-graded crushed stone base	12
New pavements, 50 percent asphaltic concrete, 50 percent dense-graded crushed stone base	13
New pavements, 75 percent asphaltic concrete, 25 percent dense-graded crushed stone base	14
New pavements, asphaltic concrete on 4-inch layer of dense-graded crushed stone base	15
Asphaltic concrete over broken and seated portland cement concrete having a modulus of 25 ksi	
9-inch concrete pavement	16a
10-inch concrete pavement	16b

Asphaltic concrete over broken and seated portland cement concrete having a modulus of 100 ksi	
9-inch concrete pavement	17a
10-inch concrete pavement	17b
Asphaltic concrete over broken and seated portland cement concrete having a modulus of 200 ksi	
9-inch concrete pavement	18a
10-inch concrete pavement	18b
Portland cement concrete over asphaltic concrete having a modulus of 200 ksi	19

#### FUTURE RESEARCH

The theme of the final session of the Sixth International Conference on Structural Design of Asphalt Pavements held on July 17, 1987, was "Paving the Gap". Several speakers stated that results of research should be in a format that is practical and easy to implement and use. Professor Peter Pell, University of Nottingham, stated that research should be practical; yet, there always will be a need to have fundamental research so frontiers will continue to be advanced. Such is the case with the concepts of work and work strain. Preliminary analyses of the observed behavior of pavements at the AASHO Road Test using principles of work strain show promise. Additional refinement of the analyses is needed. Combining the behavior of the AASHO Road Test with 45 years of Kentucky's empirical and theoretical designs would provide a sound basis for the development of a mechanistic thickness design system covering a wide range of input values for required parameters. Such a system has potential for analyzing data collected for the Long Term Pavement Performance portion of the Strategic Highway Research Program.

A cursory analysis indicated that shear may be more significant than fatigue in asphaltic concrete pavements and overlays as axleloads and tire contact pressures increase. Also, the amount of shear and the amount of work may be greatest in the top 3 to 4 inches (top 25 to 40 percent of the thickness of asphaltic concrete thickness).

## IMPLEMENTATION

Thickness design curves have been prepared for new asphaltic concrete pavements and asphaltic concrete overlays on broken and seated portland cement concrete pavement. All sets of curves are based upon the principal of work, provide equivalent designs, and may be implemented as presented in the report.

## REFERENCES

1. R. F. Baker and W. B. Drake, "Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements," Bulletin No. 1, Vol 4, Engineering Experiment Station, University of Kentucky, Lexington, KY, September 1949.
2. W. B. Drake and J. H. Havens, "Kentucky Flexible Pavement Design Studies," Bulletin No. 4, Vol 13, Engineering Experiment Station, University of Kentucky, Lexington, KY, June 1959.
3. H. F. Southgate, R. C. Deen, and J. H. Havens, "Rational Analysis of Kentucky Flexible Pavement Design Criterion," Research Report 270, Division of Research, Kentucky Department of Highways, Lexington, KY, November 1968.
4. R. C. Deen, H. F. Southgate, and J. H. Havens, "Structural Analysis of Bituminous Concrete Pavements," Research Report 305, Division of Research, Kentucky Department of Highways, Lexington, KY, May 1971.
5. J. H. Havens, R. C. Deen, and H. E. Southgate, "Design Guide for Bituminous Concrete Pavement Structures," Research Report UKTRP-81-17, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, KY, August 1981.
6. J. Michelow, "Analysis of Stresses and Displacements in an n-Layered Elastic System under a Load Uniformly Distributed on a Circular Area," unpublished, California Research Corporation, Chevron Oil Company, Richmond, CA, September 24, 1963.
7. AASHO Road Test: St. Louis Conference Proceedings, Washington, D.C., Special Report 73, Highway Research Board, 1962.
8. J. K. Mitchell and C. K. Shen, "Soil-Cement Properties Determined by Repeated Loading in Relation to Bases for Flexible Pavements," Proceedings, Second International Conference, Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, MI, 1968.
9. G. M. Dormon, and C. T. Metcalf, "Design Curves for Flexible Pavements Based on Layered Systems Theory," Record No. 71, Highway Research Board, Washington, D. C., 1965.
10. R. C. Deen, H. F. Southgate, and J. G. Mayes, "The Effect of Truck Design on Pavement Performance," Proceedings, Vol 49, The Association of Asphalt Paving Technologists, St. Paul, MN, 1980.
11. H. F. Southgate, R. C. Deen, D. H. Cain, and J. G. Mayes, "Modification to Chevron N-Layer Computer Program," Research Report UKTRP-87-28, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, KY, October 1987.
12. AASHO Road Test: Report 2 -- Materials and Construction, Highway Research Board, Washington, D. C., 1962.

13. H. F. Southgate, R. C. Deen, and J. G. Mayes, "Strain Energy Analysis of Pavement Designs for Heavy Trucks," Record 949, Transportation Research Board, Washington, D. C., 1982.
14. H. F. Southgate and R. C. Deen, "Effects of Load Distributions and Axle and Tire Configurations on Pavement Fatigue," Proceedings, Sixth International Conference, Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, MI, 1987; also Proceedings, Second National Weigh-in-Motion Conference, Vol One, Atlanta, GA, 1985.
15. AASHTO Road Test: Report 5--Pavement Research, Highway Research Board, Washington, D. C., 1962.
16. 1972 AASHTO Interim Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1972 (1982 Printing).
17. Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1986.
18. H. F. Southgate, "Thickness Design Curves for Asphaltic Concrete on a 4-inch Layer of Dense-Graded Aggregate, or on 6, 9, or 12 Inches of Stabilized Soil, or for Maximum Utilization of Dense-Graded Aggregate," Research Report UKTRP-86-14, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, KY, May 1986.
19. I. S. Sokolnikoff, Mathematical Theory of Elasticity, Second Edition, McGraw-Hill Book Company, New York, 1956.
20. Thickness Design -- Asphalt Pavements for Highways and Streets, Manual Series No. 1 (MS-1), The Asphalt Institute, College Park, MD, September 1981.
21. Shell 1963 Design Charts for Flexible Pavements, Shell International Petroleum Company Limited, London, UK, 1963; also, Addendum to the Shell Pavement Design Manual, London, UK, 1985.
22. H. F. Southgate and R. C. Deen, "Distributions of Strain Components and Work within Flexible Pavement Structures," Research Report UKTRP-86-21, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, KY, September 1986.
23. K. Atkinson, Elementary Numerical Analysis, John Wiley & Sons, New York, NY, 1985.
24. H. F. Southgate, D. C. Newberry, Jr., R. C. Deen, and J. H. Havens, "Resurfacing, Restoration, and Rehabilitation of Interstate Highways: Criteria and Logic Used to Determine January 3, 1977 Needs and Estimates of Costs," Research Report 475, Division of Research, Kentucky Department of Highways, Lexington, KY, July 1977.

25. J. M. B. de Sousa, "Dynamic Properties of Pavement Materials," Dissertation Series UCB-ITS-DS-86-3, Institute of Transportation Studies, University of California, Berkeley, December 1986.
26. H. F. Southgate and R. C. Deen, "Thickness Design Curves for Portland Cement Concrete Pavements," Research Report UKTRP-84-3, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, Lexington, KY, February 1984.

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TABLE 1. REGRESSION COEFFICIENTS TO CALCULATE DAMAGE FACTORS FOR VARIOUS AXLE CONFIGURATIONS

$$\log(\text{Damage Factor}) = a + b(\log(\text{Load})) + c(\log(\text{load}))^2$$

AXLE CONFIGURATION	COEFFICIENTS		
	a	b	c
Two-Tired Single Front Axle	-3.540112	2.728860	0.289133
Four-Tired Single Rear Axle	-3.439501	0.423747	1.846657
Eight-Tired Tandem Axle	-2.979479	-1.265144	2.007989
Twelve-Tired Tridem Axle	-2.740987	-1.873428	1.964442
Sixteen-Tired Quad Axle	-2.589482	-2.224981	1.923512

TABLE 2. VALUES FOR COEFFICIENTS TO CALCULATE OVERLAY THICKNESS OF ASPHALTIC CONCRETE ON BROKEN PCC

$$OLT = a + b \cdot \log(EAL) + c \cdot (\log(EAL))^2$$

where OLT = overlay thickness and

$$a, b, \text{ and } c = \text{regression coefficients} = f(\text{CBR}) \\ = d + e \cdot \log(\text{CBR}) + f \cdot (\log(\text{CBR}))^2$$

MODULUS OF BROKEN PCC (ksi)	THICKNESS OF BROKEN PCC (inches)	COEFFICIENT	COEFFICIENT		
			d	e	f
25	9	a	-39.55341825	7.559521621	-0.181394144
		b	12.27682107	-2.774707235	0.0884178858
		c	-0.612631087	0.210962531	-0.0077519796
	10	a	-70.28424248	21.272191482	-1.578324527
		b	20.54802239	-6.5107715697	0.46901845188
		c	-1.169270477	0.4633902917	-0.0335234984
100	9	a	-51.97485309	10.138201032	-0.494600481
		b	15.181766663	-3.5133660621	0.1729875084
		c	-0.819240853	0.2696910591	-0.0137252471
	10	a	-72.03822884	15.174210016	-0.837114874
		b	20.439819522	-4.8767616024	0.2654316355
		c	-1.171268401	0.3613069921	-0.0199286739
200	9	a	-58.32810531	11.980305722	-0.661994556
		b	16.282821001	-3.9776753781	0.2156855133
		c	-0.898751519	0.3016042334	-0.0165359824
	10	a	13.616843885	-0.538274742	0.0051893437
		b	-5.016295968	-0.3930013094	0.0298301792
		c	0.6511910044	0.0458919219	-0.0036225523

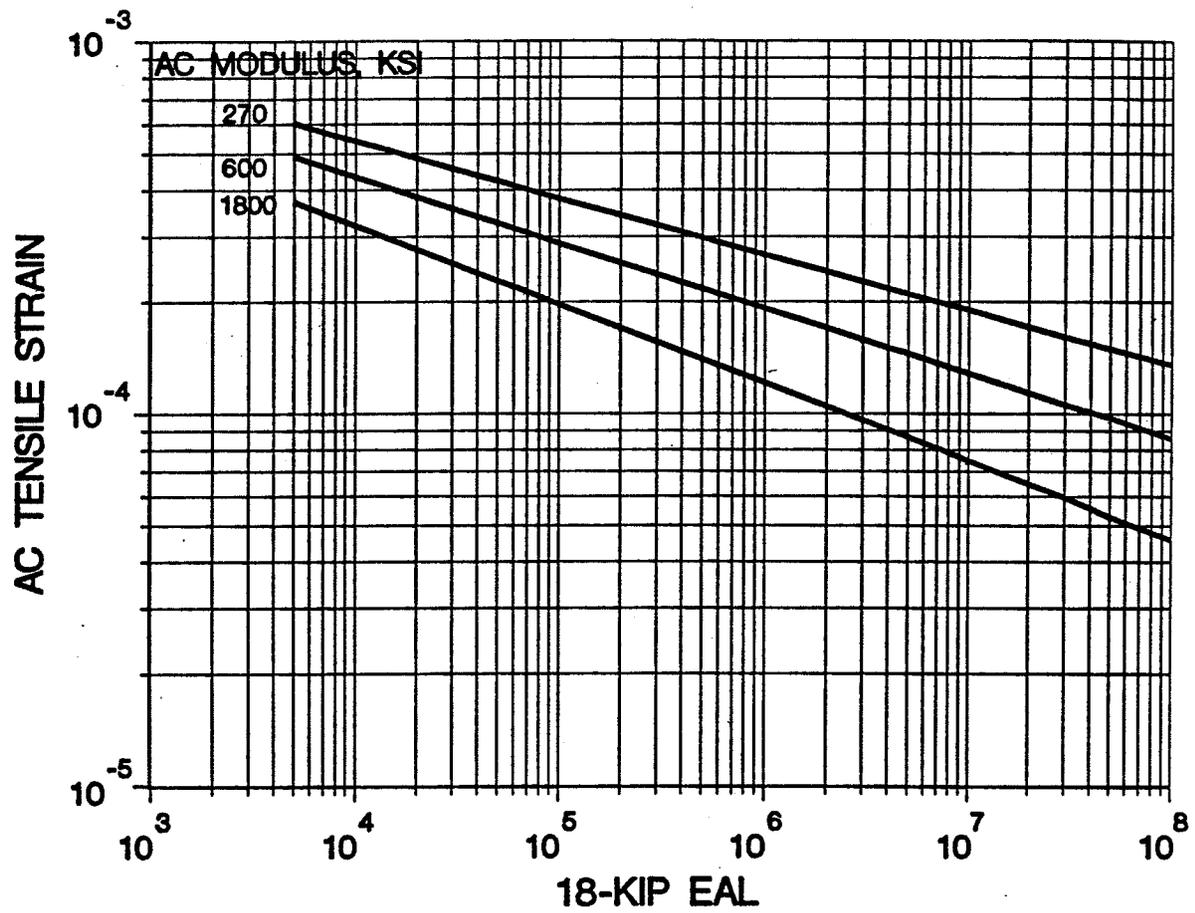


Figure 1. Relationship between Tensile Strain at Bottom of Asphaltic Concrete and Repetitions of 18-kip Single Axleload.

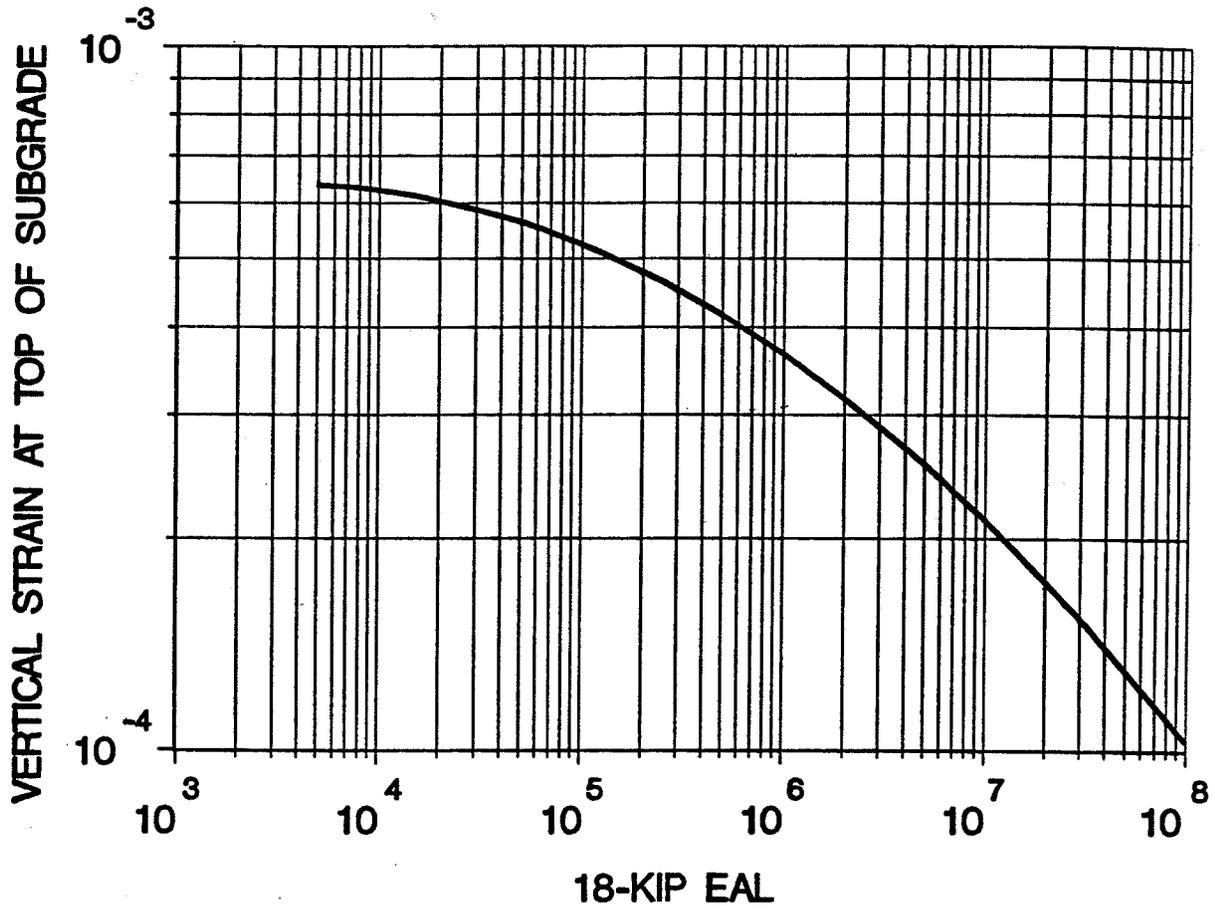


Figure 2. Relationship between Vertical Compressive Strain at Top of Subgrade and Repetitions of 18-kip Single Axleload.

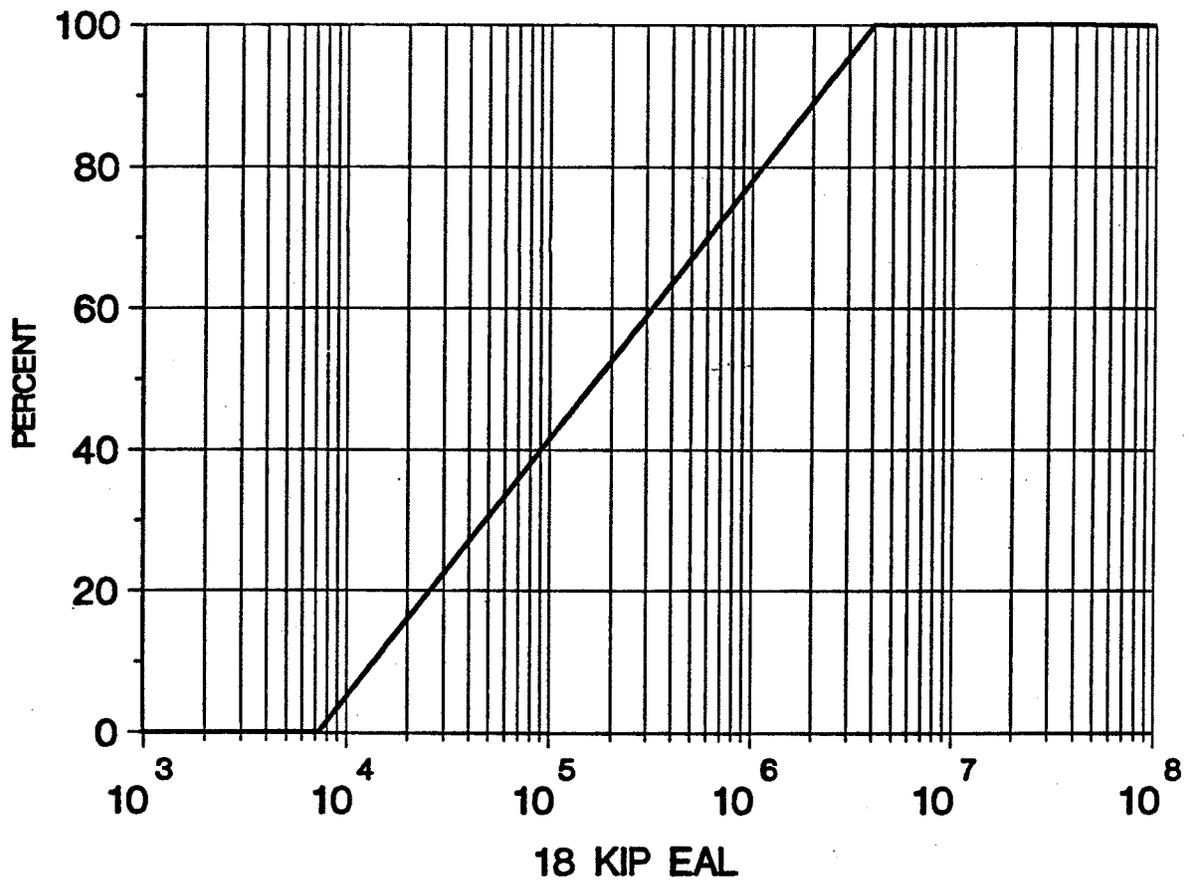


Figure 3. Adjustment of Design Thickness for Rutting as a Function of Repetitions of 18-kip EAL.

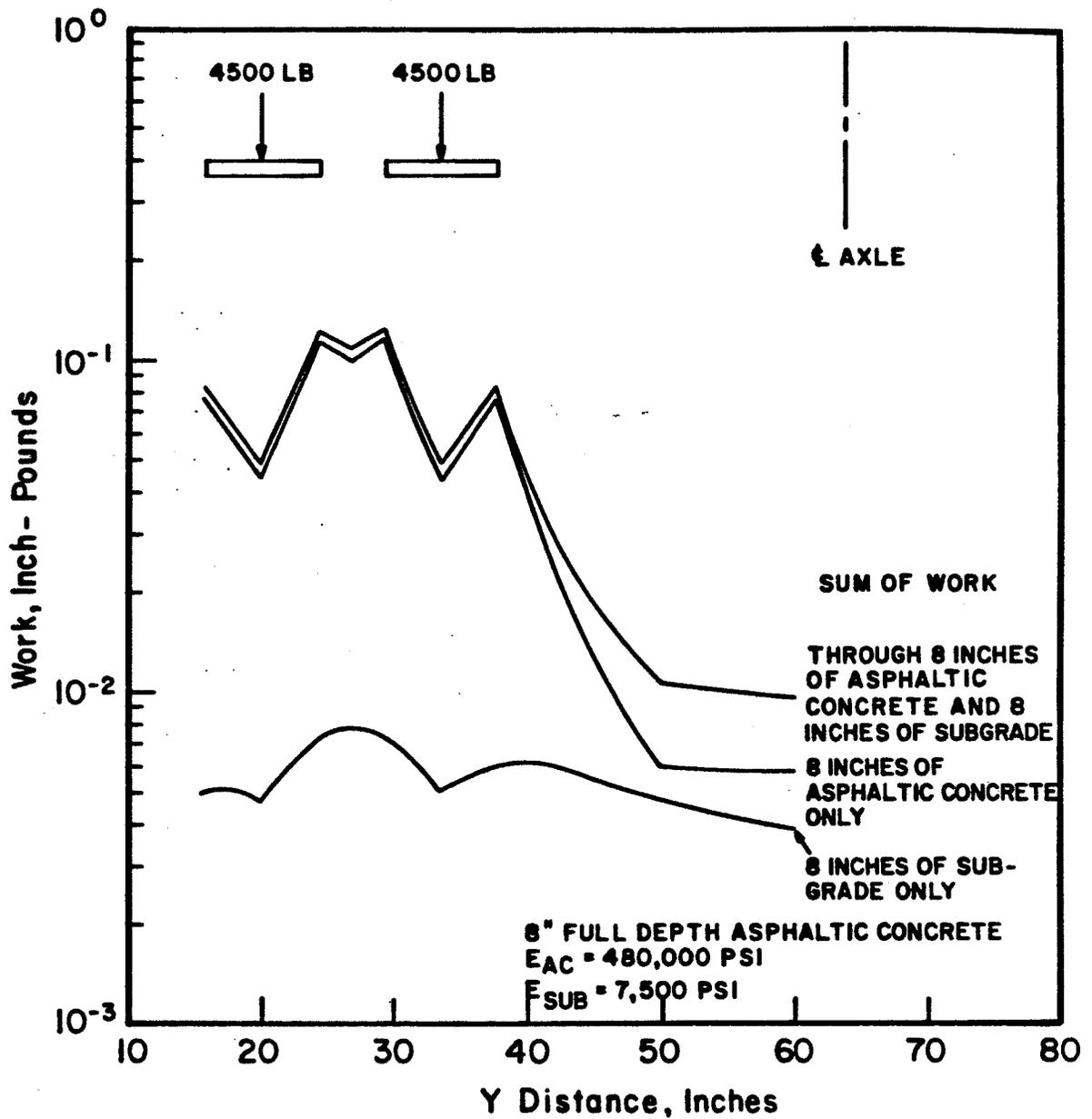


Figure 4. Work at the Bottom of Asphaltic Concrete versus Location along Axis of Axle.

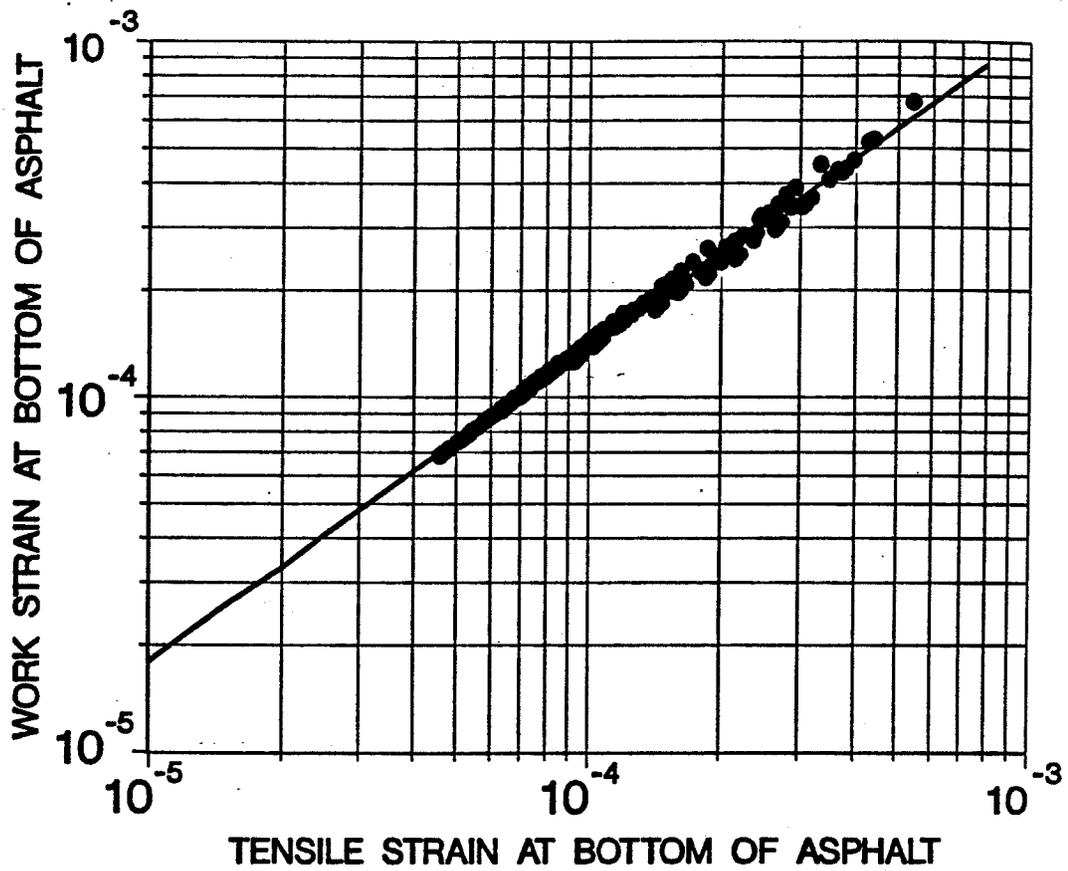


Figure 5. Relationship between Work Strain and Tensile Strain at Bottom of Asphaltic Concrete.

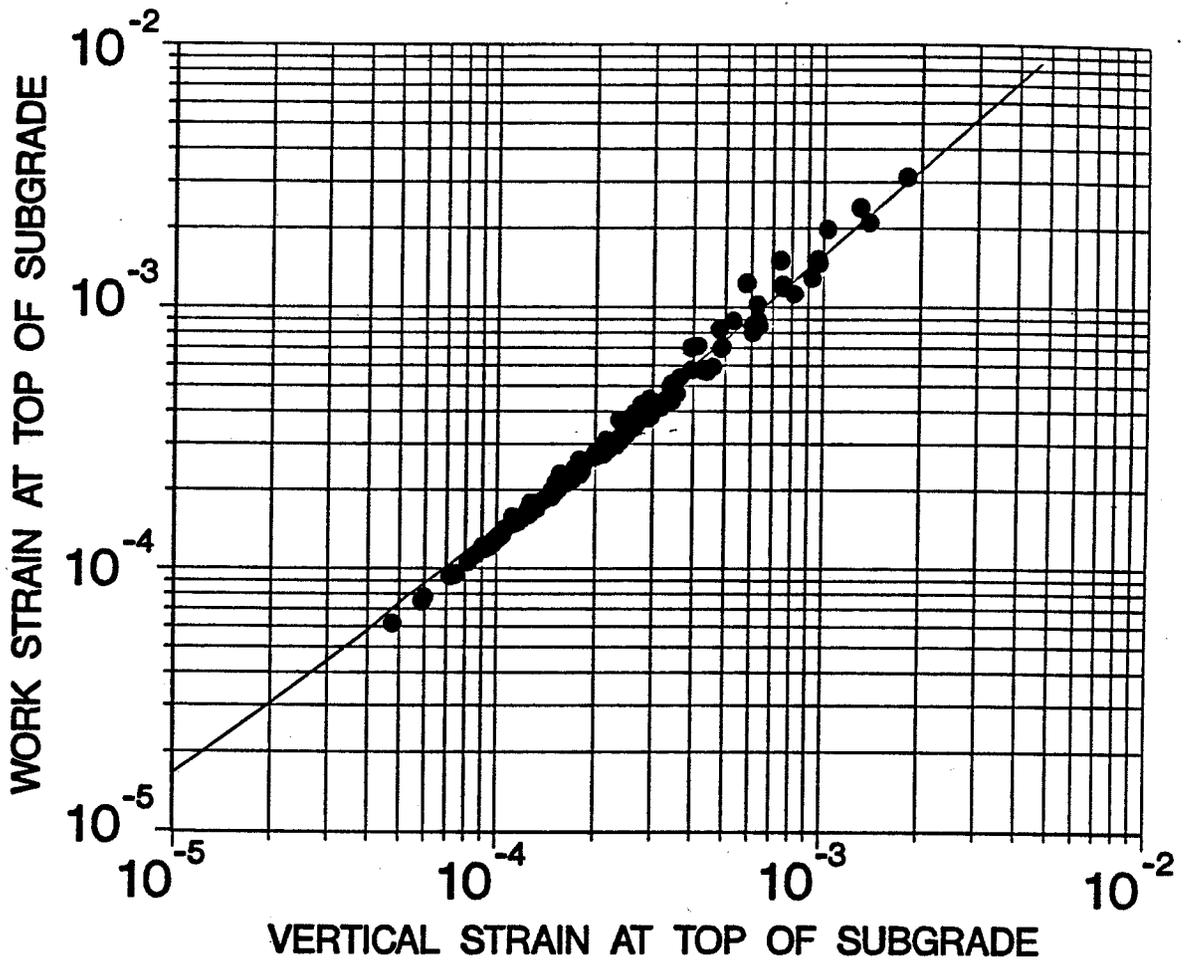


Figure 6. Relationship between Work Strain and Vertical Compressive Strain at Top of Subgrade.

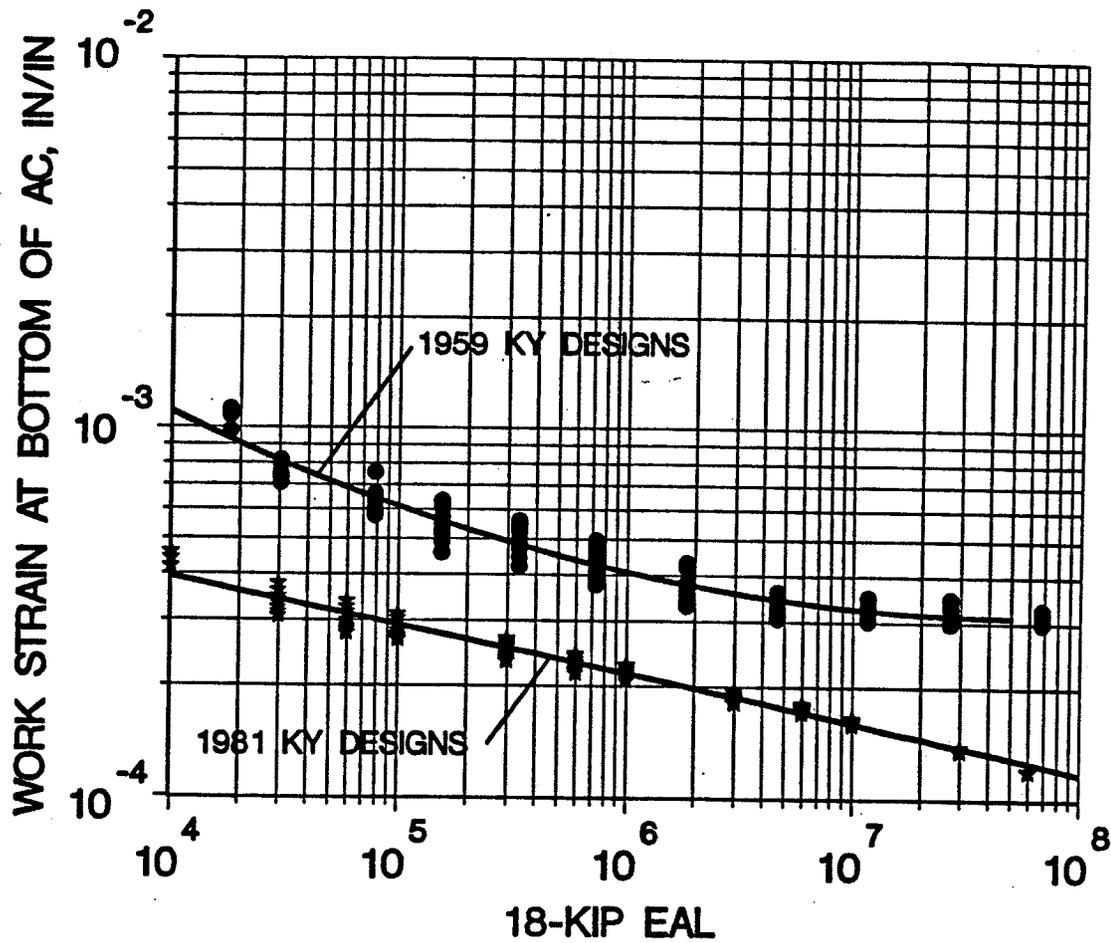


Figure 7. Relationship between Work Strain at Bottom of Asphaltic Concrete and Repetitions of 18-kip EAL.

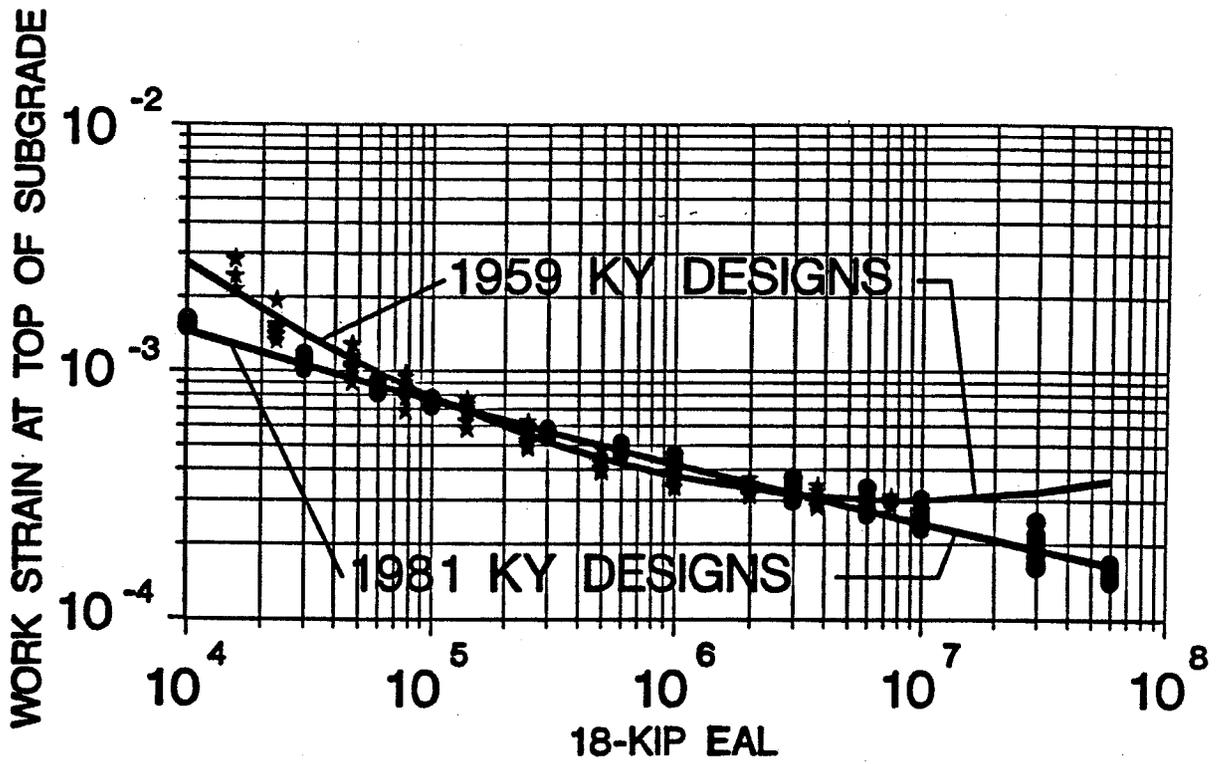


Figure B. Relationship between Work Strain at Top of Subgrade and Repetitions of 18-kip EAL.

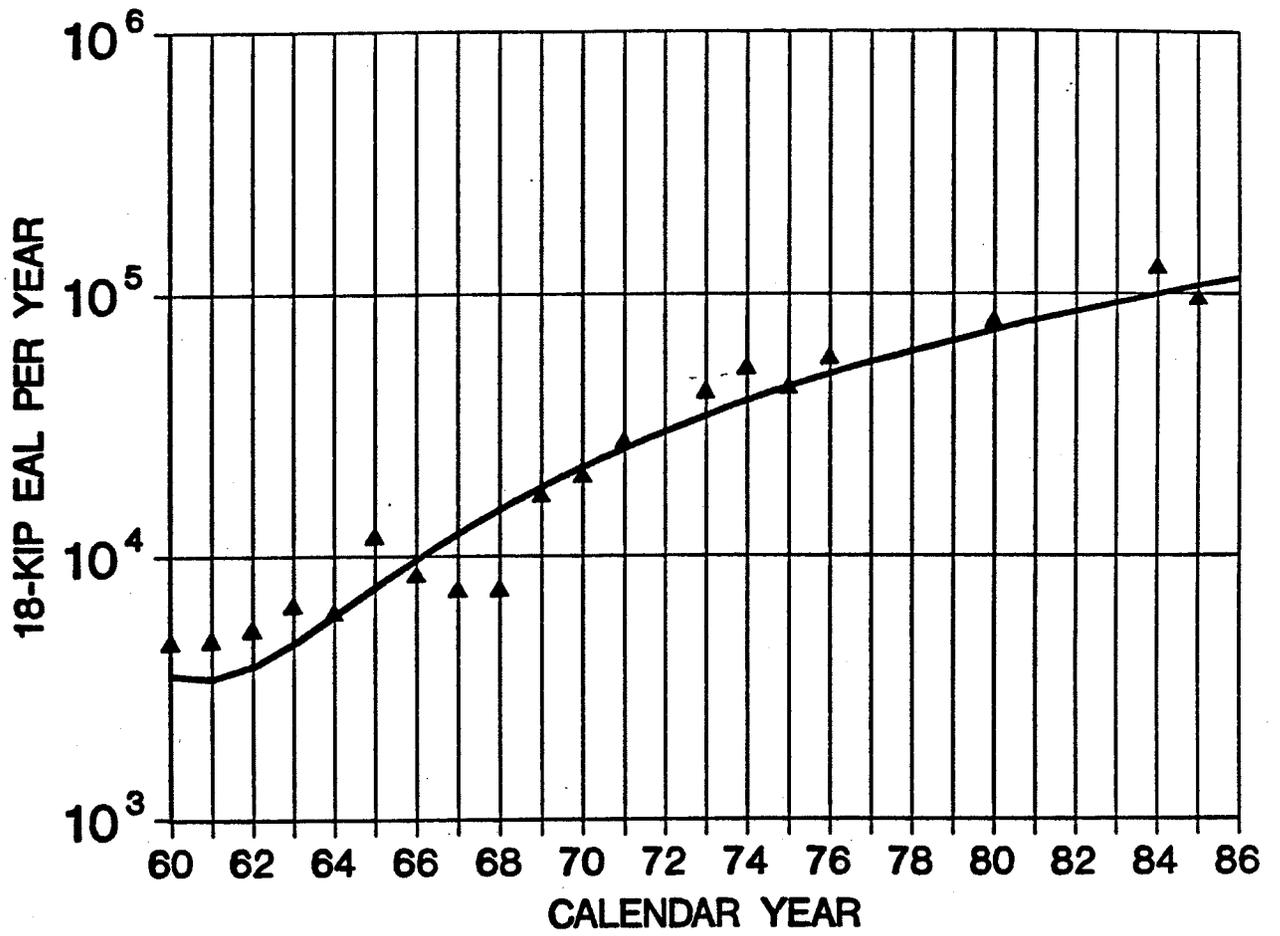


Figure 9. Estimated Annual 18-kip EAL versus Calendar Year.

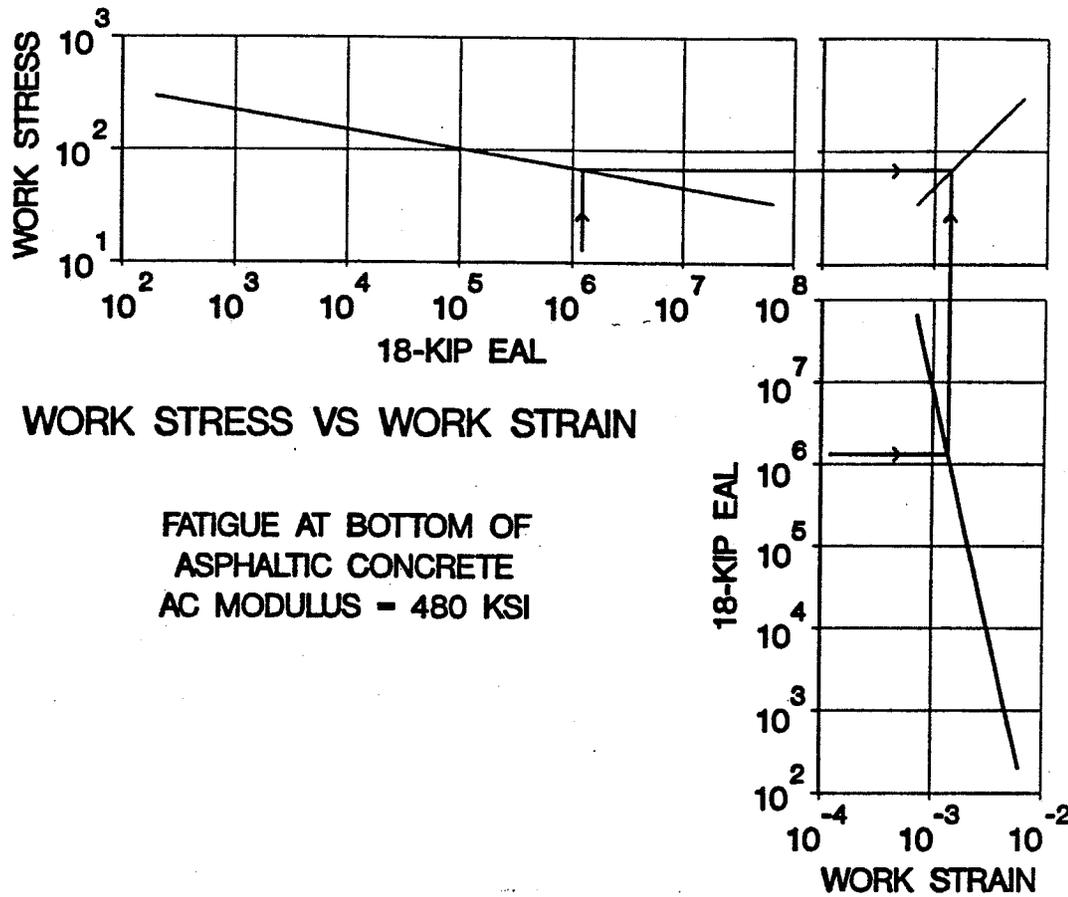


Figure 10. Pavement Design Procedure for Either Stress Control or Strain Control.

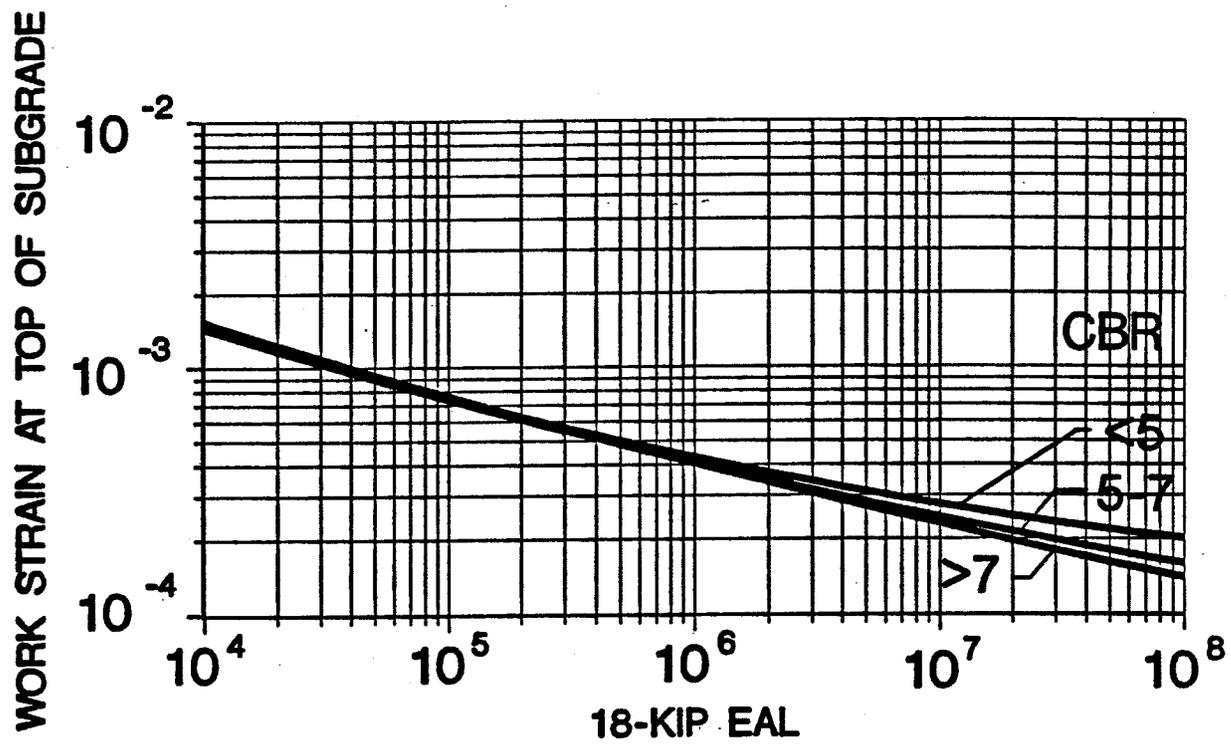


Figure 11. Work Strain at Top of Subgrade vs 18-kip EAL Criterion for Three Ranges of CBRs.

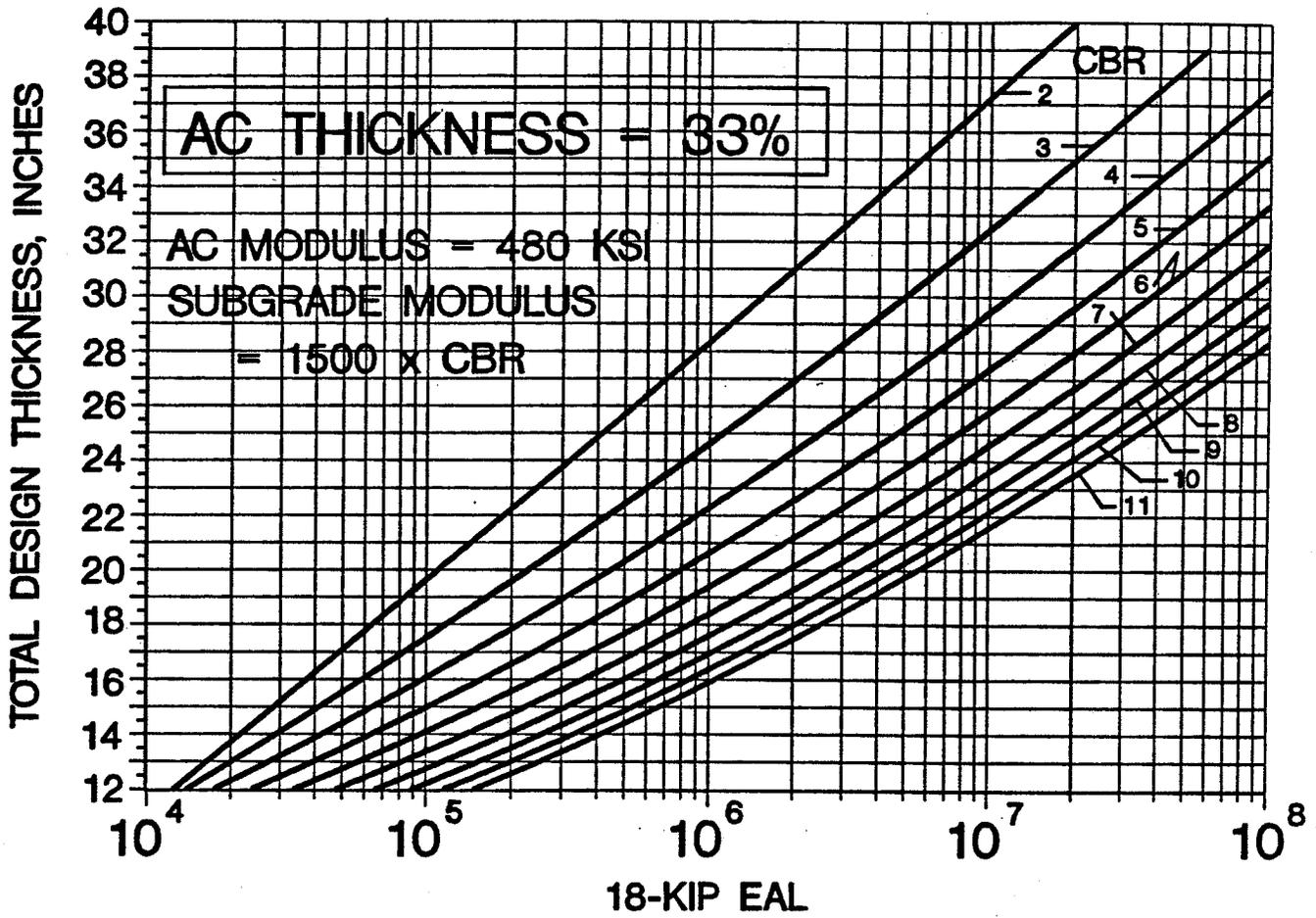


Figure 12. Thickness Design Curves for Pavement Structures Comprised of 33 Percent Asphaltic Concrete and 67 Percent Crushed Stone Aggregate Base.

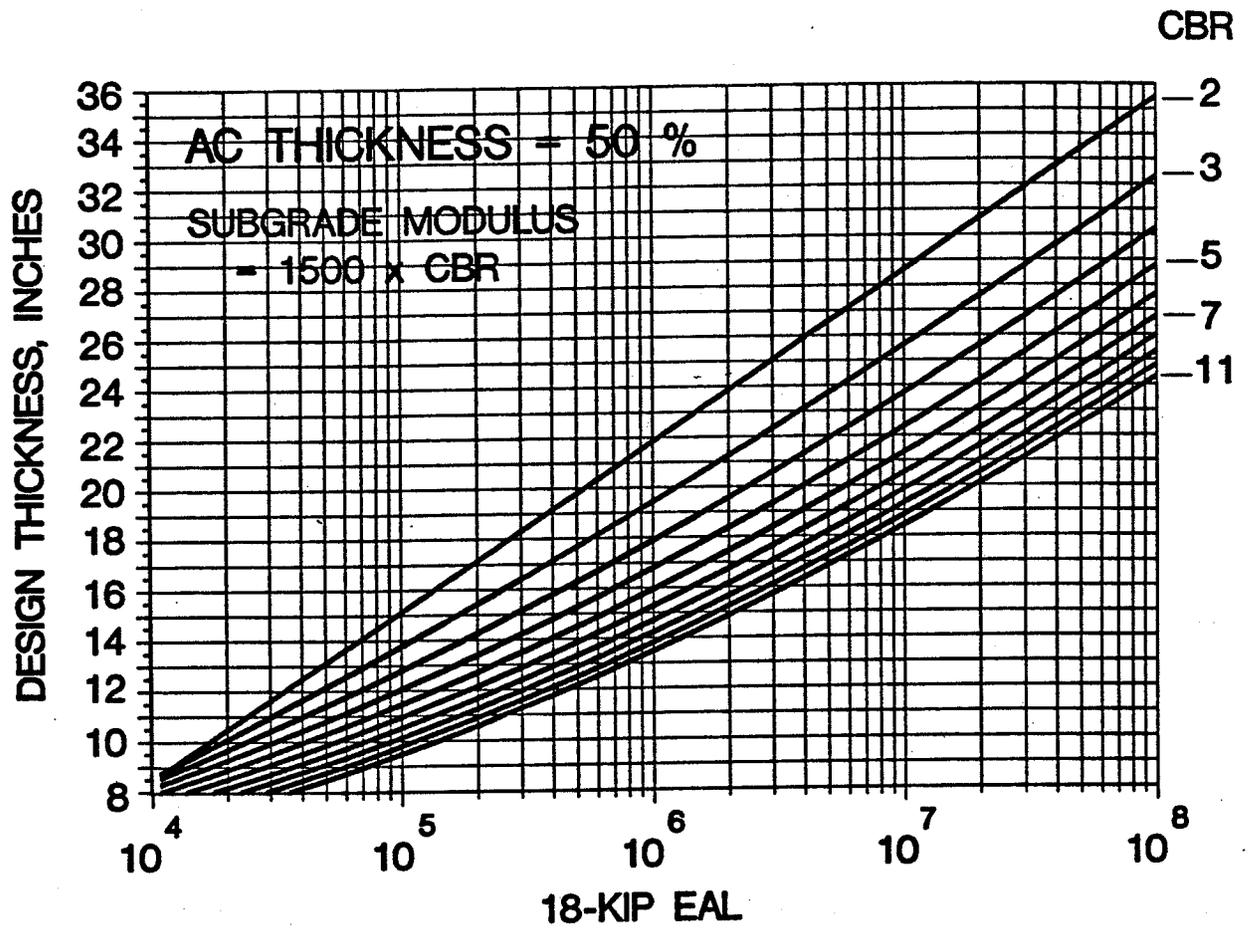


Figure 13. Thickness Design Curves for Pavement Structures Comprised of 50 Percent Asphaltic Concrete and 50 Percent Crushed Stone Aggregate Base.

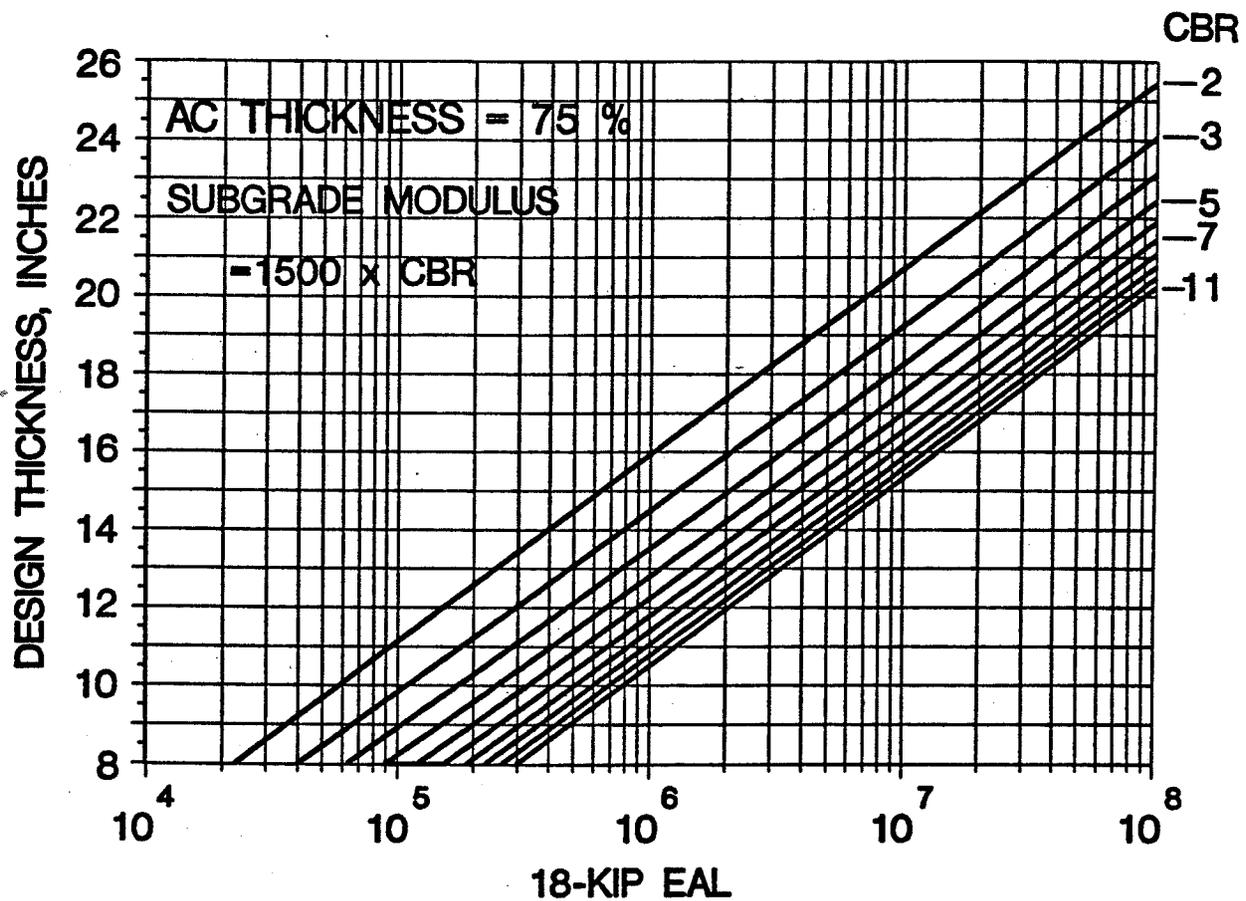


Figure 14. Thickness Design Curves for Pavement Structures Comprised of 75 Percent Asphaltic Concrete and 25 Percent Crushed Stone Aggregate Base.

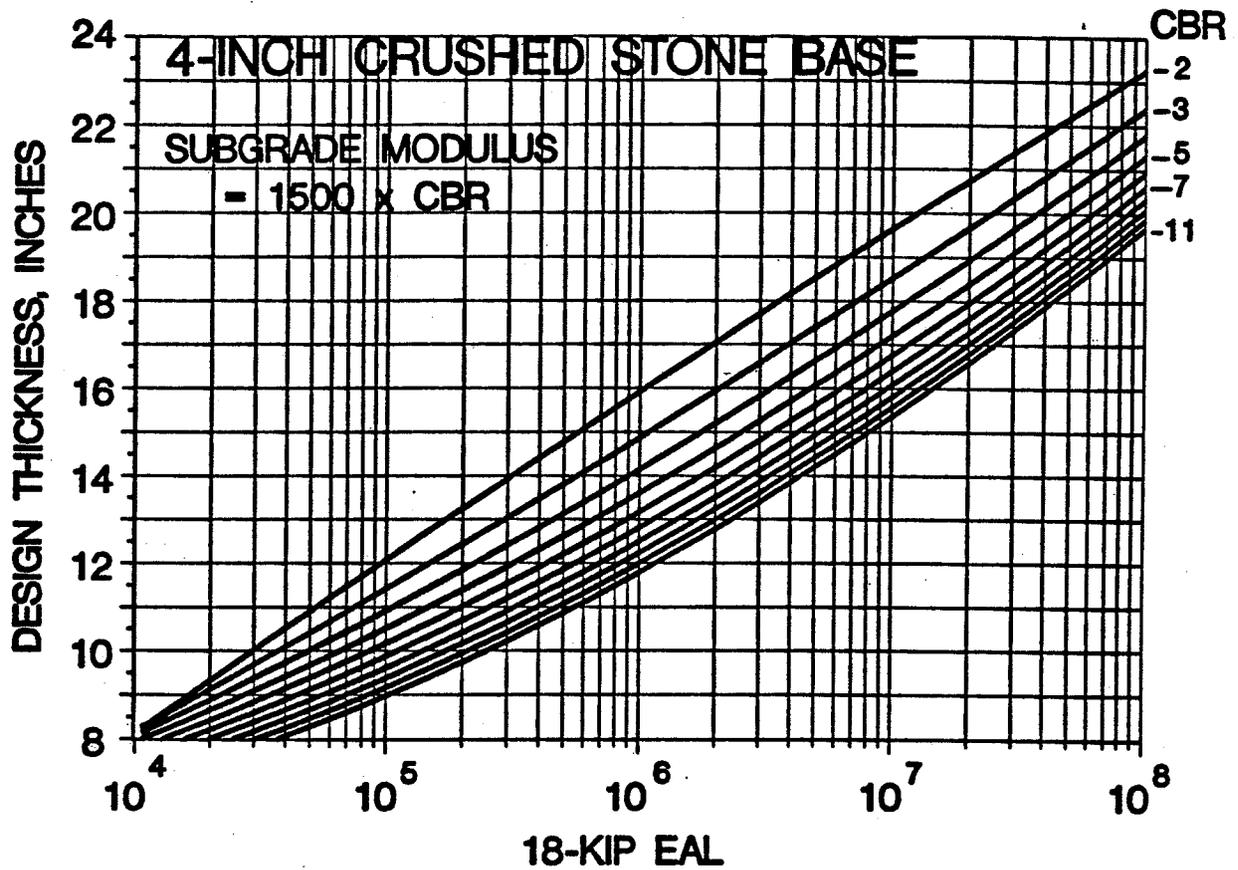


Figure 15. Thickness Design Curves for Pavement Structures Comprised of Asphaltic Concrete on 4 Inches of Crushed Stone Aggregate Base.

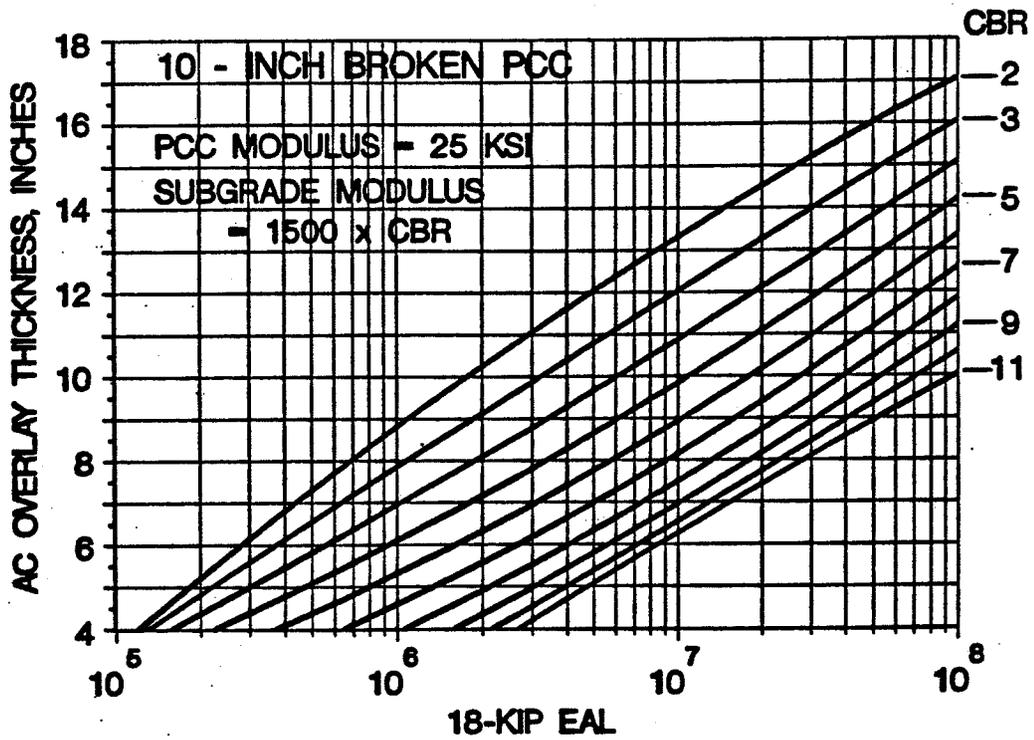
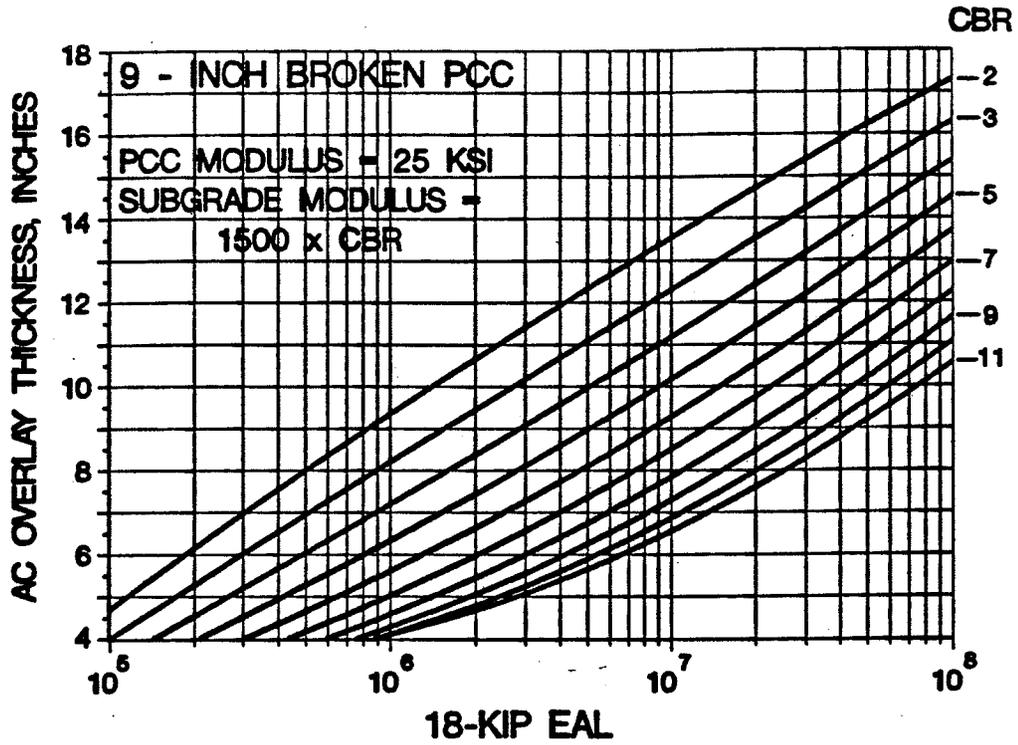


Figure 16. Thickness Design Curves for Asphaltic Concrete on Broken and Seated Portland Cement Concrete Having a Young's Modulus of 25 ksi.

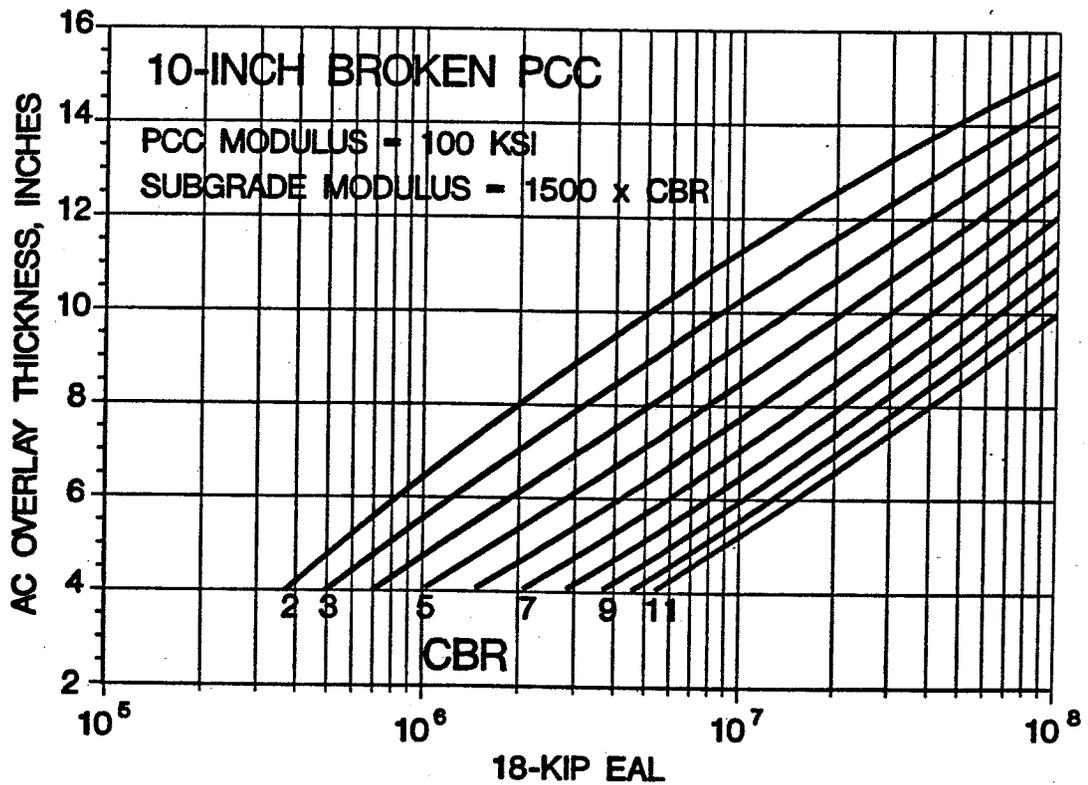
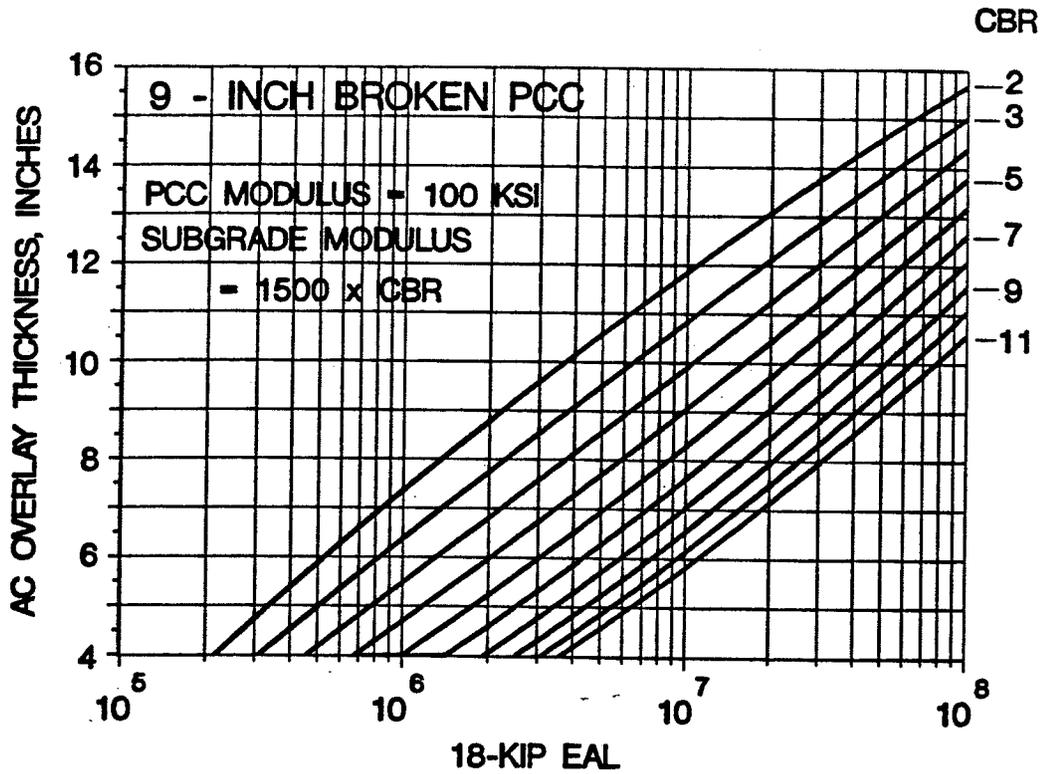


Figure 17. Thickness Design Curves for Asphaltic Concrete on Broken and Seated Portland Cement Concrete Having a Young's Modulus of 100 ksi.

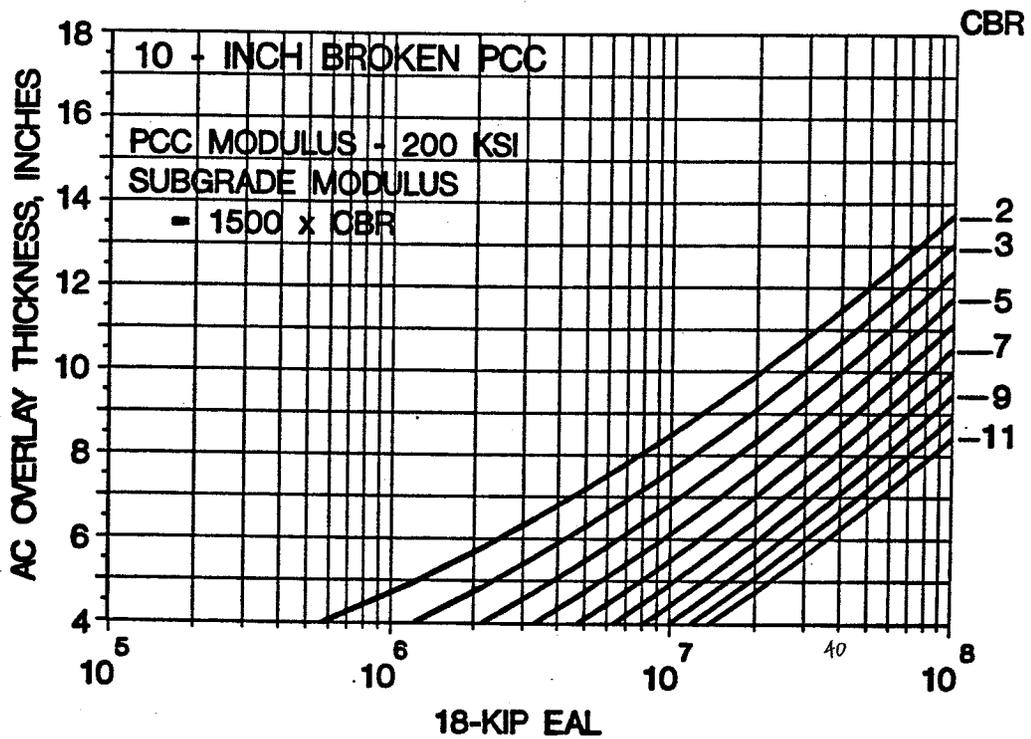
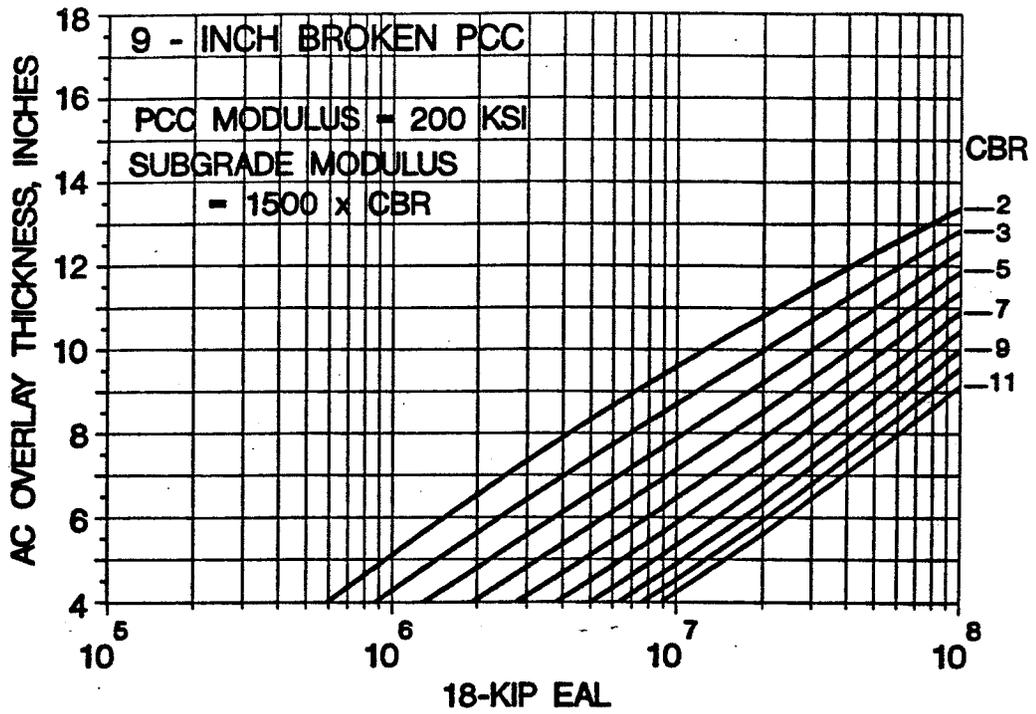


Figure 18. Thickness Design Curves for Asphaltic Concrete on Broken and Seated Portland Cement Concrete Having a Young's Modulus of 200 ksi.

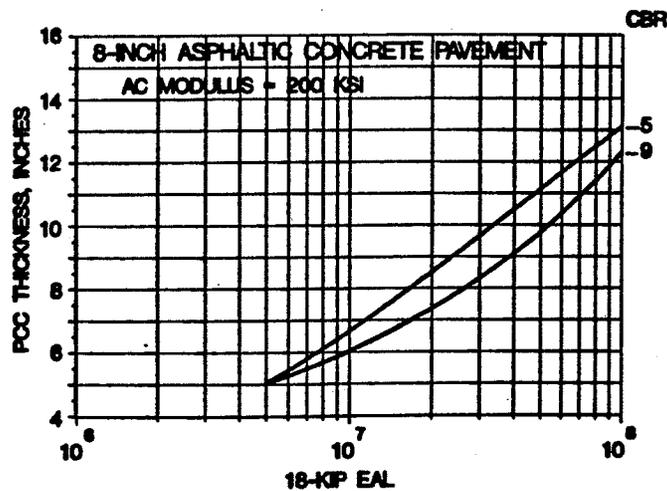
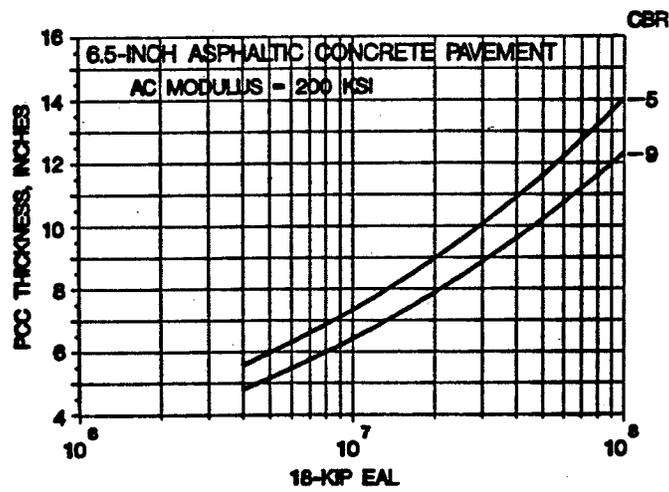
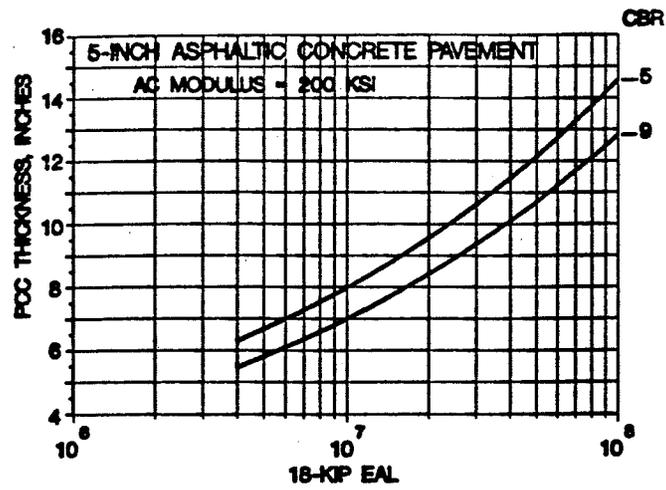


Figure 19. Thickness Design Curves for Portland Cement Concrete on Asphaltic Concrete Having a Young's Modulus of Elasticity of 200 ksi.