

Research Report
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**KENTUCKY RESEARCH:
A FLEXIBLE PAVEMENT DESIGN AND MANAGEMENT
SYSTEM**

by

Herbert F. Southgate
Research Engineer Chief

Robert C. Deen
Assistant Director of Research

James H. Havens
Director of Research

and

William B. Drake
Assistant State Highway Engineer
for Research

Bureau of Highways
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

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EXECUTIVE SUMMARY

Various strategies for designing pavement structures are discussed. Initial full-life design, stage designs and planned extensions of service life, final design, surface renewals for deslicking, no-defect designs for high-type high-volume facilities, and allowable-defect designs are considered. Economics enter in terms of salvage value of existing pavements and alternate designs using different proportions of materials within the structure.

The elastic model represented in Chevron's n-layered computer program is the basis for theoretical relationships. Ranges of values are given for input variables such as Young's moduli, Poisson's ratio, thicknesses for layers, tire pressure, and load. The Kentucky CBR is related to modulus by $E = 1500 \times \text{CBR}$ and is correlated with the AASHO Soil Support value and other strength relationships. The modulus of crushed stone base is shown to be a function of the moduli of the asphaltic concrete and subgrade. Appropriate relationships are given.

Graphs show interrelationships between asphaltic concrete thickness and asphaltic tensile strains, subgrade strains, and surface deflections. Structures are interpolated as a function of asphaltic concrete thickness and percent of asphaltic concrete in the total thickness. These graphs are interpolated to illustrate the relationship of Kentucky CBR versus total pavement thickness for structures in which the asphaltic concrete has a specified modulus and accounts for a specified proportion of the total thickness. The CBR - total thickness graphs are combined to make the Kentucky design nomographs. All of the above are theoretical and empirical relationships which are not dependent upon fatigue criteria.

A tensile strain-fatigue criterion is derived and correlated with Kentucky experience. The analysis is based on limiting elastic energies and may be treated thereafter in terms of limiting strains or limiting stresses. Fatigue energies are the summation of $\sigma_i \epsilon_i N_i / 2$, where N_i is the number of repetitions, σ_i is the stress, and ϵ_i is the strain.

The subgrade strain criterion is also associated with Kentucky field experience and is extended as a function of axleload; load and repetitions are interrelated to provide equivalent axleloads (EAL) by $\text{EAL}'s = N_i (1.2504)^{P - 18}$ for single axleloads, where $N_i =$ repetitions and $P =$ axleload in kips for single axleloads, and $\text{EAL}'s = N_i (1.1254)^{P - 34}$ where $P =$ tandem axleload in kips. These values are illustrated as log strain - log repetitions of 18-kip (80-kN) equivalents. A method for determining a design CBR value is discussed.

Methods of determining design periods, or design life, are discussed. Ways of determining appropriate traffic volumes are discussed and include available AADT data, use of classification and weight data from W-4 Tables, and modifications to account for known and unusual traffic variations such as for recreational areas, rural routes, and heavy hauling operations.

The methodology for computing load-damage factors is given, discussed, and compared to damage factors developed by the AASHO Design Committee from Test Road data. The value of subgrade strain correlated to Kentucky experience is that strain developed for an 18-kip (80-kN) axleload applied to typical Kentucky designs. Other axleloads producing other strains in the same structures are expressed as ratios to the 18-kip (80-kN) axleload. Thus, the coupling of repetitions, axleloads, and subgrade strain is accomplished in the load - repetitions equations. Graphs relating repetitions of 18-kip (80-kN) axleloads to subgrade and asphalt strains allow determination of the respective strain values for input to the CBR - thickness graphs or the Kentucky design nomographs to obtain design thicknesses with respect to the asphaltic concrete moduli.

The Kentucky criterion for rutting allows maximum rutting for the least traffic but minimum rutting for the highest traffic. A procedure is developed for adjusting thicknesses to a final design thickness for each of the four moduli given in the design nomograph.

The appropriate modulus for a given locale is determined from pavement temperature histories. A method is given in this paper. For Kentucky conditions, the recommended modulus is a function of the percentage of asphaltic concrete thickness in terms of the total design thickness.

The choice of percentage of asphaltic concrete of the total design thickness and their respective moduli permit development of design charts showing the thickness versus log repetitions of 18-kip (80-kN) axleloads. Each line on the chart is for a given CBR. All the designer needs to use these charts is the design CBR and number of repetitions. Since the design charts are mathematical solutions coupled with fatigue criteria, changes in the fatigue criteria can be properly made by simply relabeling the CBR lines to appropriate equivalent values.

The design charts can be used to design overlays and to determine when overlay construction should be scheduled in terms of accumulated EAL's. Again, AADT charts and yearly W-4 table classification and weight data can be readily used to estimate accumulated EAL's and to modify the design period, or design year, to account for increased weight limits, changes in style of cargo hauling equipment, routing, and distribution of vehicle types in the traffic stream.

Verification of this design system involve several sources, including some 30 to 40 years of design and behavioral experience within Kentucky and including a full-depth asphaltic concrete research pavement. Recent investigations involve re-analysis of AASHO Test Road data -- both single and tandem axleloads. Results indicate the AASHO Test Road data can be analyzed and explained by elastic theory. The design level of terminal serviceability is shown to be a function of design repetitions. This relationship is also expressed in terms of axleloads. Coupling the variable terminal serviceabilities with axleloads and, in turn, with Kentucky equivalent damage factors permit superpositioning of the AASHO Test Road data in terms of 18-kip (80-kN) EAL's for single axleloads. An additional relationship equating equal damage for single and tandem axles using AASHO Road Test data permits superpositioning of the tandem-axleload data over the single-axleload data with virtually no increase in data spread. Superimposed on the AASHO Road Test data are Kentucky moduli solutions showing excellent agreement of field data and theory.

Airport pavement thickness designs obtained from the Asphalt Institute's Full-Depth Asphalt Pavements for Air Carrier Airports is closely duplicated by converting aircraft wheel loads to equivalent 18-kip (80-kN) axleloads. A method is given equating moduli with frequency of loading cycle. It is shown that the Kentucky asphalt tensile strain criteria can be expressed in terms of $N = K(1/\epsilon_a)^c$, where K and c vary according to pavement temperature and ϵ_a is the asphaltic tensile strain.

The verification of this design system involves the incorporation of test data from Kentucky, Canada, the AASHO Test Road at Ottawa, Illinois, and test roads in Colorado, San Diego, and Kentucky. In addition, The Asphalt Institute's airport pavement design method uses aircraft strut and wheel loads which can be converted to an equivalent highway 18-kip (80-kN) EAL loading for pavement design

Herbert F. Southgate, Research Engineer Chief, Kentucky Bureau of Highways
Robert C. Deen, Assistant Director of Research, Kentucky Bureau of Highways
James H. Havens, Director of Research, Kentucky Bureau of Highways
William B. Drake, Jr., Assistant State Highway Engineer for Research, Kentucky Bureau of Highways

by the Kentucky method. In one example, The Asphalt Institute's airport pavement thickness was 17.0 inches (332 mm) full-depth asphaltic concrete. The same aircraft loadings resulted in a Kentucky design thickness of 17.2 inches (337 mm).

Pavement management concepts are discussed and a method is presented illustrating the required data and its use to accumulate EAL's annually for comparison with the design EAL. This method can be used to determine overlay priorities, overlay design thicknesses, and scheduled financing. Condition survey analyses coupled with economic analyses may indicate a pavement structure should be reconstructed, or even relieved by constructing a new corridor. A discussion of automatic feedback of field data is presented. Pavement condition reports may be fed into the data bank. However, such data should be analyzed separately to prevent improper adjustments to the design system due to causes of failure other than pavement fatigue.

Construction of overlays for any purpose should be logged into the data bank. The overlay, whether for extending service life or improving skid resistance, provides an additional structural thickness and will modify the design life.

INTRODUCTION

Pavement design is a continually changing field as a result of the dynamic interactions of the development of improved concepts and more and better data collection systems. In early stages of highway and airport development, pavement design was a rule-of-thumb procedure based on past experiences. With the post-World War II increase in vehicular traffic (both weight and number of vehicles), more and more attention has been directed toward fundamental concepts. Empiricism is necessary to relate fundamental concepts and empirical definitions of failure of a pavement system.

Processes involved in the approach to pavement design described in this paper are directed primarily towards providing structural adequacy of "flexible" highway pavements. Little attention is given to the task of pavement-type selection (from the two generic pavement types commonly used -- rigid and flexible). No attention is directed towards an even more basic point in the decision-making process, the determination of whether a highway should be built or whether funds should be used for other public services. This paper is devoted principally to a description of a model whereby the structural adequacy of a pavement can be assured. No attempt is made herein to describe in detail problems of pavement design as it relates to functional adequacy of the pavement system to perform its intended purposes. Some aspects of functional performance, however, which are closely related to the pavement design schema used in Kentucky will be mentioned briefly at various points in the discussion.

Pavement design has often been approached from two general but differing points of view. The practicing engineer on the firing line often approaches the task from the standpoint of pavement performance. Educators and researchers, on the other hand, often approach the problem on the basis of theoretical concepts. The structural design model presented herein is hopefully an appropriate merger of the two approaches, providing a basic and fundamental theory, applicable to many situations, to which has been wedded the many years of experience with highway pavement performance in the Commonwealth of Kentucky.

PAVEMENT DESIGN MODEL

Historically, the Kentucky asphaltic concrete thickness design method (1, 2) was based on observations of pavement performance and (or) empirical pavement deflection test data. Later correlations of empirical and theoretical deflections were obtained using the Chevron n-layered computer program furnished by Chevron Research Company as a "privileged" duplicate. This program is based upon elastic theory for which Michelow (3) assumed

1. the asphaltic pavement is a semi-infinite solid of n layers,
2. the elastic characteristics of one layer can be different from those of another,
3. the component layers are homogeneous and isotropic and are characterized by Young's modulus and Poisson's ratio,
4. a uniformly distributed load on a circular area is placed on and normal to the free surface of the pavement,
5. the interfaces between layers are rough (i.e., strains at the interface of two layers are identical), and
6. the bottom layer is a semi-infinite solid.

Subgrade Moduli -- The determination of stresses, strains, and deflections using the Chevron program required the input of elastic characteristics of the materials contained in the various layers of the pavement system. Values for the moduli of the subgrade (E_3) investigated ranged from 3,000 psi (20 MPa) to 60,000 psi (400

MPa). For the convenience of plotting the results in terms of a "CBR scale," the relationship developed by Heukelom and Foster (4) was used. The relationship is not precise but suggests that the subgrade modulus (in psi) is approximately equal to the product of the California Bearing Ratio (CBR) and 1500. A review of the literature (5) indicated that this relationship is an acceptable approximation for evaluating subgrade moduli and provides a simple and practical approach to this estimation. Heukelom and Foster's relationship is valid only for CBR values between about 2.5 and 17, and the extrapolation of the relationship into the range of CBR's above 17 is questionable but was done only for the purpose of plotting results to a CBR scale.

The Kentucky CBR test procedure differs from the ASTM procedure (6) only with regard to the duration of soaking. Because of the behavior of some Kentucky soils, CBR samples are allowed to soak until swelling has virtually ceased. Three months have been needed before some soils ceased to swell. Figure 1 (7) shows the superimposed correlation of the Kentucky CBR-modulus relationship with AASHTO Soil Support, dynamic and static CBR's, and R-values.

Moduli of Granular Base -- Testing of pavements in place by Heukelom and Klomp (8) has shown that the effective elastic moduli of granular base courses (E_2) tended to be related to the modulus of the underlying subgrade. The ratio of the base modulus to the subgrade modulus is a function of the thickness of the granular base, and in situ test results show that the range of this ratio is generally between 1.5 and 4 -- a value of 2.8 was selected in this study as being typical at a CBR of 7 (see Figure 2). Comparison of elastic analyses with 1959 Kentucky design curves and with field data (2) indicated this assumption to be reasonable. It was further

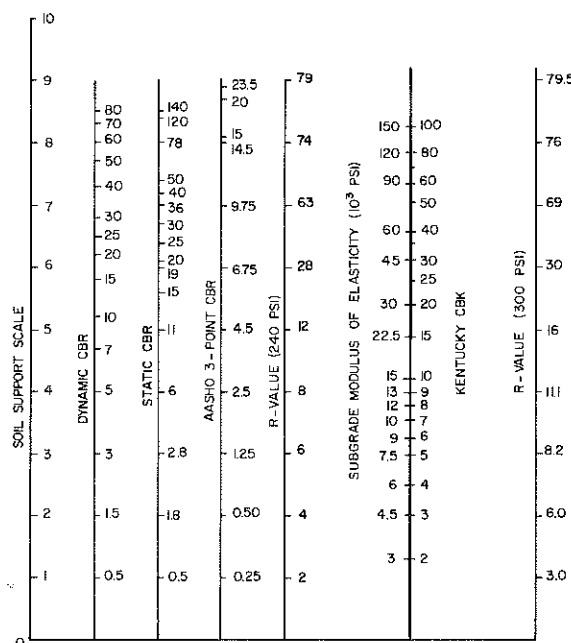


Figure 1. Relationship between Kentucky CBR - Subgrade Modulus of Elasticity and Other Soil Strength Parameters.

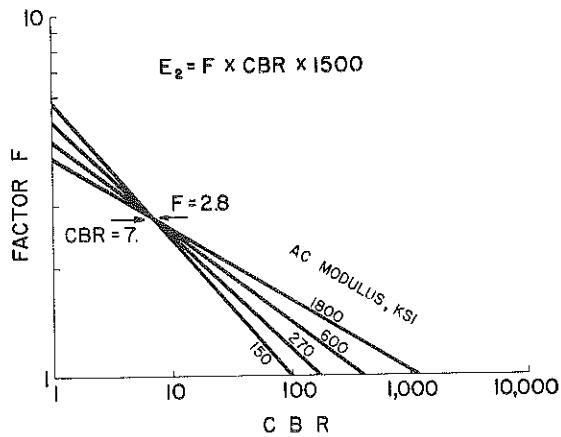


Figure 2. Relationship between the Moduli of the Subgrade and the Moduli of the Granular Base.

assumed in Figure 2 that the ratio of E_2 to E_3 would be equal to one when $E_1 = E_3$. The curves in Figure 2 were then obtained by assuming a straight-line relationship on a log-log plot. A review of the literature (9, 10) indicated that Figure 2 gives reasonable values for good quality granular bases within the range of practical design situations (CBR < 20); therefore, this figure was used throughout the analysis to relate the modulus of granular base to subgrade support values. It is noted in Figure 2 that E_2 values are a function of E_1 and E_3 only.

Moduli of Asphaltic Concrete -- The effective moduli of asphalt-bound layers depend upon the pavement temperature and time of loading. Subgrade strains are critical when the asphaltic layer is warm and its modulus of elasticity is relatively low. On the other hand, strains in the asphaltic layer are critical at lower temperatures when its modulus is relatively high. To investigate the effect of the modulus of the asphaltic layer (E_1) upon thickness requirements, a wide range of moduli (1,800,000 psi to 150,000 psi (12 GPa to 1 GPa)) was used (see Figure 2).

Poisson's Ratio -- Typical values of Poisson's ratio have been reported elsewhere (11, 12) (see Table 1). Values for the various layers used in this analysis have been added to Table 1. Figure 3 illustrates results of a sensitivity analysis of three pavement structures by varying Poisson's ratio for a given material while holding other variables constant. Within the normal range of Poisson's ratios, subgrade compressive strains are affected only to a minor degree by changes in Poisson's ratio of the subgrade. Asphaltic tensile strains and surface deflections essentially were not affected by changes in Poisson's ratios for any layers.

Loading -- A 9,000-pound (40-kN) wheel load and a tire pressure of 80 psi (550 kPa) were taken to represent the loading throughout the analysis. Figure 4 illustrates results of a sensitivity analysis for variations in asphaltic concrete and subgrade moduli, wheel loads, and tire pressures. Changes in tire pressure and Poisson's ratios were relatively insignificant.

Development of Design Curves -- Strain and deflection relationships obtained from the computer output were plotted as in Figure 5 to show the effect of the variation of asphaltic concrete and dense-graded aggregate thickness upon (1) vertical compressive subgrade strains directly beneath the load, (2) pavement surface

TABLE 1. TYPICAL VALUES OF POISSON'S RATIO (11)

MATERIAL	POISSON'S RATIO	VALUES USED IN KENTUCKY
Asphaltic concrete	0.25 to 0.35	0.40
Unstabilized granular subgrades, subbases, and bases	0.30 to 0.40	0.40
Silty subgrades	0.35 to 0.45	0.45
Clay subgrades	0.40 to 0.50	0.45
Soil cement	0.10 to 0.25	-
Dorman and Edwards (12)	0.35 to 0.45	-

deflections directly beneath the load, and (3) tangential strains at the bottom of the asphaltic concrete layers and directly beneath the load for specified values of the moduli of the subgrade (E_3) and of the asphalt-bound layer (E_1) (13). Data were read from this matrix of graphs and used to prepare graphs similar to that shown in Figure 6 - on which the axes were thickness of the asphaltic-bound layer (ordinate) and log of the ratio of the asphaltic concrete thickness to the total thickness (abscissa). From these, graphs were prepared for various asphaltic concrete moduli and for given ratios of asphaltic concrete thickness to total pavement thickness (see Figure 7). Using the relationships of Figure 7, design nomographs similar to Figure 8 were prepared.

Failure Criteria -- The criterion concerning limiting strains in the asphaltic concrete was based on interpretative analyses of other work (14). Van der Poel (15, 16) indicated that a safe limit for asphalt was in the order of 1×10^{-3} at 30 F (-1 C). Because asphaltic concrete consists of approximately 10 percent binder by volume, that fixes the safe strain level of asphaltic concrete at 30 F (-1 C) in the order of magnitude of 1×10^{-4} . Others (11, 14, 17, 18) have established (by interpretative analyses of pavements and fatigue test data) that the magnitude of asphalt strain to ensure 1×10^6 repetitions at 50 F (10 C) was 1.45×10^{-4} . Limiting values of strain (all at 50 F (10 C)) as a function of number of repetitions N of the base load (18-kip (80-kN) axleload in EAL computations) as given by Dorman and Metcalf (18) can be represented by the equation $\log \epsilon_A = -3.84 - 0.199 (\log N - 6.0)$. Other fatigue curves representing other temperatures, i.e., other values for E_1 , were derived from curves shown in Figures 9 and 10. The relationship given by Kallas (19) between temperature and E_1 provided guidance at this stage.

Some investigators suggest a fatigue diagram of the load - log N type. Fatigue theorists (20, 21, 22) have suggested and shown in certain instances that a log load - log N plot is more realistic. Pell (21) used an equation of the form $N = K' (1/\epsilon_A)^c$, where c is the slope of the $\log \epsilon_A - \log N$ plot and K' is a constant. Pell, Deacon (20), and others have suggested that the value of c lies between 5.5 and 6.5 and is a function of the modulus of elasticity of the asphaltic concrete. Pell's work further suggested that the family of curves relating $\log \epsilon_A$ to $\log N$ for different E_1 values is parallel. The use of such a relation in this study produced such irrational results (as E_1 decreased, the total pavement thickness decreased) that an alternative relationship was sought.

Plotting (to a log-log scale) the 9-kip (40-kN) tensile strain versus the tensile stress at the bottom of the asphaltic layer, it was noted that, for a given E_1 , the curves depicting structural influences appeared to converge to a single point near a strain of 2×10^{-3} . Extrapolating Dorman and Metcalf's data (18), represented by the equation given above, to a value of $N = 1$, the asphaltic tensile strain was found to be 2.24×10^{-3} . That strain was thus taken to be the limiting or critical asphaltic tensile strain for a single application of a 9-kip (40-kN) wheel load. Constructing lines tangent to the strain-stress curves at a strain of 2.24×10^{-3} , moduli lines representing the limiting relations for asphaltic strain-stress - independent of structural influences - were obtained. The stress-strain ratios shown in Figure 9 are in terms of bulk moduli ($E_1 = 0.6 K_1$, where K_1 is the bulk modulus).

For a total pavement thickness consisting of 33 percent asphaltic concrete thickness (with a modulus of 480 ksi (3.31 MPa), typical of pavements in Kentucky), it was observed that the tensile strain at the bottom of the bound layer for a CBR of 7 and total thickness of 23 inches (584 mm) (control pavement) was 1.490×10^{-4} . The traffic associated with that control point was 8×10^6 EAL's. In Figure 9, a line drawn perpendicular to the line for an asphaltic concrete modulus of 480 ksi (3.31 MPa), as determined above, at a strain of 1.490×10^{-4} intersected the other asphaltic moduli lines at strains that were assumed to be critical strains at 8×10^6 EAL's. Based on a straight-line variation between $\log \epsilon_A$ and $\log N$, the curves shown in Figure 10 were obtained as representing critical asphaltic concrete strains.

The limiting asphaltic stress-strain curves shown in Figure 9 are shown again in Figure 11. For any given modulus of asphaltic concrete, the limiting strain for a single application of a catastrophic load [EAL = $N(1.25)^{P-18}$, where P is the axleload in kips (13)] is taken to be 2.24×10^{-3} . As shown in Figure 10, another known

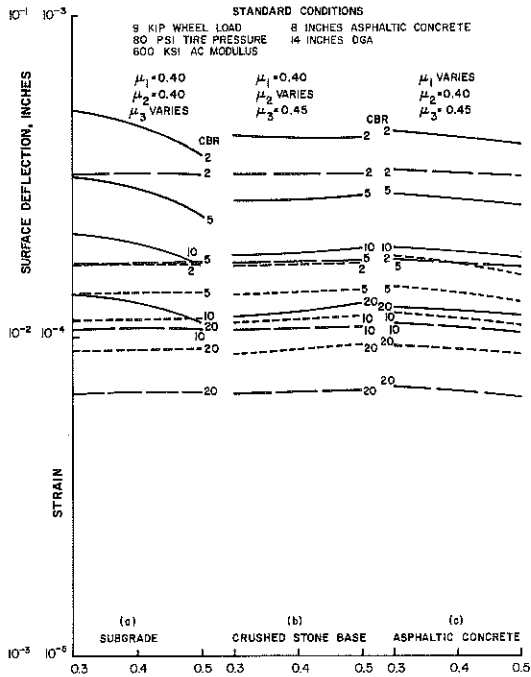
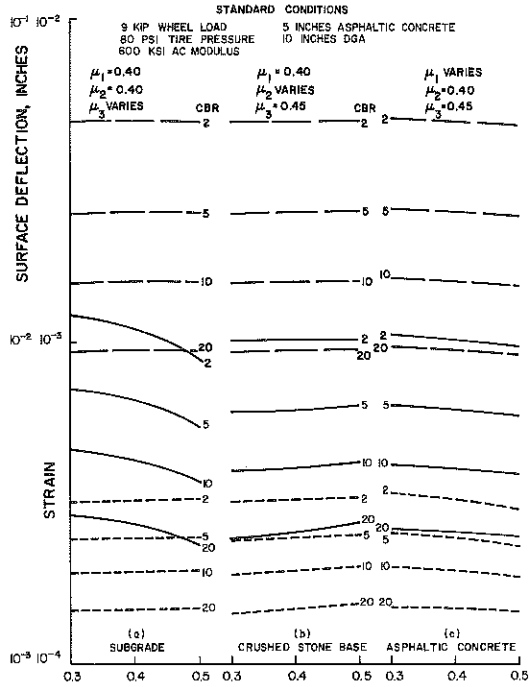
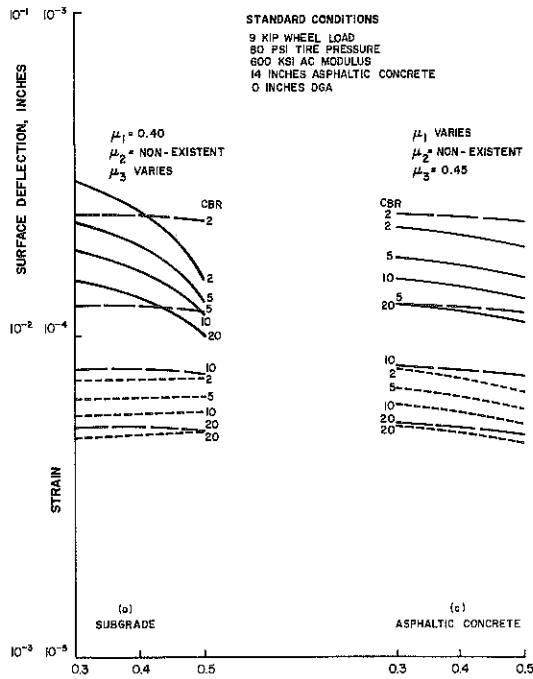
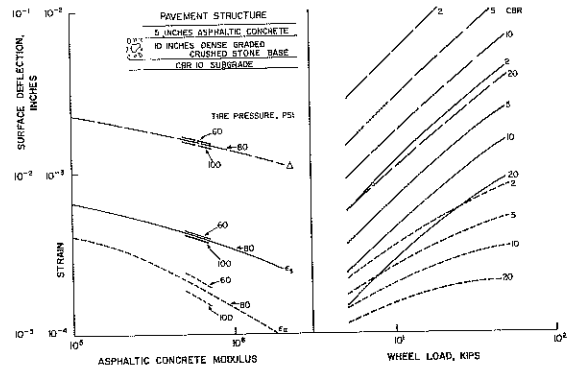
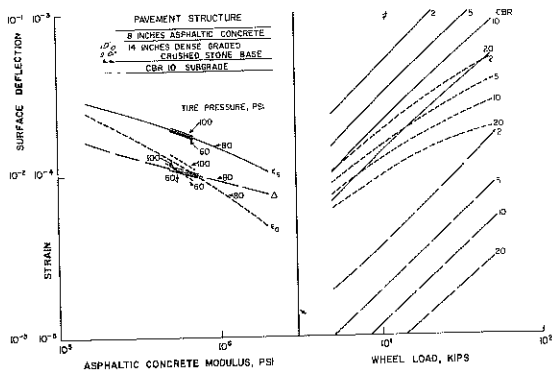


Figure 3. Influence of Variations of Poisson's Ratio upon Surface Deflection, Tensile Strain of the Asphaltic Concrete, and Vertical Compressive Subgrade Strain.

point of limiting strain falls on the line perpendicular to the stress-strain curves for 8×10^6 repetitions. Based on a logarithmic scale between these two points, lines of equal numbers of repetitions shown in Figure 11 were obtained. The limiting asphaltic concrete tensile strain for any combination of number of repetitions and modulus of asphaltic concrete can be read from curves shown in Figure 11 and are the same as those shown in Figure 9. The curves shown in Figure 11 converge to a common strain value at $N = 1$. That is a unique feature in the development of this schema. The convergence allows stress to be proportionalized according to modulus when a limiting catastrophic strain is respected.

It was observed from computations and analysis (13) that the vertical strain at the top of the subgrade, ϵ_s , for the control

pavement (CBR 7, 23-inch (584-mm) total pavement thickness; i.e., 7.7 inches (196 mm) of asphaltic concrete and 15.3 inches (388 mm) of crushed stone base) was 2.400×10^{-4} for 8×10^6 18-kip (80-kN) axleloads (9-kip (40-kN) wheel loads) would provide a high degree of assurance against rutting; that value was thus respected as controlling at 8×10^6 repetitions and a wheel load of 9 kips (40 kN). Analysis of elastic theory computations throughout a spectrum of pavement structures resulted in the curve shown in Figure 12 (13). Figure 13 was then prepared and can be used to determine the limiting vertical strains at the top of the subgrade for various equivalent single wheel loads and thus for various values of accumulative EAL's.



- - - SURFACE DEFLECTION
 ——— VERTICAL COMPRESSIVE SUBGRADE STRAIN
 - - - ASPHALTIC CONCRETE TENSILE STRAIN

Figure 4. Changes in Surface Deflections, Tensile Strains of the Asphaltic Concrete, and Vertical Compressive Subgrade Strains Caused by Variations in Asphaltic Concrete Moduli, Tire Pressures, and Wheel Loads.

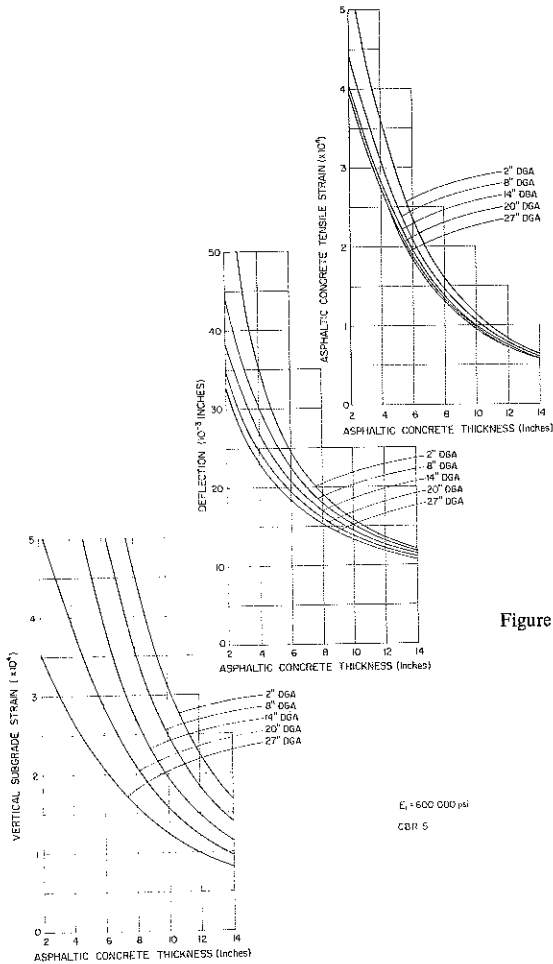
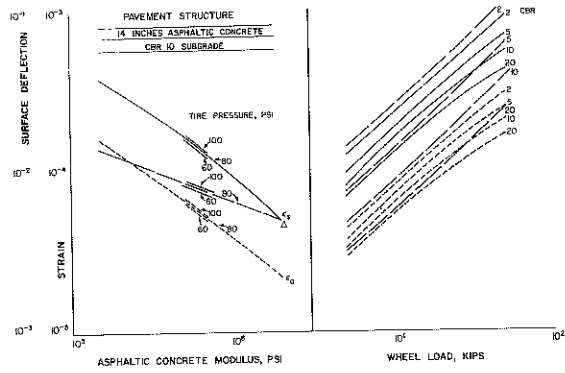


Figure 5. Influence of Variations in Asphaltic Concrete Thickness and Crushed Stone Base (DGA) Thickness upon Surface Deflection, Tensile Strain of the Asphaltic Concrete, and Vertical Compressive Subgrade Strain.

Figure 6. Typical Graph Illustrating the Relationship of Asphaltic Concrete Thickness versus Percent of Asphaltic Concrete Thickness of the Total Thickness and the Effect upon Vertical Compressive Subgrade Strain for Five CBR's.

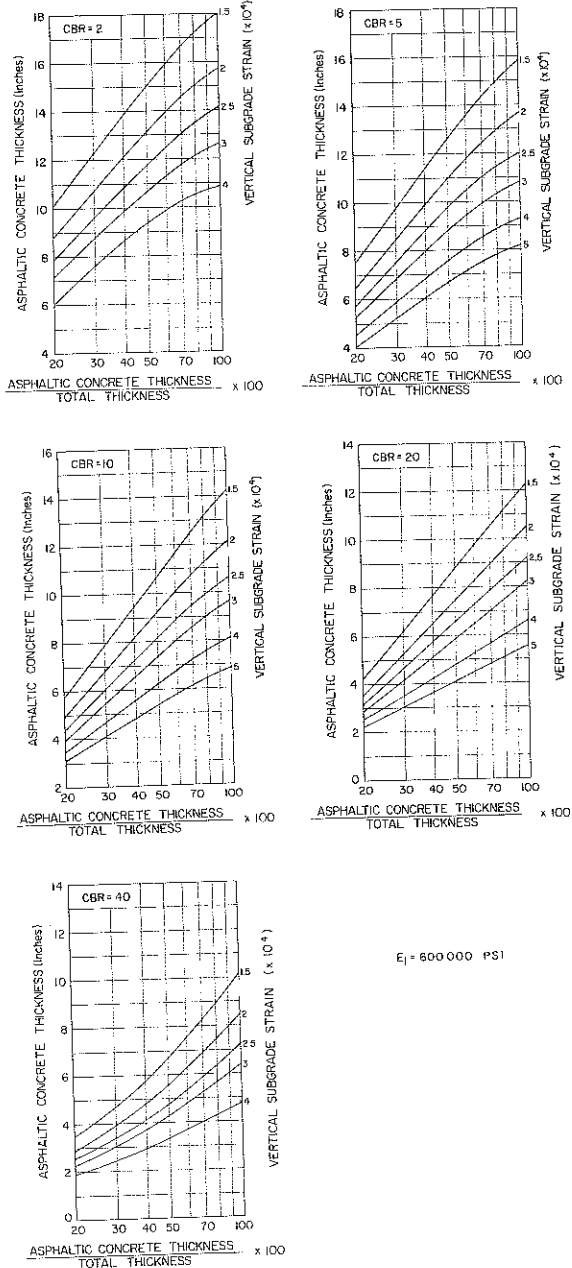


Figure 8. Nomograph for Analysis of Vertical Compressive Strains at the Top of the Subgrade and Tensile Strains at the Bottom of the Asphaltic Concrete Layer Comprising 33 Percent of the Total Pavement Thickness.

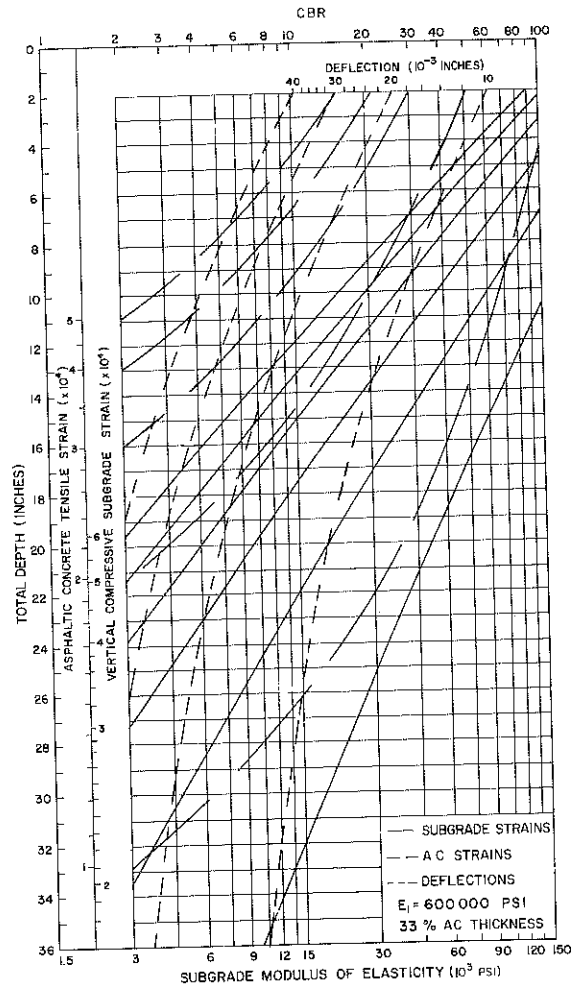
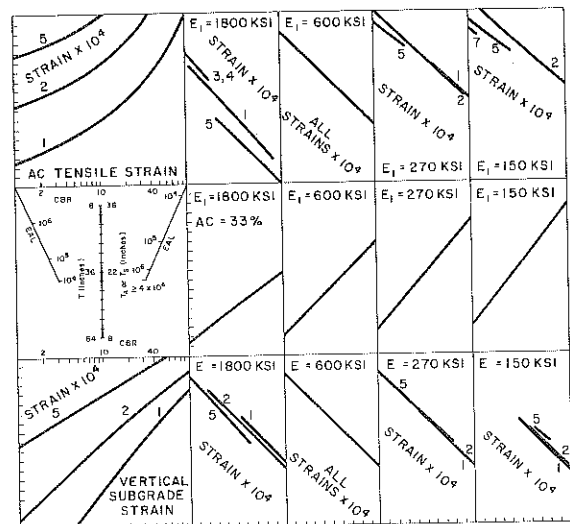


Figure 7. Total Thickness versus Kentucky CBR and Subgrade Modulus of Elasticity, Illustrating the Change in Thickness for Constant Strain and Deflection Values.



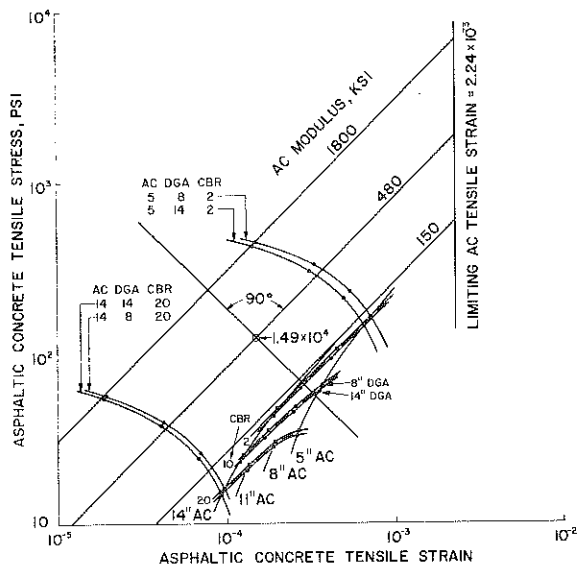


Figure 9. Asphaltic Tensile Strains - Stresses for Various Structures, CBR's, and Asphaltic Concrete Moduli of Elasticity.

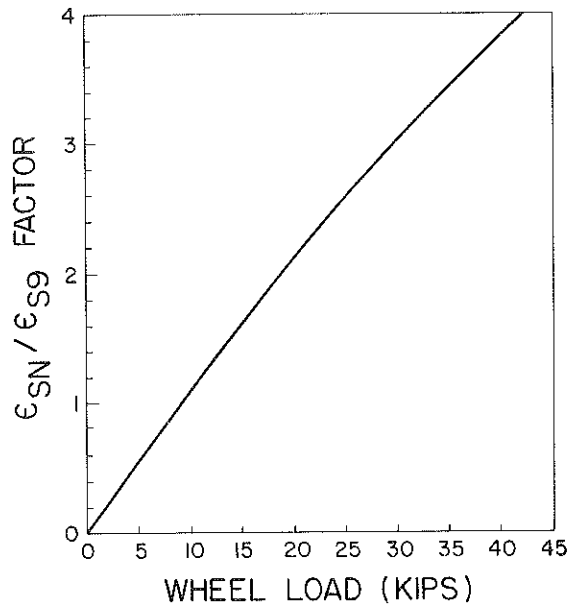


Figure 12. Ratio of Subgrade Strain to Strain under a 9-kip (40-kN) Wheel Load as a Function Equivalent, Hypothetical Wheel Load.

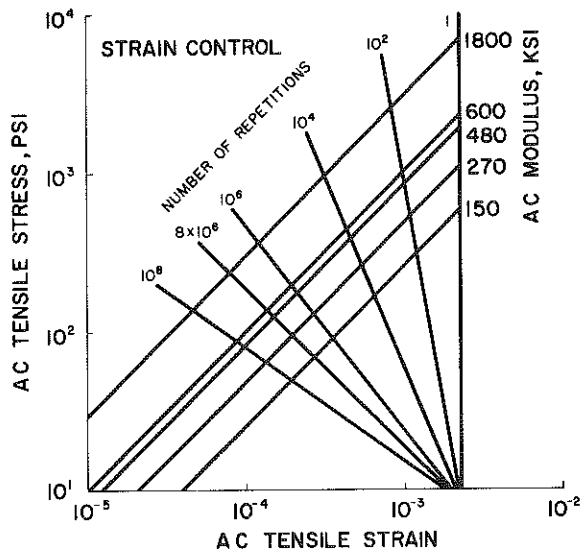


Figure 10. Asphaltic Stress - Strain Curves Showing the Application of Strain - Control Criterion.

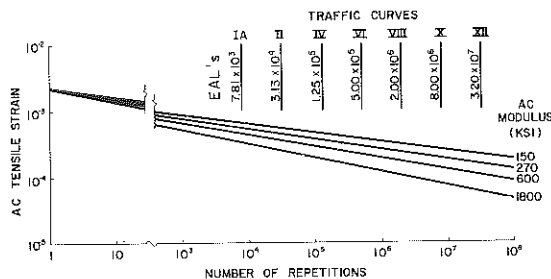


Figure 11. Limiting Asphaltic Concrete Tensile Strain as a Function of Number of Repetitions and Asphaltic Concrete Moduli of Elasticity.

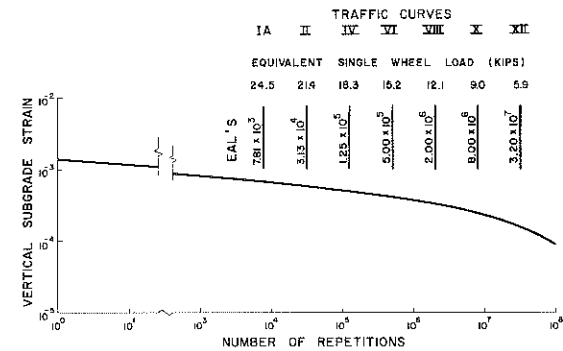


Figure 13. Limiting Subgrade Vertical Compressive Strain as a Function of Number of Repetitions and Equivalent Single Axleload.

To complete the fatigue analysis, results were plotted in terms of modulus values, layer thicknesses, etc. from influence graphs, satisfying limiting strains. That was done for the following proportions of T_1 and T_2 : $T_1 = 1/3 T$ and $T_2 = 2/3 T$, $T_1 = 1/2 T$ and $T_2 = 1/2 T$, $T_1 = 3/4 T$ and $T_2 = 1/4 T$, and $T_1 = T$ and $T_2 = 0$, where T_1 = thickness of layer 1 (asphaltic concrete), T_2 = thickness of layer 2 (granular base), and T = total pavement thickness. Graphs (Figures 8, 14, and 15) were drawn to permit continuous interpolations.

DESIGN PROCEDURE

Design Period (and Design Life) -- The design life is the time period of useful performance and is normally considered to be 20 years. Pavements may be designed for an ultimate 20-year life but be constructed in stages. Low-class roads may be designed in stages or merely designed for a proportionately shorter life. Usually it will not be practical to design pavements for low-class roads to last 20 years. Economic analysis or limitations of funds may dictate the design period.

Traffic Volumes -- Traffic volumes are normally forecast in connection with needs studies and in the planning stages for all new routes and for major improvements of existing routes. Whereas

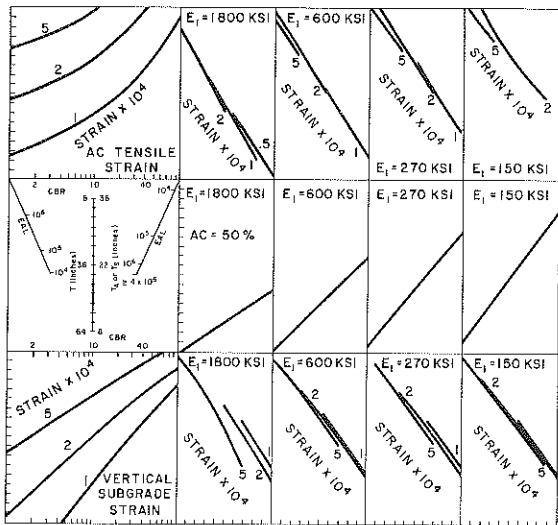


Figure 14. Nomograph for Analysis of Vertical Compressive Strains at the Top of the Subgrade and Tensile Strains at the Bottom of the Asphaltic Concrete Layer Comprising 50 percent of the Total Pavement Thickness.

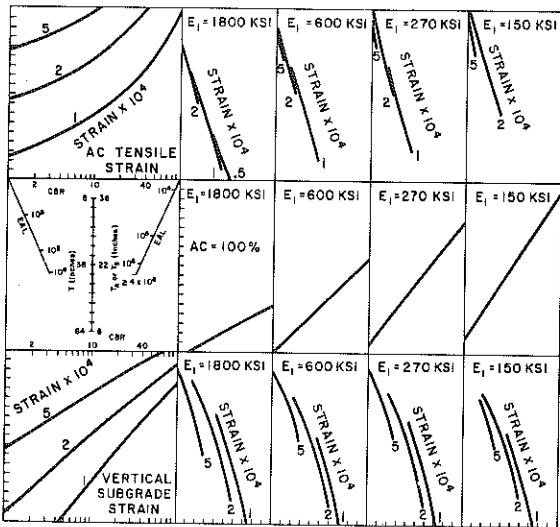


Figure 15. Nomograph for Analysis of Vertical Compressive Strains at the Top of the Subgrade and Tensile Strains at the Bottom of the Asphaltic Concrete Layer Comprising 100 Percent of the Total Pavement Thickness.

anticipated traffic volume is an important consideration in geometric design, the composition of the traffic in terms of axle weights, classifications, and lane distributions is essential to the structural design of the pavement. Traffic volumes used for EAL computations should, therefore, be reconciled with other planning forecasts of traffic. The design life of the pavement may differ from the geometric design period.

It may be possible to estimate the AADT for each calendar year; otherwise, a normal growth curve must be assumed. In the absence of specific guiding information, a constant yearly increase

typified by the compound interest equation may suffice:

$$AADT_n = AADT_1(1 + i)^n \quad 1$$

where $AADT_n$ = AADT in the nth year,
 $AADT_1$ = the beginning AADT,
 i = yearly growth factor, and
 n = number of years from the beginning.

Thus, the AADT for each year may be calculated and then summed through n years; or an "effective" AADT may be calculated as $(AADT_n + AADT_1)/2$, which, when multiplied by the number of years, yields a cursory estimate of the total design-life traffic.

In Kentucky, traffic-flow maps have been published approximately every 2 years. Thus, reasonable projections can be made from trends -- that is, until the capacity of a facility is reached. The growth is then zero and pavement damage essentially would be accumulated at a fixed rate. The only significant change would result from raising of permissible axleload and/or gross vehicle weight limits and/or changes in vehicle types.

Classification volumes can be estimated from Figure 16. These trends were developed from volume and classification data (23, 24, 25) for Kentucky four- and six-lane interstate highways for the period 1963 through 1975. Two-directional AADT were converted to one-directional AADT, which then can be combined with lane-distribution data for more detailed analysis. Solid lines represent the range of data and dashed lines are extrapolations. Figure 17 illustrates another representation of W-4 data. Note the steady decline of C3A and C4A vehicles and the steady increase of C5A vehicles.

The Highway Capacity Manual (26) defines level of service in terms of traffic flow, volume, and capacity of passenger automobiles. According to these definitions, 1976 Kentucky hourly traffic counts by classification and by lane distribution were grouped and summed for a six-lane highway. Percentages in a given lane were calculated for each vehicle classification and summarized in Figure 18. These counts were made in a metropolitan area near Cincinnati, Ohio. The nighttime data supported a service level A rating and a B rating for midday and early evening. During peak hours, the morning traffic to and evening traffic from Cincinnati had a rating of D; the other directions had a rating of C. Figure 19 contains a similar summary for four-lane highways in Kentucky. Figures 18 and 19 could also be modified to account for variations in truck volumes, percent grade, and grade length (26).

Traffic classifications and volumes may be significantly different from one area to another (24). Kentucky studies (23, 24) have shown that truck traffic is a function of direction (modified by local service use in metropolitan areas) and geographical and associated socio-economic factors -- such as recreational facilities, coal-mine areas, and agriculture. Equally as significant are changes in the legal maximum allowable gross weight, invariably leading to an increase in the percentages of the larger combination vehicles. Tri-tandem axle configurations have also been permitted. Since 1973, the single-unit four-axle truck has not only appeared but has also increased in size and load capacity -- resulting in accelerated fatigue failures. Oversized tires also have permitted increased legal gross weights.

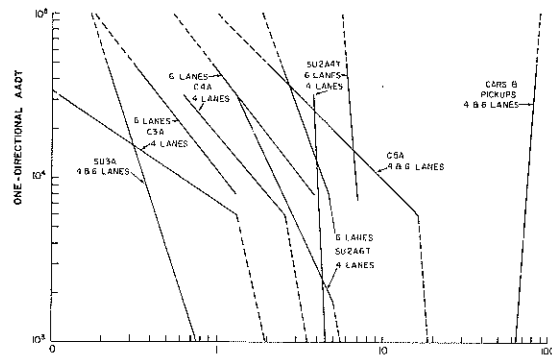


Figure 16. Vehicle Classifications as a Function of One-Directional AADT.

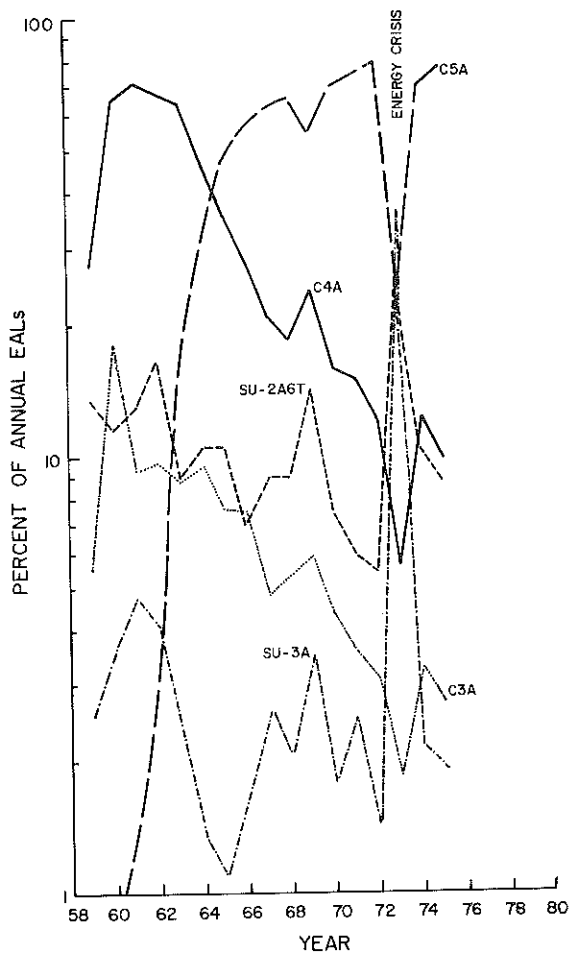


Figure 17. Changes in Vehicle Classifications with Time Expressed as Percentage of Annual EALs.

Axleload Damage Factors -- The relationship between repetitions and single and tandem axleloads is given in Figure 20. Kentucky has chosen the 18-kip (80-kN) single axleload as the reference single axleload with a damage factor of 1.0. The relative damage produced by some other axleload is expressed as

$$\text{Damage Factor} = N_p/N_B \quad 2$$

where N_p = number of repetitions of single or tandem axleload of magnitude P, and

$$N_B = \text{number of repetitions of an 18-kip (80-kN) single axleload or a 34-kip (151-kN) tandem axleload.} \quad 3$$

Every year since 1959, weight and classification data have been published in "W-4" tables. Equivalent 18-kip (80-kN) damage factors were assigned according to

$$f = (1.2504)^{P-18} \text{ for single axleloads} \quad 3$$

and

$$f = (1.1254)^{P-34} \text{ for tandem axleloads,} \quad 4$$

where f = damage equivalence factor and P = axleload in kips. Average EAL's per vehicle for various vehicle types are shown in Table 2. Such values were obtained from the ratio of total equivalent damage to total number of vehicles in the classification.

Kentucky experience (13) related 8×10^6 repetitions of an 18-kip (80-kN) axleload with an asphaltic concrete tensile strain of 1.49×10^{-4} (Figures 9-11) and a vertical compressive subgrade strain of 2.40×10^{-4} (Figure 13). The curve of Figure 12 is an average of many curves, each for a different pavement structure, which are a function of wheel load and vertical compressive subgrade strain. Within the normal range of wheel loads of 20 kips (89 kN) or less (single axleloads of 40 kips (178 kN) or less), the difference between the curves was negligible but became more prominent as the loads increased. Thus, for the normal range of highway axleloads, Figures 12 and 13 are realistic.

Strain Criteria -- Figures 11 and 13 provide strain criteria for repetitions of 18-kip (80-kN) EAL's. These criteria are used as limiting strains for input to the Kentucky CBR-thickness graphs (Figure 7) and the thickness design nomographs (Figure 8). Nomographs (such as Figures 8, 14, and 15) were developed for four percentages of asphaltic concrete of the total thickness (33, 50, 75, and 100).

For Kentucky conditions, the strain criteria used were results of observation and experience. Subgrade conditions generally have been critical and asphalt fatigue has been minor and secondary to the subgrade problems.

Design CBR -- CBR test values reflect the supporting strength of soil. Moreover, the test procedure intentionally conditions the soil -- by soaking -- to reflect its least or minimum supporting strength; this is presumed to be representative of soil strength during sustained wet seasons when the ground is saturated, or nearly so. At other times, the soil may be much stronger, and pavements

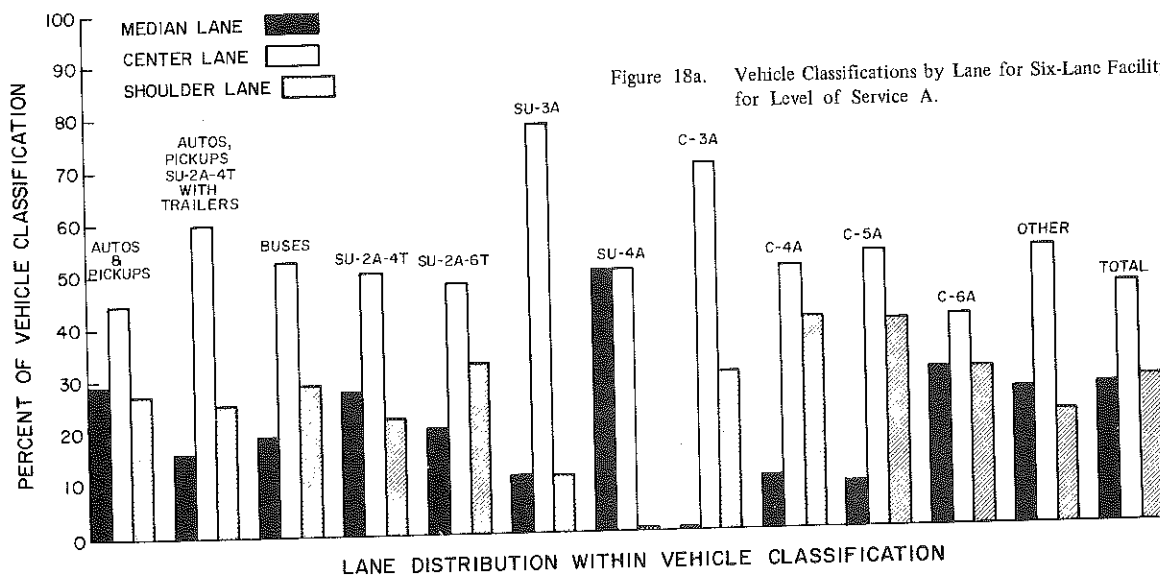
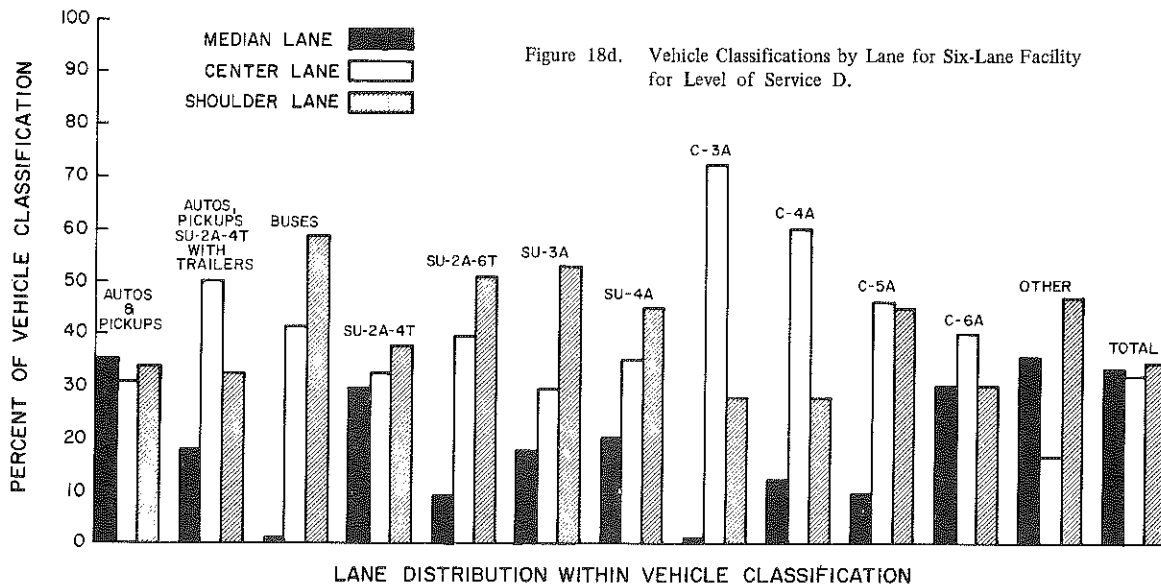
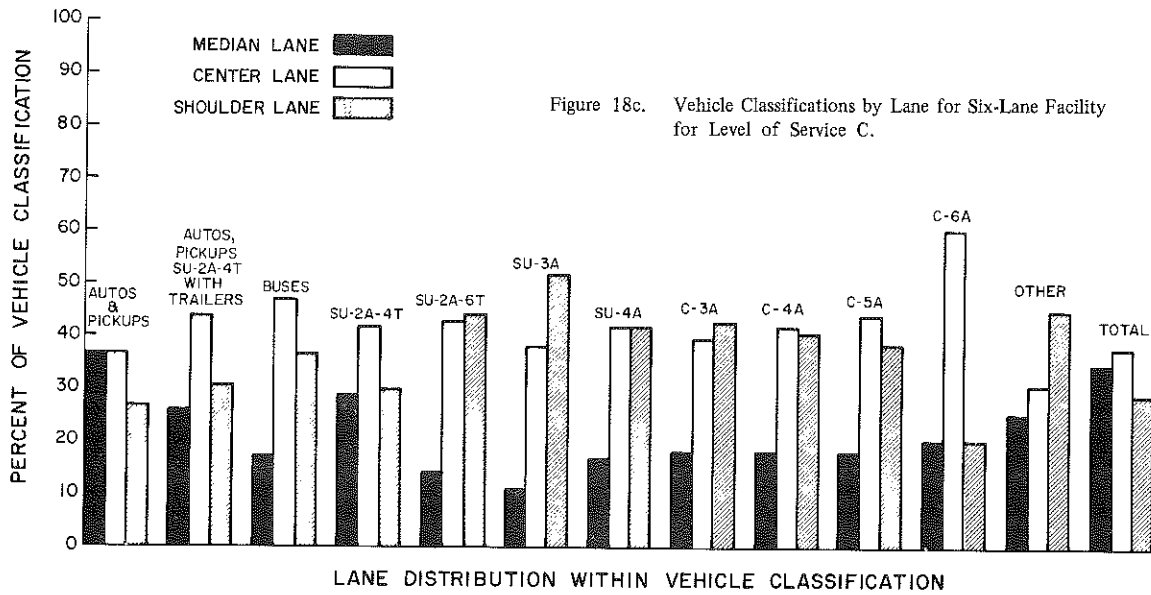
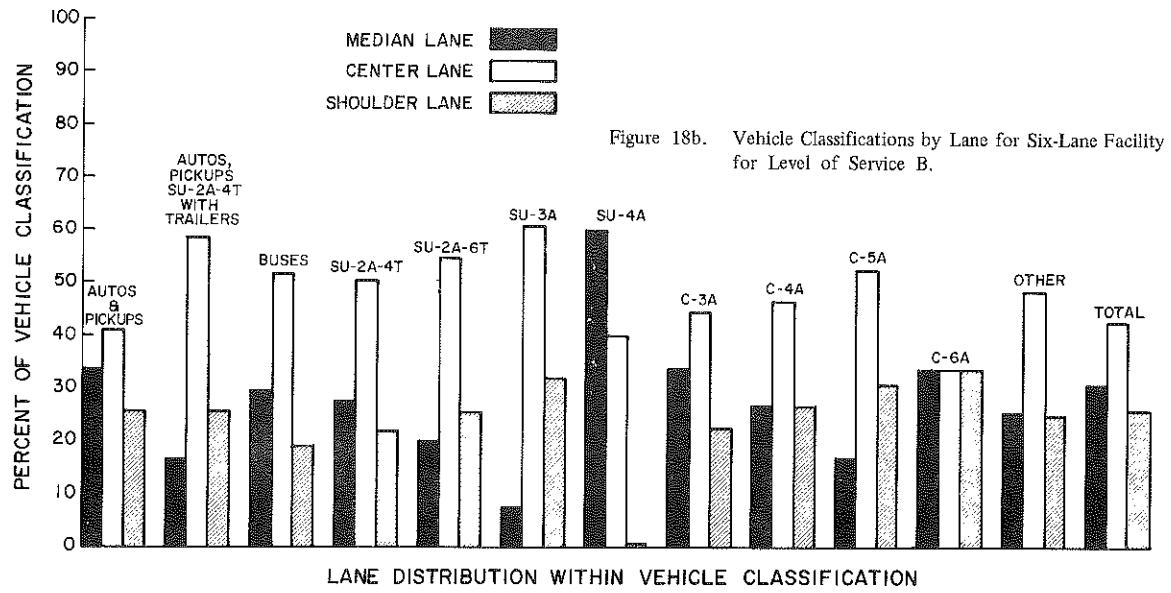
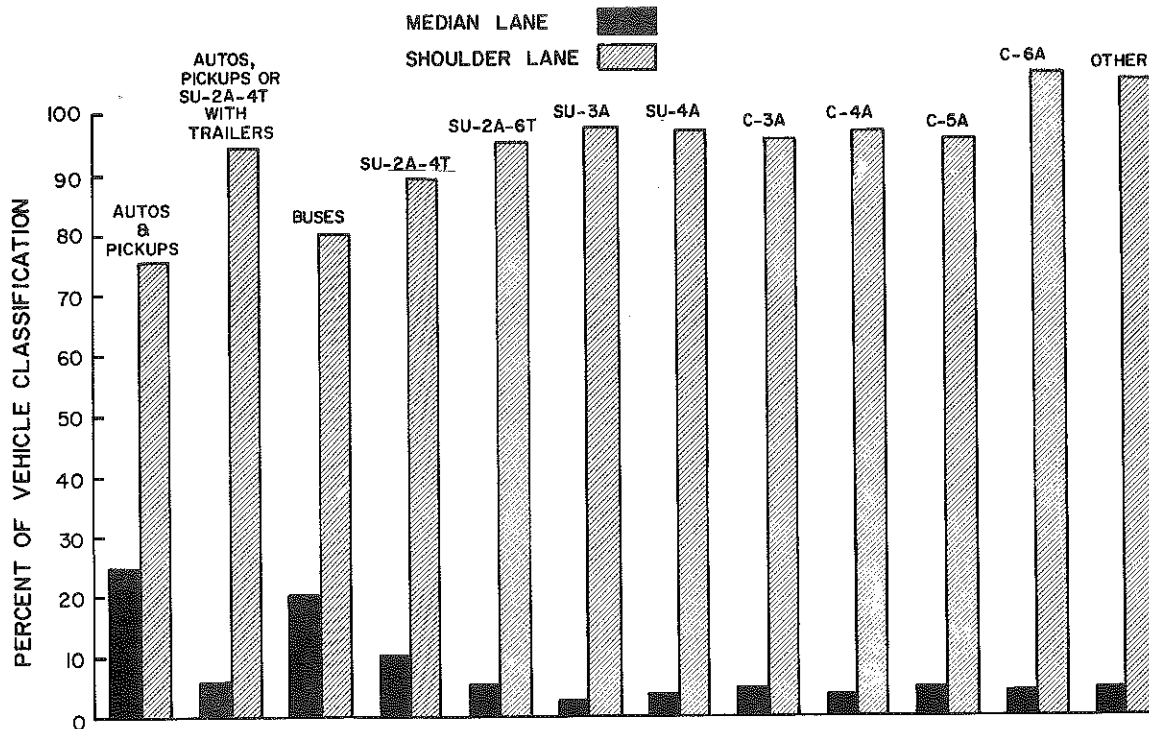


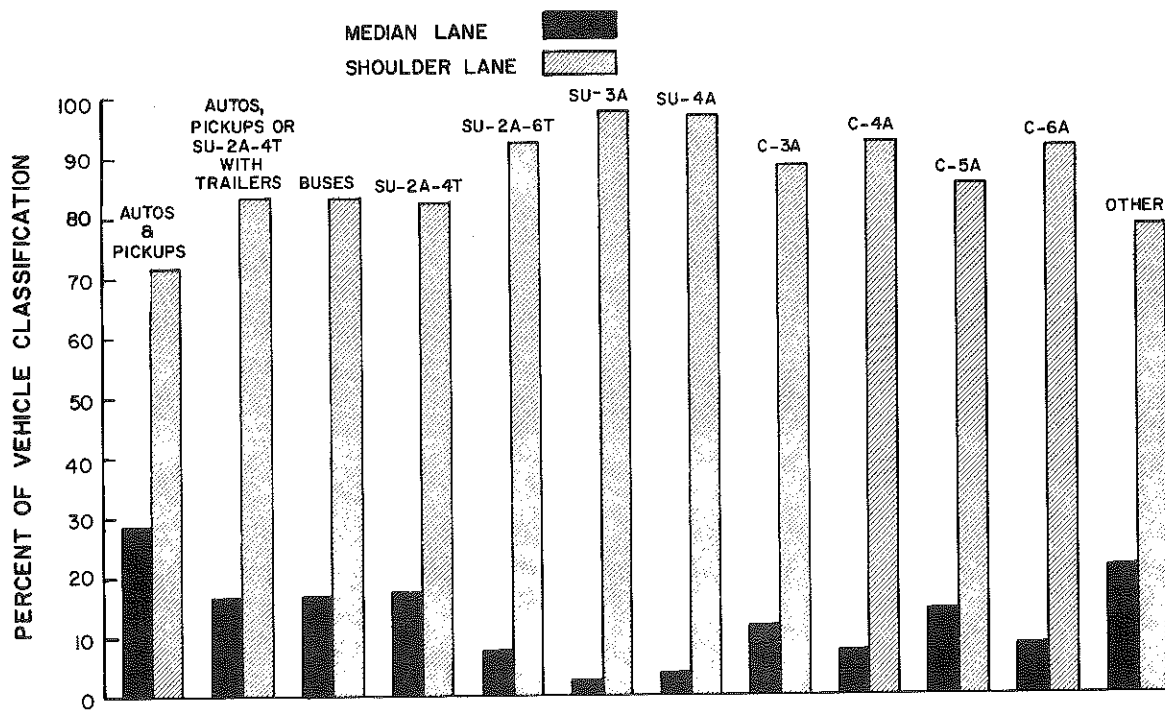
Figure 18a. Vehicle Classifications by Lane for Six-Lane Facility for Level of Service A.





LANE DISTRIBUTION WITHIN VEHICLE CLASSIFICATION

Figure 19a. Vehicle Classifications by Lane for Four-Lane Facility for Level of Service A.



LANE DISTRIBUTION WITHIN VEHICLE CLASSIFICATION

Figure 19b. Vehicle Classifications by Lane for Four-Lane Facility for Level of Service B.

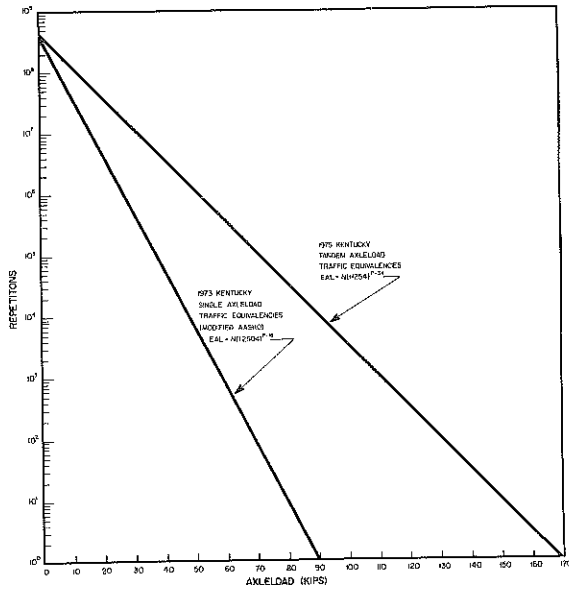


Figure 20. Repetitions of Single and Tandem Axleloads versus Axleload.

thereon would be capable of withstanding heavier loads. If pavements were not designed for the minimum capabilities of the foundation soil, it would be necessary to impose further restrictions seasonally on axleloads to prevent premature failures.

The CBR value does not assure immunity against frost heave. However, CBR does have a compensating effect in the design of the pavement structure. Greater pavement thicknesses are required for low-CBR soils than for high-CBR soils. It is usually the low-CBR soils which are more sensitive to frost. Usually, it will not be economical or practical to eliminate frost-sensitive soils. Very high-type pavements are usually of such thickness that the supporting soil lies below the freezing line. Of course, this is not true for thinner pavements. Therefore, the structure providing the greatest template depth is preferred where frost-sensitive soils are encountered. Pavements less than 6 inches (150 mm) in thickness or having less than 4 inches (100 mm) of asphaltic concrete should be regarded dubiously from this point of view. Rock subgrade is recommended where suitable materials are economically available.

Ideally, analysis of soil surveys and exploration reports will not only assure rejection of soils ineligible for service as subgrade (foundation under pavements) but may even be reserved from cuts and used as the final lift throughout a section of roadway. However, because of the necessity of stockpiling and double handling, this may not always prove to be economical. It is recommended that the designer consider comparative costs of design alternatives and exercise due judgment.

Soil surveys may indicate wide variations in CBR's along the length of a specific project route. It is presumed that adequate pavement thicknesses will be provided throughout the project. The designer must, therefore, consider the contiguity of the soils and perhaps sectionalize the project according to minimum CBR's. An

TABLE 2. UNIT EAL'S FOR SELECTED VEHICLE TYPES

VEHICLE TYPE	NUMBER OF WEIGHED VEHICLES	TOTAL EQUIVALENT 18-KIP(80-KN) LOADS	AVERAGE 18-KIP (80-KN) PER VEHICLE
SU2A4T; Single-Unit, 2 Axles, 4 Tires	8,564	518.2	0.0605
SU2A6T; single-Unit, 2 Axles, 6 Tires	19,058	5,627.6	0.2953
SU3A; Single-Unit, 3 Axles	2,848	1,818.7	0.6386
C3A; Combination, 3 Axles	4,701	2,986.7	0.6353
C4A; Combination, 4 Axles	15,217	11,434.7	0.7514
C5A; Combination, 5 Axis	21,673	13,583.1	0.6267
Automobiles and Pickups	--	--	0.0501

NOTE: Data from Kentucky W-4 tables for years 1959-1973, except for Automobiles and Pickups. Average EAL per Automobile and Pickup obtained from Division of Research data.

analog graph may be helpful. The designer must respect all minimum CBR's or else some sections of pavement will be "underdesigned," "overdesigned" must be admitted as a natural consequence therefrom. Here again, subjective judgment is admissible. For example, consider two high-CBR sections having relatively long lengths separated by an intervening short section having a low CBR. The designer is privileged to decide whether to require the low-CBR section to be "upgraded" to the same quality as the abutting high-CBR sections or to make a separate design for the low-CBR section. Of course, the designer should consider relative economics of the two alternatives, but he may also consider continuity and uniformity of pavement section and construction control as pertinent factors. Usually, it will be found impractical to vary the design thickness within short distances.

Soils having CBR's of less than 3 should normally be considered ineligible and unsuitable for use as pavement foundations. However, such soils might be utilized after upgrading or with special pavement designs.

Rutting Criteria -- The lowest traffic volume used for pavement design in Kentucky (allowable-defect design) has one 18-kip (80-kN) EAL per day (7,300 EAL's in 20 years). For this low category of traffic, the minimum thickness is controlled by the asphaltic tensile strain criteria. The concept employed is (1) generally, the geometrics for these rural roads will restrict the speed such that hydroplaning should not be a problem and (2) maximum rutting should be allowed so long as the integrity of the pavement is retained.

A second criteria was that rutting should be a minimum for any pavement having a design 18-kip (80-kN) EAL of 4×10^6 or more in 20 years (no-defect design for high-type, high-volume facilities). The method for adjusting the allowable rutting between these two EAL values is shown in Figure 21. The thickness obtained from the nomographs (Figures 8, 14, and 15) can be adjusted for rutting by Figure 21 or by the nomograph in Figure 22.

Alternative Pavement Thicknesses --

1. If the design EAL is known, the limiting subgrade strain can be determined from the curve shown in Figure 13. Likewise, Figure 11 shows the limiting asphaltic tensile strain values. If a design is desired for an asphaltic concrete with a modulus other than the four specifically shown in Figures 8, 14, and 15, it will be necessary to know the limiting asphaltic concrete strain for each of the four modulus values so that interpolations can be made later.
2. Enter the top portion (for asphaltic strain control) of Figure 23 at the design CBR. Draw a line vertically to limiting strain values (from Figure 11) for each E_1 ; mark each point.
3. Draw horizontal lines from each of the points obtained above to the respective E_1 modulus quadrants; mark the point at the appropriate strain values.
4. From those points, draw lines vertically; and mark points on the turning lines.
5. From those points, draw lines horizontally; and read T_A values for each E_1 modulus on the thickness scale.
6. Repeat Step 2 but use the lower portion (for subgrade strain control) of Figure 23. Only one value of limiting subgrade strain is given for a fixed value of repetitions and is independent of E_1 moduli.

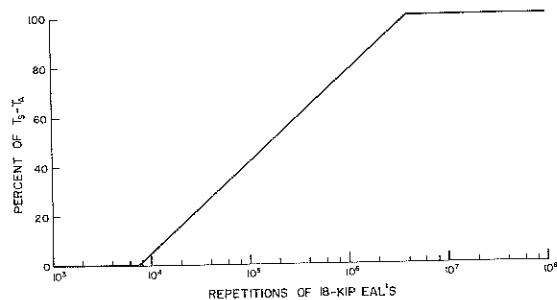


Figure 21. Adjustment of Design Thickness for Rutting as a Function of Repetitions of 18-kip (80-kN) EAL.

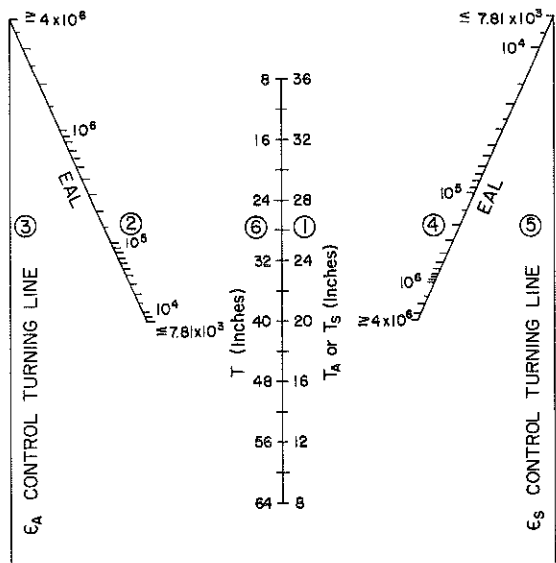


Figure 22. Nomograph to Adjust Design Thicknesses for Rutting.

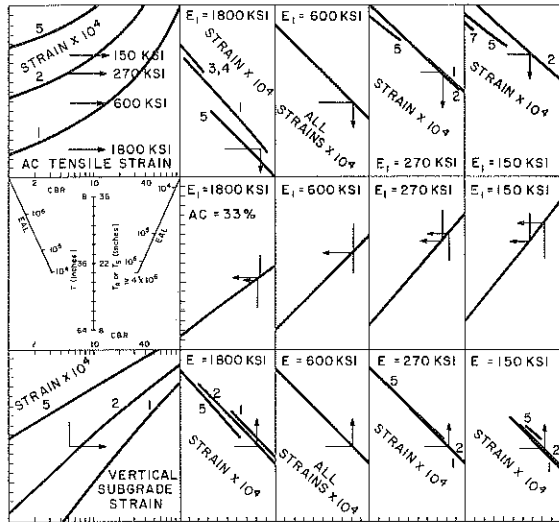


Figure 23. Illustration of Use of Figure 8.

7. Draw a horizontal line to the right through all four quadrants and locate the strain value in each quadrant.

8. Repeat Steps 4 and 5 to obtain values of T_S for each E_1 modulus.

9. Plot each design total thickness from Steps 5 and 8 (arithmetic scale) versus $\log E_1$, and fit a smooth curve to the points as shown in Figure 24.

10. Repeat Steps 1 through 8 using Figures 14 and 15.

11. From Figure 24, read the total thickness T_A for each ratio of thickness of asphaltic concrete to total thickness, and plot the resulting total thickness values (arithmetic scale) versus \log of percentage asphaltic concrete thickness as shown in Figure 25. Repeat this step using T_S from Figure 24.

12. Select from Figure 25 the final design total thickness values for T_A and T_S for the desired ratio of asphaltic concrete thickness to total thickness.

13. If the design EAL is 4×10^6 or greater, the design total thickness for each E_1 modulus is the greater of T_A and T_S . If the design EAL is 7.81×10^3 or less, the total thickness design is T_A .

14. For $7.81 \times 10^3 < \text{EAL} < 4 \times 10^6$, read from Figure 25 the total thickness (T_A For asphaltic concrete strain control) and mark on Scale 1 in Figure 26. Draw a straight line from T_A on Scale 1 through the design EAL value on Scale 2; mark the intersection point on Line 3.

15. For the desired ratio of asphaltic concrete thickness to total thickness shown in Figure 25, read the total thickness (T_S for subgrade strain control); mark on Scale 1. Draw a straight line from T_S on Scale 1 through the design EAL value on Scale 4; mark the intersection point on Line 5.

16. Connect the intersection points on Lines 3 and 5 by a straight line, and read the final adjusted design thickness on Scale 6.

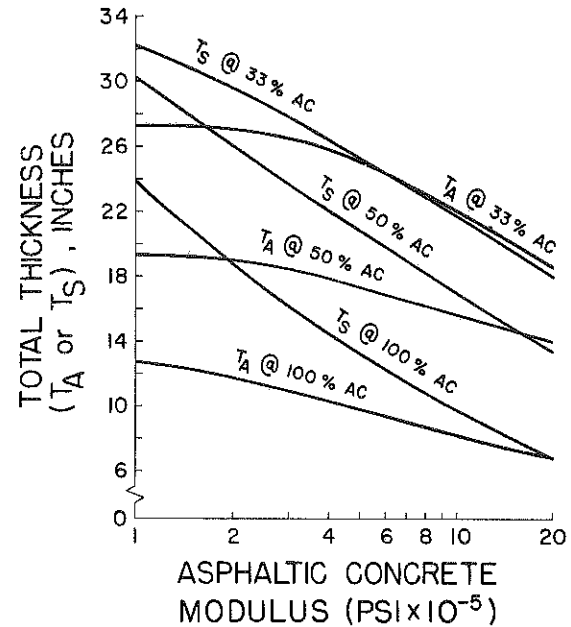


Figure 24. Total Pavement Thickness (T_A AND T_S) as a Function of Asphaltic Concrete Modulus.

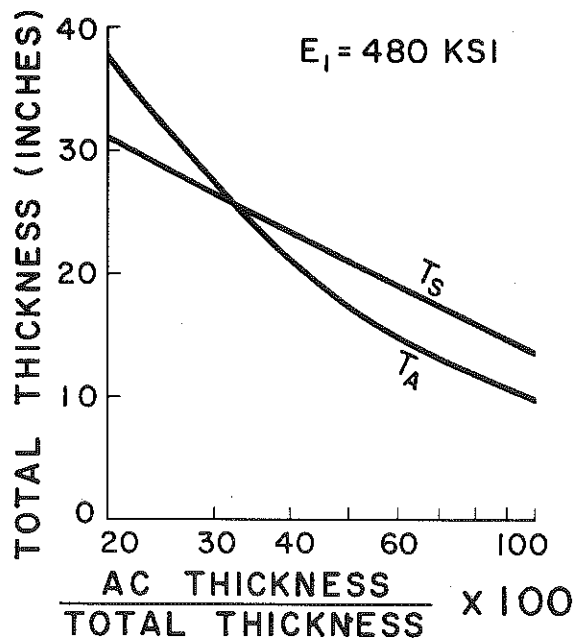


Figure 25. Total Pavement Thickness (T_A and T_S) as a Function of Ratio of Thickness of Asphaltic Concrete to Total Thickness.

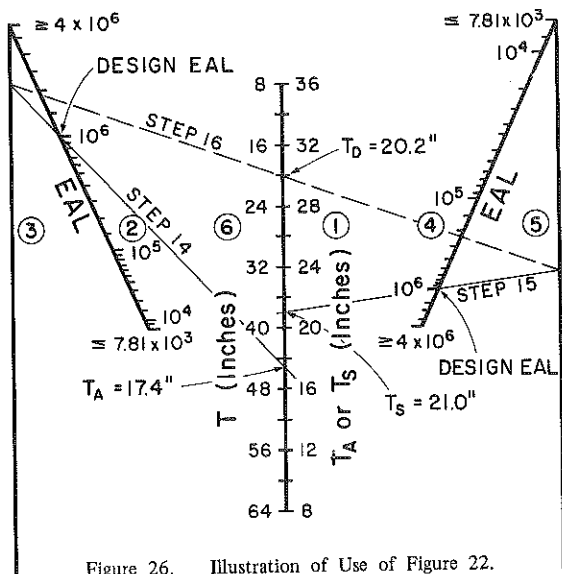


Figure 26. Illustration of Use of Figure 22.

Asphaltic Concrete Modulus of Elasticity - Generally, design systems do not account for the possible range of values of the modulus of elasticity of bituminous concrete. That has generally proved to be more than adequate because such design systems have been applied to rather limited situations in which the stiffness characterization of bituminous mixtures actually used in practice falls within a very limited range. The effective moduli of asphalt-bound layers depend on the pavement temperature and time of loading. As design systems begin to take into account to greater degrees the range of pavement temperatures and times of loading, the modulus of the bituminous concrete becomes more and more significant.

Initial and preliminary analysis of the performance of Kentucky flexible pavements (thickness being 1/3 asphaltic concrete and 2/3 crushed stone base) in comparison with theoretical computations indicates empirically that the bituminous concretes used in Kentucky typically have an apparent modulus of elasticity of about 480,000 psi (3.3 MPa) that corresponds to the modulus at 64 F (18 C) (the mean annual pavement temperature) obtained from an independent correlation between modulus and average pavement temperature (13). Weighting distributions of pavement temperature of more than 64 F (18 C) for various thicknesses of asphaltic concrete suggest that 76 F (24 C) might be considered an equivalent "design" temperature for full-depth asphaltic concrete pavements.

Designs with lesser proportions of the total thickness being asphaltic concrete might be expected to be less sensitive to rutting of the asphaltic concrete than full-depth designs. The reduced susceptibility might be considered as an increase in the effective modulus of elasticity of the asphaltic concrete. Correlating the mean pavement temperature with the modulus of elasticity (Figure 27) according to Southgate and Deen (27, 28) makes it possible to determine and plot (Figure 28) the moduli corresponding to 64 F (18 C) (thickness being 1/3 asphaltic concrete) and 70 F (23.3 C) (full-depth asphaltic concrete). Based on a straight-line relationship, the change in asphaltic concrete modulus as the temperature sensitivity to rutting varies is described as shown in Figure 28. Designs obtained by the use of modulus values shown in Figure 28 would surely perform at least equal to current designs (employing usual proportions of dense-graded aggregate base and asphaltic concrete surface courses). Other more refined weightings should be regarded as admissible.

The logic in the development of Figure 28 is supported by Figure 29 (29). All data in Figure 29 were adjusted to the reference average pavement temperature of 60 F (15.6 C). In this case, the temperature distribution through the pavement is the important factor. During the early spring season, the temperatures at the bottom and in the lower half of the asphaltic concrete are nearly

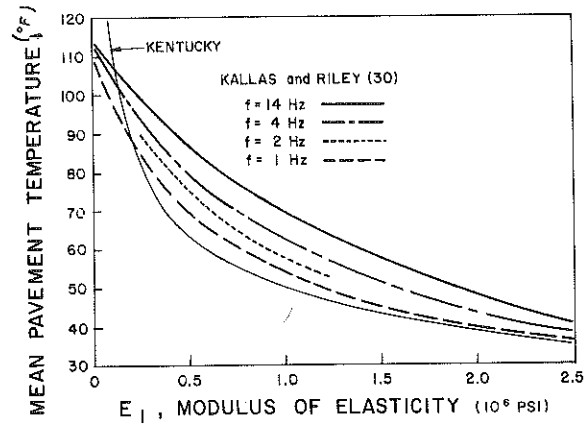


Figure 27. Asphaltic Concrete Modulus as a Function of Mean Pavement Temperature and Frequency of Loading.

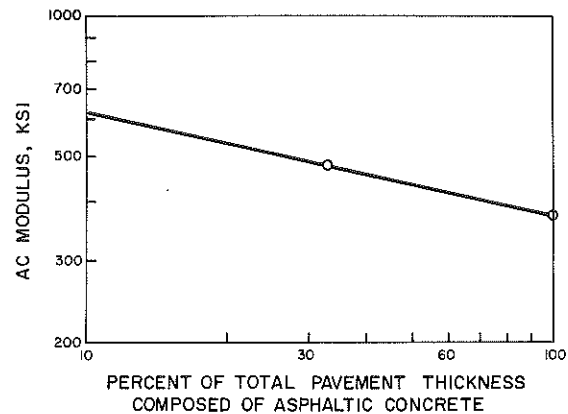


Figure 28. Weighting of Asphaltic Concrete Modulus for Ratio of Thickness of Asphaltic Concrete to Total Pavement Thickness.

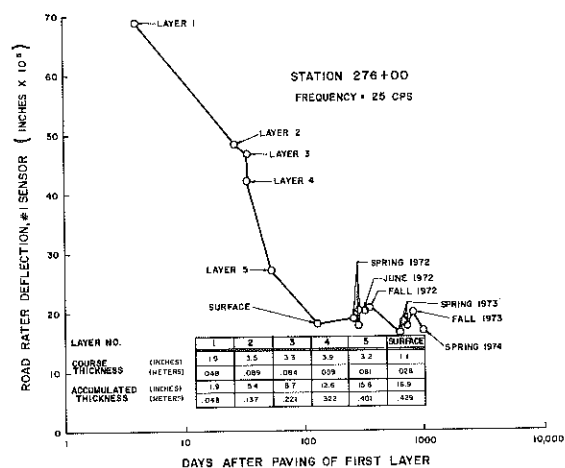


Figure 29. Road Rater Deflection Tests Illustrating Thickness, Season, and Age Effects at Reference Pavement Temperature.

the same and relatively low. These cool temperatures are primarily the results of the cool basement soil layer. As summer progresses, the pavement and basement soil temperatures increase due to the solar energy heat input over a long period. Consequently, by early fall, the lower asphaltic concrete and the basement soil temperatures are relatively warm. Thus, the average of the temperatures at the surface, mid-depth, and bottom of the asphaltic concrete layer is relatively high. Alternately, if pavement temperature distributions are converted to elastic moduli, the average or effective elastic modulus for the fall period is lower than the early summer season and markedly lower than the spring season. Recorded pavement temperatures of an experimental full-depth asphaltic concrete pavement (29) in Kentucky were converted to equivalent elastic moduli (Figure 30) using the relationship shown in Figure 27 (30, 31). Note that the pattern of moduli is approximately the same with increasing depth without regard to the total thickness. Thus, the real pavement support begins 3 to 4 inches (76 to 102 mm) below the surface, and the effective average strength increases markedly as the asphaltic concrete thickness increases.

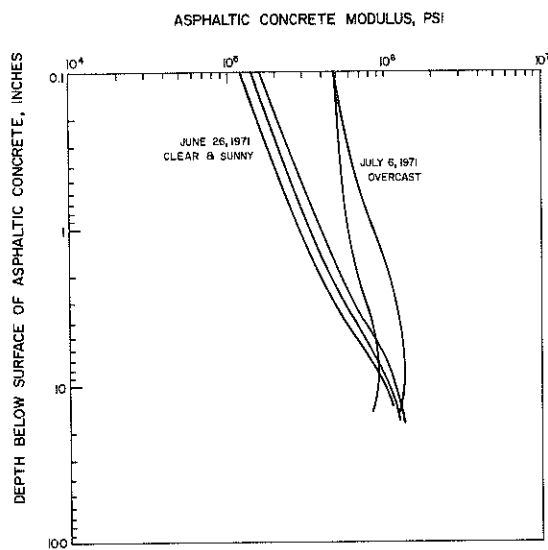


Figure 30. Asphaltic Concrete Modulus as a Function of Depth below Asphaltic Concrete Surface and Cloud Conditions.

Note in Figure 29 that the adjusted deflections for Fall 1972 and Fall 1973 are higher than their respective spring and summer values. Deflections of the spring season are critical due to increased moisture contents of the subgrades, but the higher pavement temperatures of the early fall season make the modulus of the asphaltic concrete critical for rutting, permanent deformations within the asphaltic concrete, and vertical compressive deflections and strains in the subgrade. Subgrade deflection and/or strain becomes more critical as the number of axles increases in a weight group and as the load (weight) increases. The equivalent effects of the tandem and tri-tandem axleloads occur at greater depths, respectively. Thus, modern legal axleload limits, coupled with an increasing number of axles within a weight group, dictate thicker bound layers to minimize temperature effects and for load distribution to protect the subgrade.

Simplified Thickness Design Charts -- Alternative but equivalent thickness designs can be determined easily using the simplified design charts in Figures 31-35. Each line on a chart is for a given CBR. Though each line is for a Kentucky CBR value, these are convertible to other soil support values by Figure 1. Since the design charts are mathematical solutions coupled with fatigue criteria, changes in the fatigue criteria can be made by simply relabeling the CBR lines to appropriate equivalent values.

Overlay Design System -- The design charts (Figures 31-35) can be used to design overlays and to determine when overlay construction should be scheduled in terms of accumulated EAL's. Likewise, the design charts can be used to design pavements for stage construction. For staged designs, the designer should choose the final EAL value, the CBR, and the final percentage of asphaltic concrete. For example, the CBR = 5 curve at 9×10^6 18-kip (80-kN) EAL requires a total thickness of 18 inches (457 mm) for a 75 percent asphaltic concrete structure. Thus, 4.5 inches (114 mm) of a crushed stone base would be overlaid with 13.5 inches (343 mm) of asphaltic concrete. A 67-percent design having 4.5 inches (104 mm) of crushed stone base would have a total thickness of 13.5 inches (343 mm) and would be capable of supporting 7×10^5 repetitions of an 18-kip (80-kN) axleload. If the final 9×10^6 EAL design was to be a 25-year design, 7×10^5 EAL's should last approximately 2 years. Several advantages can be readily seen. The initial financing can be reduced; but more importantly, the weakest areas in the pavement should be evident by then. Weak areas can be strengthened or repaired, and leveling provides additional material where it is required. Figure 36 illustrates approximately when another overlay is required and approximately how long that overlay should last before requiring the next overlay. The design is for an EAL level, and the time frame will shift according to the traffic volume and classification distributions. If the expected traffic rate is exceeded, the EAL's will be accumulated faster and the overlay will be required sooner, and vice versa. Thus, the need for updating EAL levels for projects is a real and essential part of pavement management and will be discussed later. Such updating permits financing to be programmed and minimized, provided the overlay is constructed prior to reaching the expected fatigue limit or design EAL level.

VERIFICATION

Axleload Damage Factors -- Figure 20 gives the Kentucky relationship of repetitions versus single and tandem axleloads.

Figure 37 illustrates three relationships of repetitions versus axleloads. The 1959 Kentucky equivalencies were similar to those used in California (32, 33). The AASHO equivalencies (7) are those associated with a structural number of 5.0 and level of serviceability of 2.5. Because of data obtained from the AASHO Test Road and the general acceptance of the subsequent thickness design system (7), the Kentucky equivalency relationship was changed in 1973 from the 10-kip (44-kN) axleload base to 18 kips (80 kN) and was made tangent to the AASHO curve at 8×10^6 repetitions of an 18-kip (80-kN) axleload.

Figure 38 illustrates separate sets of axleload-repetition lines (7) resulting from the AASHO equation,

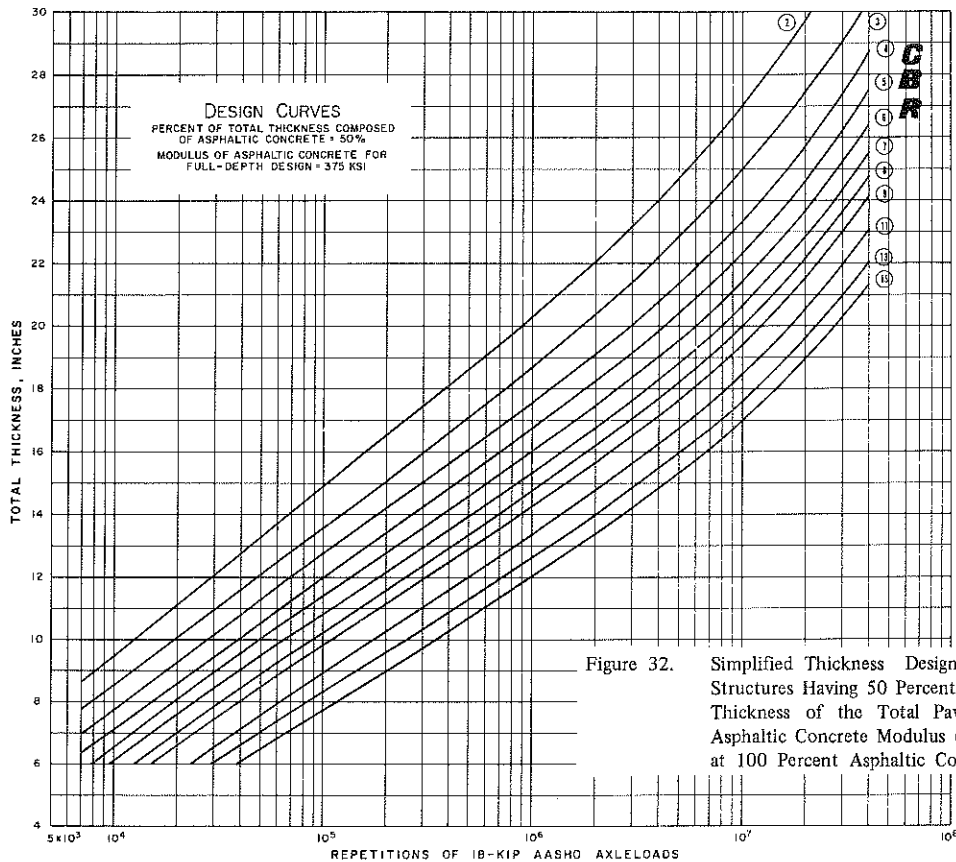
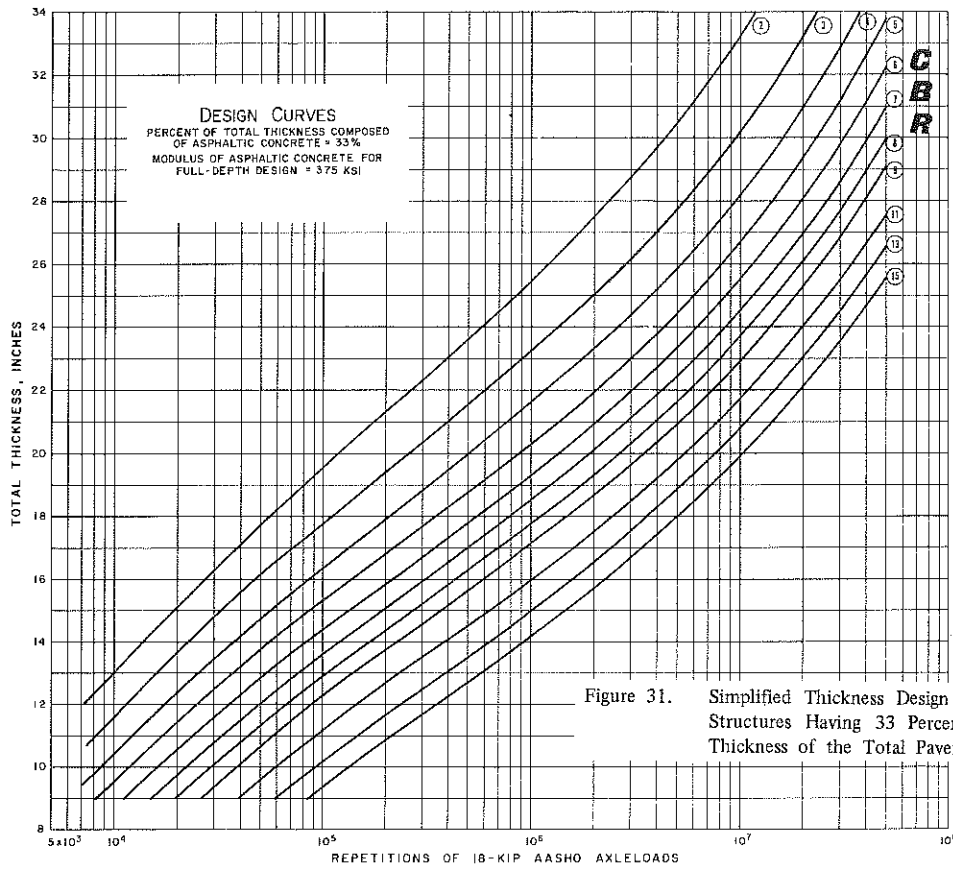
$$\log W_t = 5.93 + 9.36 \log (SN + 1) - 4.79 \log \left[\frac{L_x + L_2}{L_1} + 4.33 \log L_2 + G_t/B_x \right] \quad 5$$

where W_t = repetitions,
 L_x = axleload,
 L_1 = 1 for single axleloads and 2 for tandem axleloads,
 SN = structural number,
 P_t = terminal serviceability,
 G_t = $\log [(4.2 - P_t)/(4.2 - 1.5)]$, and
 B_x = $0.40 + [0.081(L_x + L_2)^{3.23}] / [(SN + 1)^{5.19}(L_2)^{3.23}]$.

For any given line in Figure 38, Equation 5 is used to calculate the damage factor equivalent to an 18-kip (80-kN) damage value.

The AASHO Test Road provided data for evaluating the relative damage caused by different loads on different pavement structures. Figure 38 shows the relationship of axleloads and numbers of repetitions for various structural numbers and serviceability levels. The AASHO damage factors (7), or equivalent axleload factors, are cumbersome for a designer because an iterative design procedure must be utilized if a balanced design is to be reached. The sets of AASHO damage factors represent a method of analysis of test data and can be used to compare load and repetitions with pavement structures.

The AASHO Test Road staff chose a log-log equation to fit to the Test Road data. The AASHO Test Road had a limited range



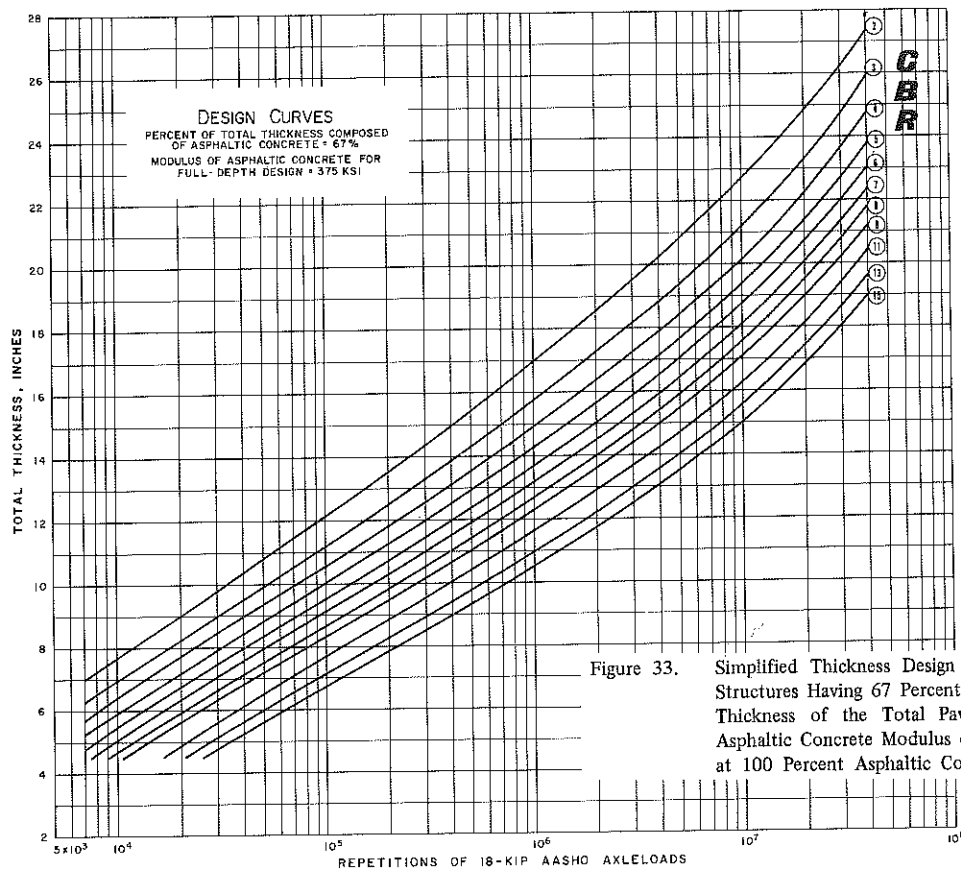


Figure 33. Simplified Thickness Design Curves for Pavement Structures Having 67 Percent of Asphaltic Concrete Thickness of the Total Pavement Thickness for Asphaltic Concrete Modulus of 375 ksi (2.59 GPa) at 100 Percent Asphaltic Concrete in Figure 28.

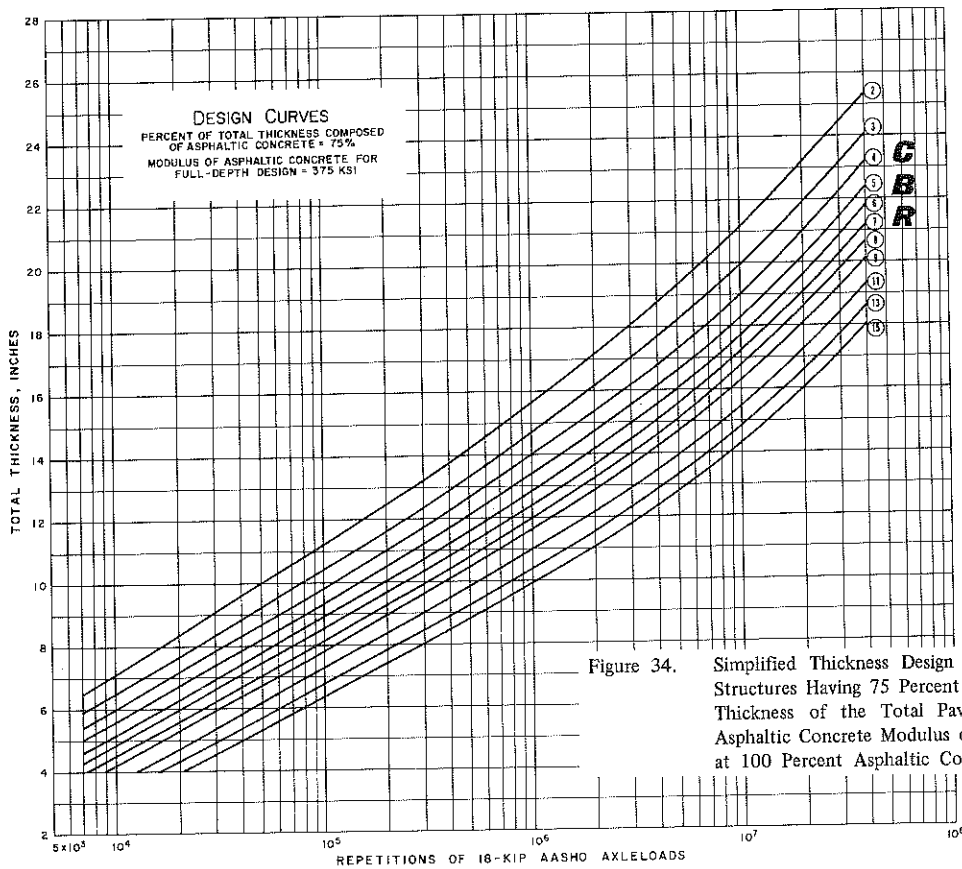


Figure 34. Simplified Thickness Design Curves for Pavement Structures Having 75 Percent of Asphaltic Concrete Thickness of the Total Pavement Thickness for Asphaltic Concrete Modulus of 375 ksi (2.59 GPa) at 100 Percent Asphaltic Concrete in Figure 28.

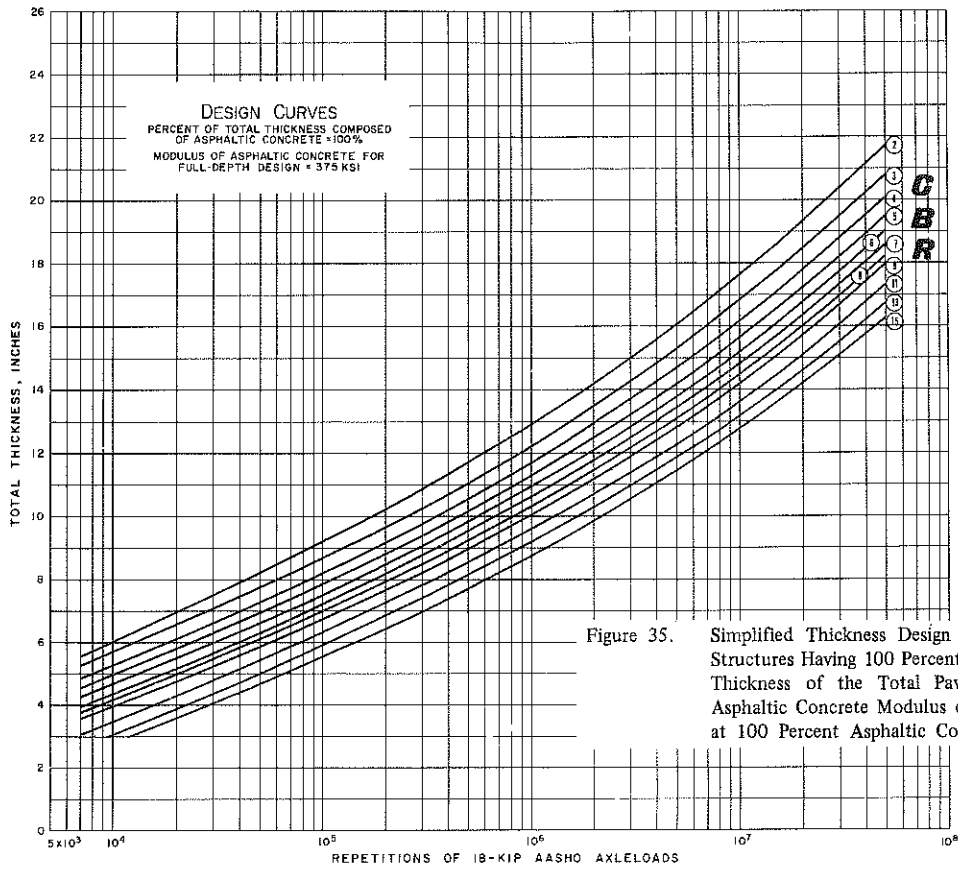
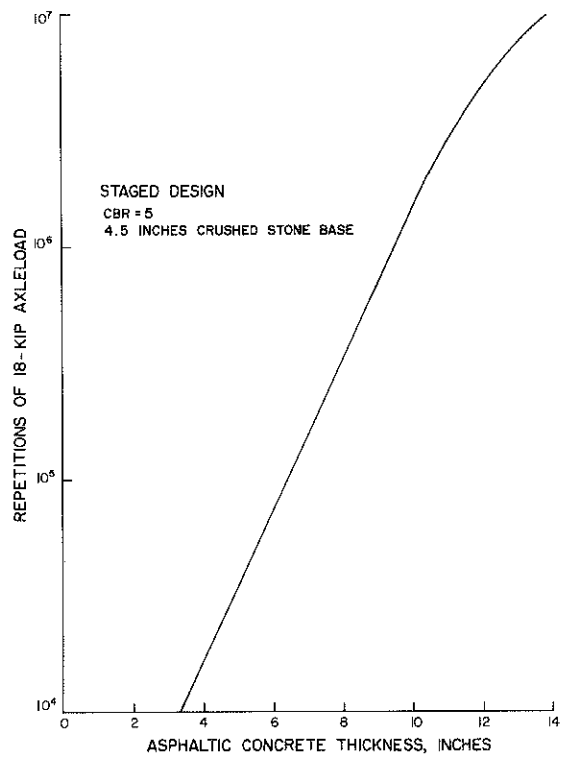


Figure 35. Simplified Thickness Design Curves for Pavement Structures Having 100 Percent of Asphaltic Concrete Thickness of the Total Pavement Thickness for Asphaltic Concrete Modulus of 375 ksi (2.59 GPa) at 100 Percent Asphaltic Concrete in Figure 28.

Figure 36. Repetitions of 18-kip (80-kN) EAL.s as a Function of Asphaltic Concrete Thickness for Fixed Conditions.



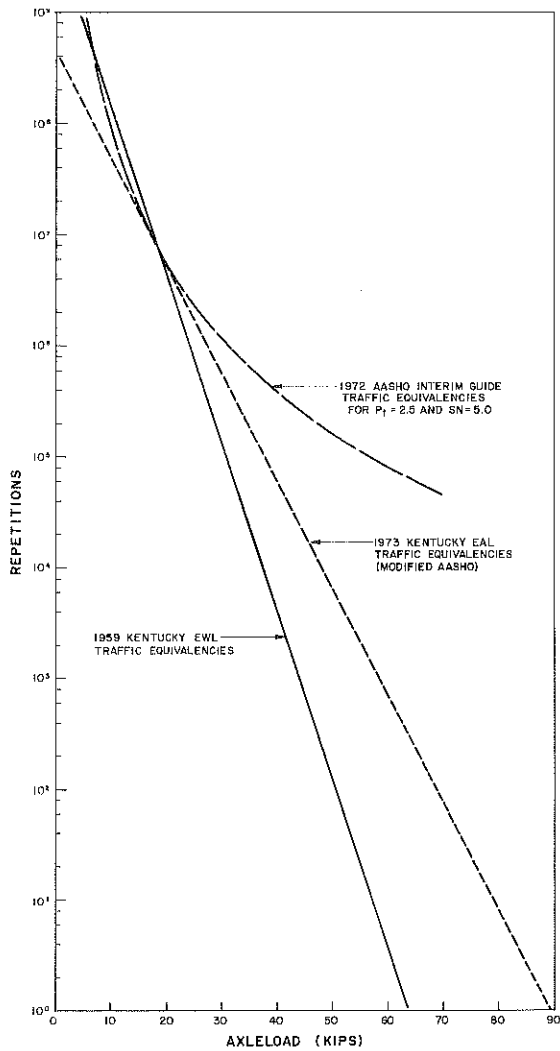


Figure 37. Repetitions versus Single Axleload Equivalencies.

of pavement thicknesses (SN values) as well as being subjected to repetitions of limited axleloads. Thus, a given SN could be evaluated only over a comparatively short range of axleloads. But for the range of axleloads, the log-log relationship was a reasonable approximation and is not much different from a semi-log relationship as shown by the 1973 Kentucky traffic equivalencies (Figure 37).

Kentucky experience indicated that the semi-log relationship was a better approximation. The 1959 and 1973 Kentucky EAL traffic equivalency relationships are shown in Figure 37. Analyses have shown the 1959 progression to provide damage factors that were too severe for heavy axleloads and too small for lighter axleloads. Thus, the "1973 Kentucky Traffic Equivalencies" were changed to have a base value of 18 kips (80 kN) and were made tangent to the AASHO log-log relationship at 8×10^6 repetitions for a SN of 5.36 -- the solution of Equation 5 for $P_t = 2.5$. This change provided the linkage of Kentucky and AASHO Test Road experience.

The Kentucky axleload - log of repetitions curve does permit one repetition of a very large axleload to cause the same damage as 8×10^6 repetitions of an 18-kip (80-kN) axleload. However, proceeding in the other direction, the mathematical impossibility of billions of repetitions of a zero axleload exists. The authors recognize this discrepancy. Experience has shown the normal range of axleloads on a highway to have the semi-log relationship, but extremely light loads would depart from the semi-log line and become asymptotic as the axleload became infinitesimally small.

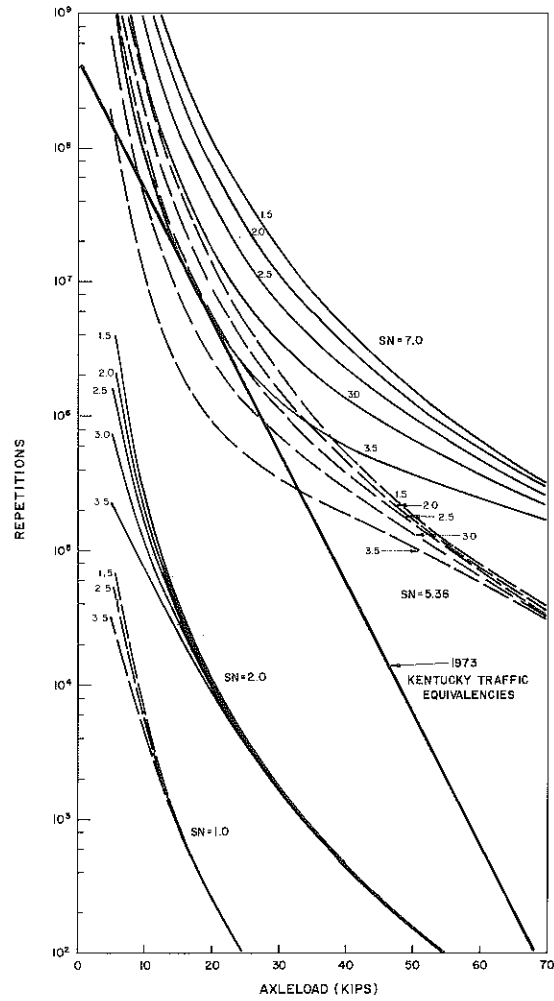


Figure 38. Relationship between Repetitions and Single Axleload for AASHO SN and P_t Variations.

Figure 39 illustrates the axleload-traffic equivalency factors for single axleloads.

Kentucky analyses (34, 35) of AASHO Test Road data (36) indicated 10^5 repetitions of a 34-kip (151-kN) tandem axleload were equivalent to 10^5 repetitions of an 18-kip (80-kN) axleload for a structural number of 3.07 at a serviceability level of 2.5 (Figure 40). Thus, 8×10^6 repetitions of both the 18-kip (80-kN) single axleload and 34-kip (151-kN) tandem axleload are equivalent as shown in Figure 40. Figure 41 shows the relationship between tandem axleloads and damage factors.

Rutting -- A rutting criterion (Figures 21 and 22) was used to adjust design thicknesses for low volume highways. The AASHO concept of terminal serviceability (P_t) is also a way of adjusting for traffic volume and/or permissible damage. Figure 42 gives the equivalent Kentucky relationship between level of serviceability and axleload for both single and tandem axle groups. Using superposition principles and the combination of Figures 39, 41, and 42, Figures 43 and 44 show that the relationship of increasing axleload with increasing P_t coincides with the concept of "rutting criteria" recognized in the 1973 criteria. The 1973 thickness curves provide for variable terminal serviceabilities which are dependent upon expected repetitions of equivalent axleloads. Higher values of terminal serviceability are used for heavily traveled routes (because of the need for a minimum of rutting, maintenance, and traffic restrictions). The terminal serviceability is allowed to decrease progressively to the minimum level for lightly traveled routes where geometrics restrict speeds and rutting is not such a prominent and

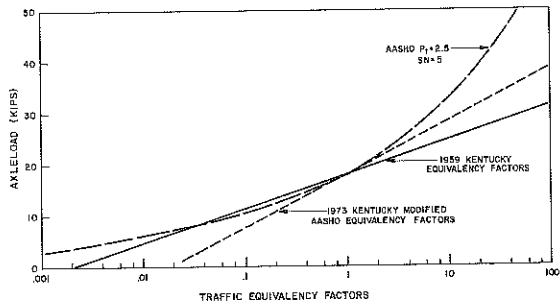


Figure 40. Relationship between AASHO SN for 10^5 Repetitions at $P_t = 2.5$ and Single and Tandem Axleloads.

Figure 39. Traffic Equivalencies Related to Single Axleloads.

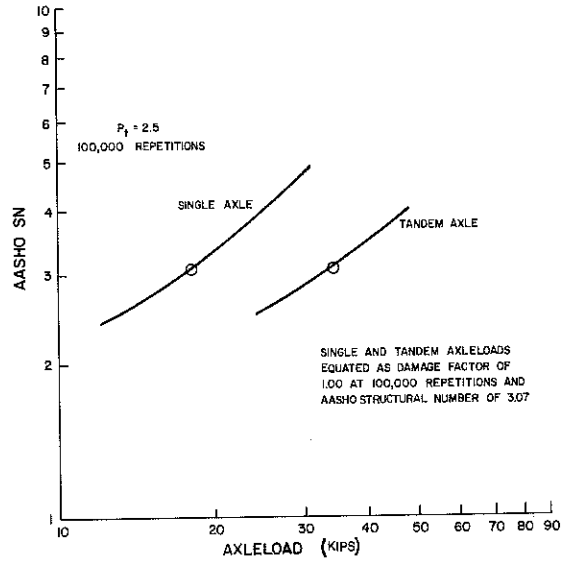


Figure 41. Traffic Equivalencies Related to Tandem Axleloads.

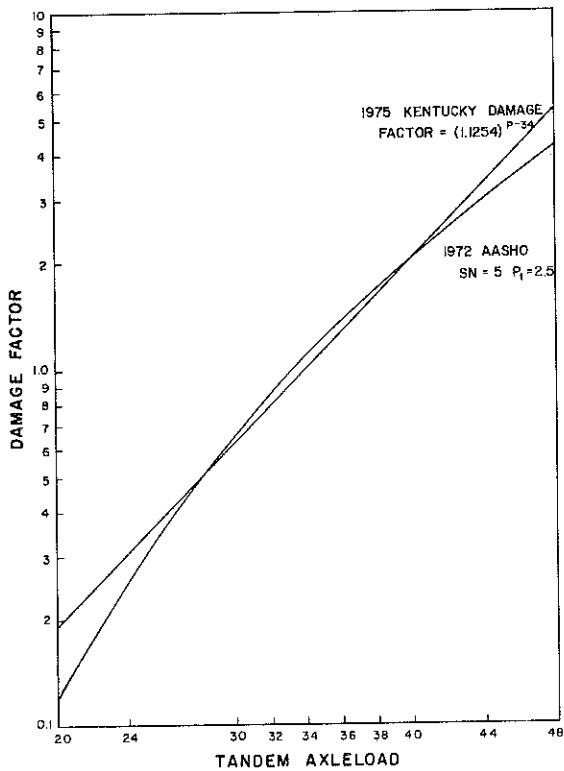
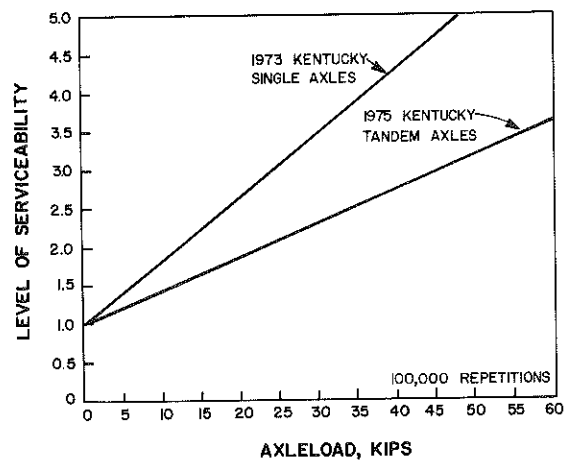


Figure 42. Level of Serviceability Related to Single and Tandem Axleloads



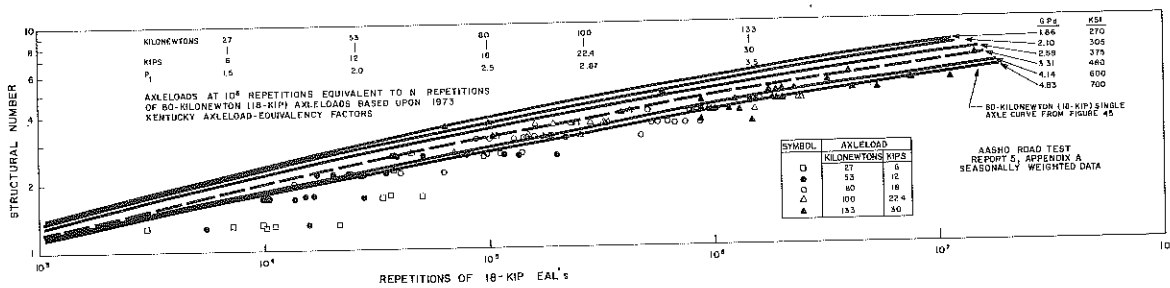
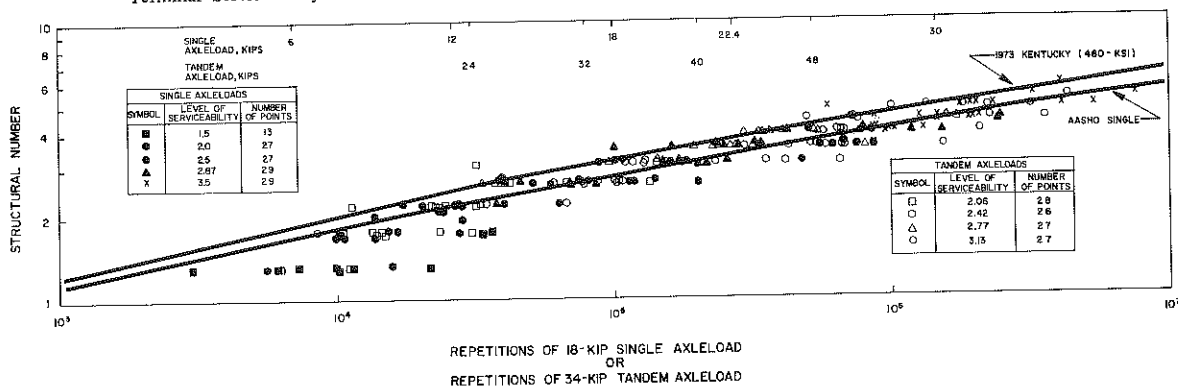


Figure 43. Relationship between SN, Repetitions of Single Axleloads at Various Terminal Serviceabilities, and 18-kip (80-kN) Single Axleloads at a Terminal Serviceability of 2.5.

Figure 44. Relationship between SN, Repetitions of Single and Tandem Axleloads for Respective Terminal Serviceabilities from Figure 42, and 18-kip (80-kN) and 34-kip (151-kN) Tandem Axleloads at a Terminal Serviceability of 2.5.



dangerous factor. The following observations of Figure 43 are made:

1. the 480-ksi (3.31-GPa) asphaltic concrete modulus Kentucky curve encompasses approximately 93 percent of all AASHO data points (Kentucky moduli lines for 270, 305, 375, and 600 ksi (1.86, 2.10, 2.59, and 4.14 GPa) have also been superimposed);
2. the 18-kip (80-kN) single-axle curve for $P_t = 2.5$ from Figure 45 (36) was superimposed onto Figure 43, showing that the curve encompasses 59 percent of the data points except for the 6-kip (27-kN) set, and is almost a direct fit of the family of asphaltic concrete elastic moduli lines;
3. the combination of 1 and 2 above confirms that the 1973 Kentucky design guide (37) does provide for variable terminal serviceability levels, according to the EAL level, with a high probability of success in the design life;
4. elastic theory provides a sound basis for accurately describing the observed performance data of both Kentucky highways and the AASHO Test Road;
5. one level of terminal serviceability should not be used for the wide range of EAL's, as suggested by AASHO thickness design nomographs, and
6. AASHO nomographs provide for only approximately a 60-percent probability of successful pavement performance throughout the design life.

Figure 44 is Figure 43 with the addition of the companion tandem axleload data of the AASHO Test Road (34). There is virtually no increase in the spread of data in Figure 44 compared to Figure 43. Thus, the conclusions (34) stated above are reinforced by the addition of the tandem axleload data.

Kentucky Highway Pavement Performance - Recent investigations (13) have shown that past experiences in pavement designs correlate reasonably well with multilayered, elastic theory

computations. In the past, design procedures (1, 2) have been based largely upon empirical correlations between observed pavement performance and various traffic and pavement parameters thought to be significant with respect performance. Recently, however, emphasis has shifted to analyses based on theoretical computations of imposed stresses, strains, and deflections. This more fundamental approach is not unlike that commonly employed in more conventional structural analyses.

In an effort to evaluate and further define the structural properties of asphaltic concrete bases, the Kentucky Department of Highways designed and constructed an experimental pavement on US 60 in Boyd County in 1971. This project consisted of nine sections of full-depth asphaltic concrete consisting of thicknesses

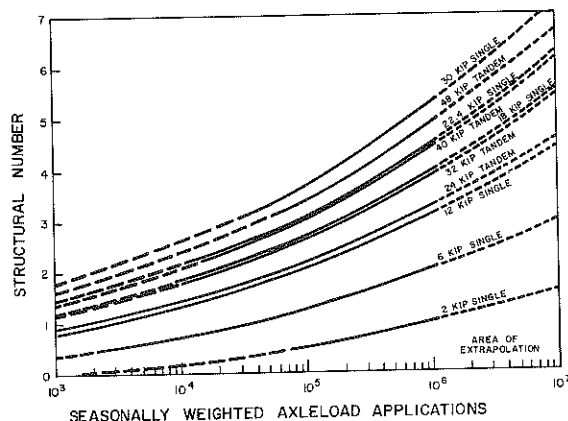


Figure 45. Main Factorial Experiment, Relationship between Design and Axle Applications (Reference 36, Figure 22, page 27).

varying from 10 to 18 inches (254 to 457 mm), and a conventional combination dense-graded aggregate and asphaltic concrete pavement. Other full-depth asphaltic concrete pavements in Kentucky have been in use since at least 1941.

The objective of the experimental study was to extend the state of knowledge and experience in flexible pavement design by documenting the construction of a series of full-depth asphaltic concrete pavements with known subgrade and pavement component properties and by determining the mechanical response of the entire structure to static and dynamic traffic loads. To attain these objectives, the following goals were established:

1. determine the properties of the compacted subgrade and surfacing during construction using laboratory tests and field inspection efforts greater than used by construction inspectors;
2. evaluate the traffic stream by weight, vehicle classification, and lane distribution and use these data to develop pavement fatigue historical data; and
3. evaluate pavement response to static and creep loads by Benkelman beam tests and dynamic deflection tests.

Deflection responses were obtained layer by layer during construction, upon completion of construction, and subsequent to construction (29). Deflections were obtained by use of Benkelman beams, the Road Rater, and the Dynaflect. Test results from one location within each test section were analyzed to determine which relationships were, or were not, meaningful. This was done as a pilot study and as a preliminary step toward final analysis.

Superimposed on Figure 46 are (1) curves for the AASHTO Test Road pavements (36) having 3-inch (76-mm) and 6-inch (152-mm) crushed stone bases, (2) curves for the three-layered control sections with 12-inch (305-mm) and 19-inch (483-mm) crushed stone bases on the US 60 road test, and (3) Kingham's (38, 39) "A" and "B" curves. Kingham's "A" curve was based upon Benkelman beam test experience with Canadian pavements consisting of approximately 3 inches (76 mm) of asphaltic concrete on 24 inches (610 mm) of crushed stone. Kingham's curve "B" was based upon tests on full-depth asphaltic concrete pavements in Colorado placed directly on a weak subgrade.

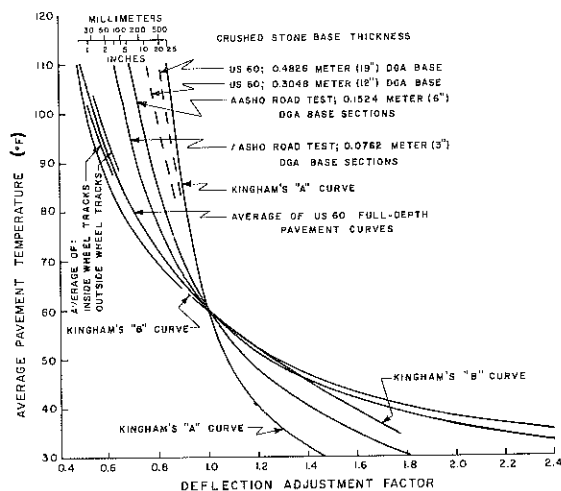


Figure 46. Average Pavement Temperature versus Benkelman Beam Deflection Adjustment Factors for Full-Depth and Three-Layered Asphaltic Concrete Pavements.

The deflection-temperature adjustment factor curves have several positions which appeared to be a function of subgrade support. While the average in-place CBR value for US 60 was 18, Kingham's curve "B" has the position of a weak subgrade. This conclusion is supported by the AASHTO Test Road pavement. The curve for the AASHTO sections having 3 inches (76 mm) of crushed stone base lies closest to the full-depth curves while the 6-inch (152-mm) crushed stone base sections were on a subgrade having

a 12-inch (305-mm) improved layer and is closer to Kingham's "A" curve. Thus, the adjustment factor curves for the three-layered pavements on US 60 fall between Kingham's "A" and "B" curves and are in the proper relative positions. The positions of the temperature adjustment factor curves may, therefore, be a function of equivalent substructure support. Closer inspection of Figure 46 suggested that the relative positions of the curves can be expressed on a logarithmic scale of crushed stone base thickness. The scale increases from Kingham's "B" curve for full-depth pavements to his "A" curve for thick crushed stone bases.

The Chevron N-layered computer program (3) was used to develop design curves (13) (Figures 8, 14, 15, and 31-35). The same program was used to simulate the dynamic loading applied by the Road Rater. Field test data (29) very closely matched the theoretical results from the N-layered program when the modulus of the asphaltic concrete was adjusted for the measured pavement temperatures using Figure 27 and the subgrade conditions were determined by in-place CBR tests. Road Rater tests on other Kentucky pavements have been duplicated by the same simulation methodology coupled with the N-layered program.

Airport Design -- Airport pavement thickness design (40) can be closely duplicated by converting aircraft wheel loads to equivalent 18-kip (80-kN) axleloads using Figure 12. Witzak (41) analyzed taxiways at the Baltimore-Washington International Airport and used the DC8-63F aircraft as the prototype for his analysis. The 10-inch (254-mm) asphaltic concrete pavement of modulus 600 ksi (4.14 GPa) subjected to a 43-kip (191-kN) wheel load of the reference aircraft and to a 9-kip (40-kN) Kentucky base wheel load yields a vertical compressive subgrade strain ratio of 4.14. A 43-kip (191-kN) wheel load has a Kentucky damage factor of approximately 2,000 compared to a 9-kip wheel load. The 13,239 repetitions of the DC8-63F reference aircraft is equal to 26,478,000 repetitions of a 9-kip (40-kN) Kentucky wheel load and corresponds to a tensile strain of 1.08×10^{-4} for the Kentucky modulus of 600 ksi (4.14 GPa). Using a Kentucky wheel load damage factor of 4.14 would yield a predicted strain value of $(4.14) \times (1.08 \times 10^{-4}) = 4.47 \times 10^{-4}$; the corresponding predicted number of repetitions of the DC8-63F would be 8,200 for 600 ksi (4.14 GPa). However, the taxiway is subjected to a loading cycle of 2 Hz, but the Kentucky system is based upon Benkelman beam test values corresponding to approximately 0.5 Hz. Figure 47 shows that 600 ksi (4.14 GPa) at 70 F (21 C) and 2 Hz corresponds to 375 ksi (2.59 GPa) at 70 F (21 C) and 0.5 Hz. According to Figure 11, a tensile strain of 4.47×10^{-4} has an approximate corresponding value of 14,000 repetitions. Therefore, an airport pavement can be designed using the Kentucky system provided the moduli are adjusted for frequency of load applications and for wheel loads.

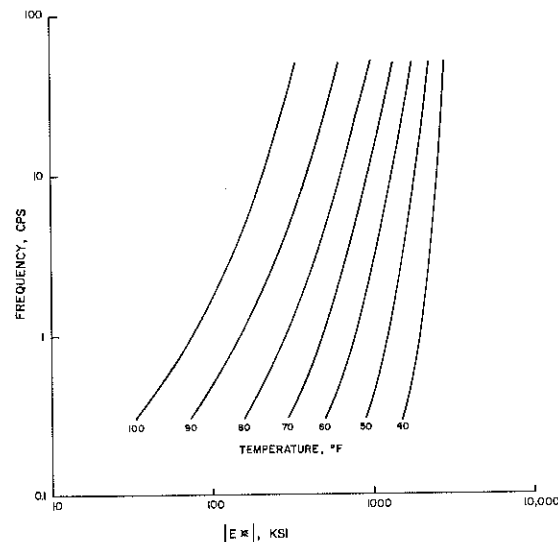


Figure 47. Asphaltic Concrete Modulus Variations as a Function of Mean Pavement Temperature and Frequency of Loading.

Witczak's airport pavement example above (41) was based upon Kingham's fatigue criteria (42) which used the concept of parallel lines on a log strain - log repetitions plot. Kingham used the equation $N = K(1/\epsilon_a)^c$ where c had a value of 4.215. The Kentucky strain-fatigue relationship (Figure 11) can be expressed by the same equation; however, K and c vary according to the average pavement temperature as shown in Figure 48.

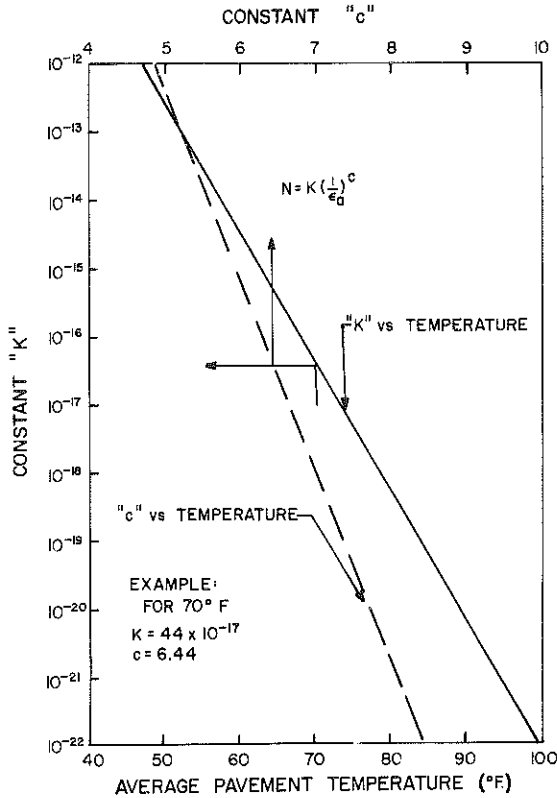


Figure 48. Effects of Average Pavement Temperature upon Values of "K" and "c" in the Equation $N = K(1/\epsilon_a)^c$.

PAVEMENT MANAGEMENT

Pavement management is only one portion of a highway or transportation management system which would include other major categories as traffic and operations, maintenance, and finance. Subsidiary functions would include overhead categories such as planning, design, materials, construction, research, etc. Others (43-48) have included under pavement management such items as user costs and socio-economic and environmental impact. Within the concept of this paper, the above factors definitely have an influence upon transportation management and do influence roadway design. Thus, these factors have already had their effect prior to the design and construction of a new pavement or an overlay to an existing pavement. Pavement management is defined herein to be limited to the factors affecting the pavement structure, not the roadway width, etc. Similar flow charts, such as Figure 49a and b, can be made for these other major functions.

The authors recommend that all data be entered into data files but do not recommend automatic incorporation for determination of revision to existing standards. Indeed, such action may cause totally unwarranted, biased, and incorrect changes. The data should be screened to determine those that properly should become a part of the data bank to evaluate the standard criteria.

In truth, pavement management has no beginning or end. Transportation departments historically inherited Indian and animal trails which have been expanded through the years to evolve into

highways. As needs have arisen, new facilities were constructed which then became a part of existing highway facilities. As society increased its mobility, additional or improved highway facilities have been required and the cycle was completed once more. Thus, one entry point must be an inventory of existing facilities.

Existing Facilities -- Figure 49a shows a basic management flow diagram for existing pavement facilities. An inventory of existing pavements should contain information about a predefined length of roadway, using milepoints or reference points of a particular highway. The basic groups of data should include an inventory of physical features of the pavement, structure, surface characteristics, behavioral indicators, and traffic-related items. While Figure 49a shows many items of information and(or) test results, the data file should contain some behavioral test data which could be any one or combination of these items. Certainly, not all of them are required.

These data files should be queried at least on an annual basis, but a query could be initiated by management at any level, by inspection reports, or by a complaint from the general public, etc. These reviews can be accomplished manually but more efficiently through computer programs which analyze the computerized data files. Such a review of one data file may indicate a pavement to possibly need some remedial action. Other data files would then be reviewed for this particular pavement length to indicate sources of data which may lead to a technically proper solution. If the same problem appears in a significant number of pavements, then there is a distinct possibility that standards may need revision. If such a revision is to be enacted, data files may need to be reanalyzed to determine the impact of such changes in criteria upon the existing pavement facilities and the resulting total needs.

Analysis of the **Maintenance Expenditures File** can have very profound effects upon pavement management and the assigning of priorities. If expenditures appear to have become excessive, two immediate possibilities exist. Either the pavement structure is in jeopardy or the existing facility is so overburdened that extensive repair is not the answer. For the latter possibility, additional lanes or a new parallel facility may be needed. Thus, the problem should be referred to planning functions for analysis. If the pavement structure is in jeopardy, then a routine overlay is not the appropriate solution either. Figure 49a suggests an appropriate solution.

In Kentucky, research performs an important function and supplies vital input to management. Many pavement problems are routine and no longer require research. However, some repetitive problems do require research, and solutions are reviewed by management. Some solutions involve answers to specific case histories. In many instances, other solutions involve proposed changes to specifications, criteria, and(or) standards. In Kentucky, management and research work together and in many instances research efforts and proposals have saved unnecessary expenditures by management.

Assuming that standards do not need to be revised, inspection of the accumulated EAL to date, design EAL level, and the current year may suggest when the design EAL could be expected to be reached. Thus, a predetermined percentage of the design EAL could be established as the indicator at which a particular pavement would be added to the Needs File along with pertinent technical, economic, and calendar data, etc. for scheduling an appropriate overlay.

A pavement can become slippery for many reasons, such as abrasion under traffic, excessive loads, high asphalt contents and(or) low voids content, polishing aggregates, etc. Analyses of the **Accident File** indicates high accident-rate areas, which, when coupled with traffic volumes, may be abnormally high and may suggest the need for testing by the skid-test crew. Whatever action is recommended for an existing facility is then subjected to an economic analysis which is submitted with the technical data to intermediate or top management for consideration. This action will be discussed later.

New Facilities -- Figure 49b shows a basic management flow diagram for new facilities, which are the result of two actions. The first results from an overcrowded facility which cannot possibly perform its intended function. The second action comes from the initiation of a totally new system, such as the creation of parkways or the passage of the statute which authorized the interstate system. Other more subtle actions can be the result of changes in design

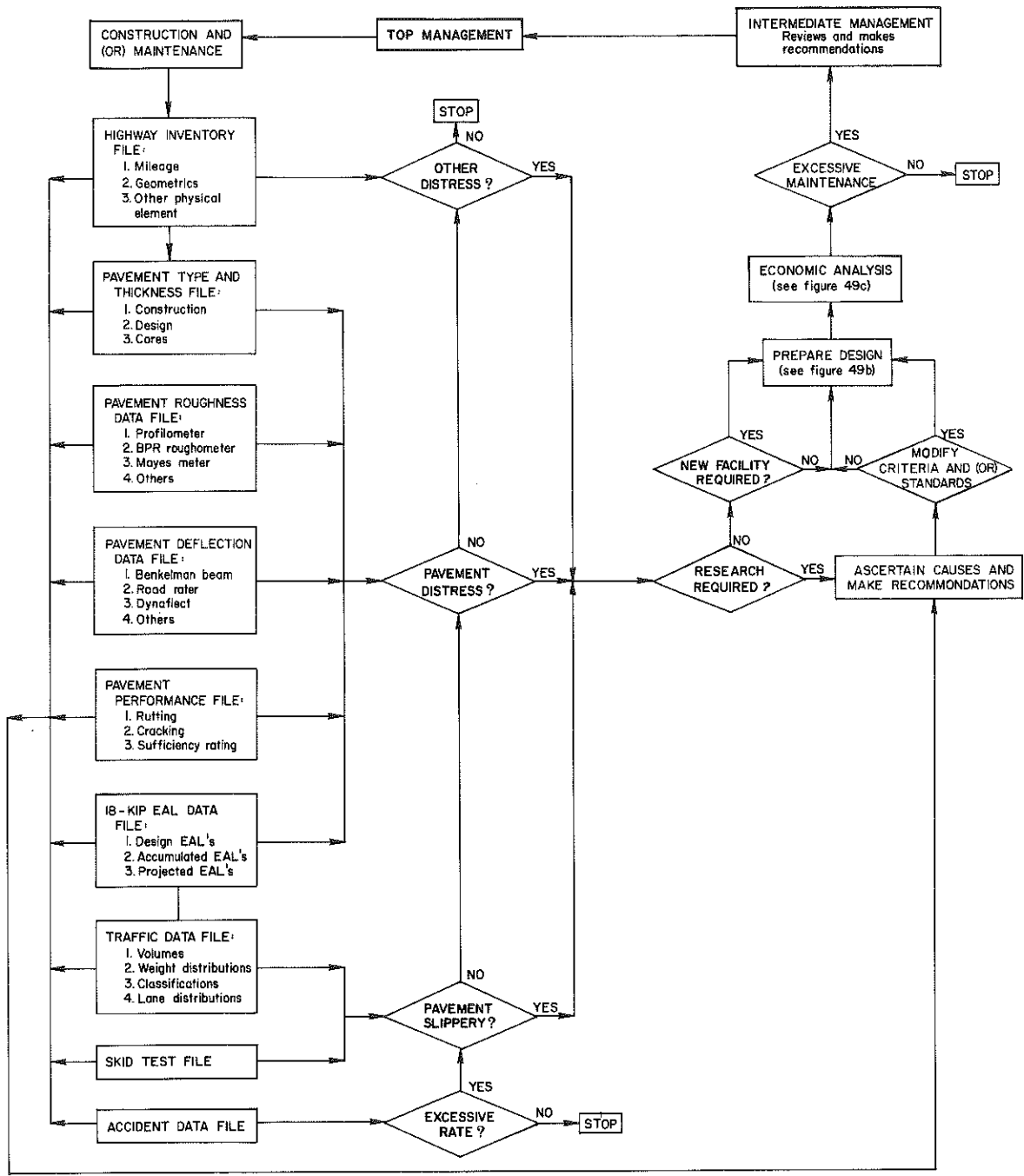


Figure 49a. Flow Diagram for Analysis of Data Files for Pavement Problems of Existing Facilities.

criteria, standards, and traffic parameters such as changes in style of cargo hauler and changes in vehicle classifications of the traffic stream.

Naturally, a proper pavement design must be predicated on soil conditions over the project length and the expected traffic. Most often, traffic is the least predictable and causes the most variation between design and actual life in terms of years. The simplest process is to estimate the current first-year AADT and to expand this value through the term of years by the compound interest equation. The compounding rate, usually between five and six percent, may be updated periodically and by class of highway. Very little information is available about the accuracy of these or other forecasts. A cursory analysis of approximately 40 projects in 1959

(2) indicated that 15.9 percent of the projects did or would reach the estimated, 10-year, equivalent loadings in 6.8 years. The standard deviation was 68 percent. This example does not necessarily impute the method of making forecasts but implies that if an average, compounding rate is used, the forecast will also represent an average situation. However, three standard deviations indicate a high improbability that the actual outcome of a particular project would exceed twice or be less than half the estimate.

Economic Analysis - Figure 49c illustrates the major factors that influence an economic analysis of recommended technical action issuing from Figures 49a and b.

Management - Figure 49d illustrates the management process which usually will include both intermediate and top levels.

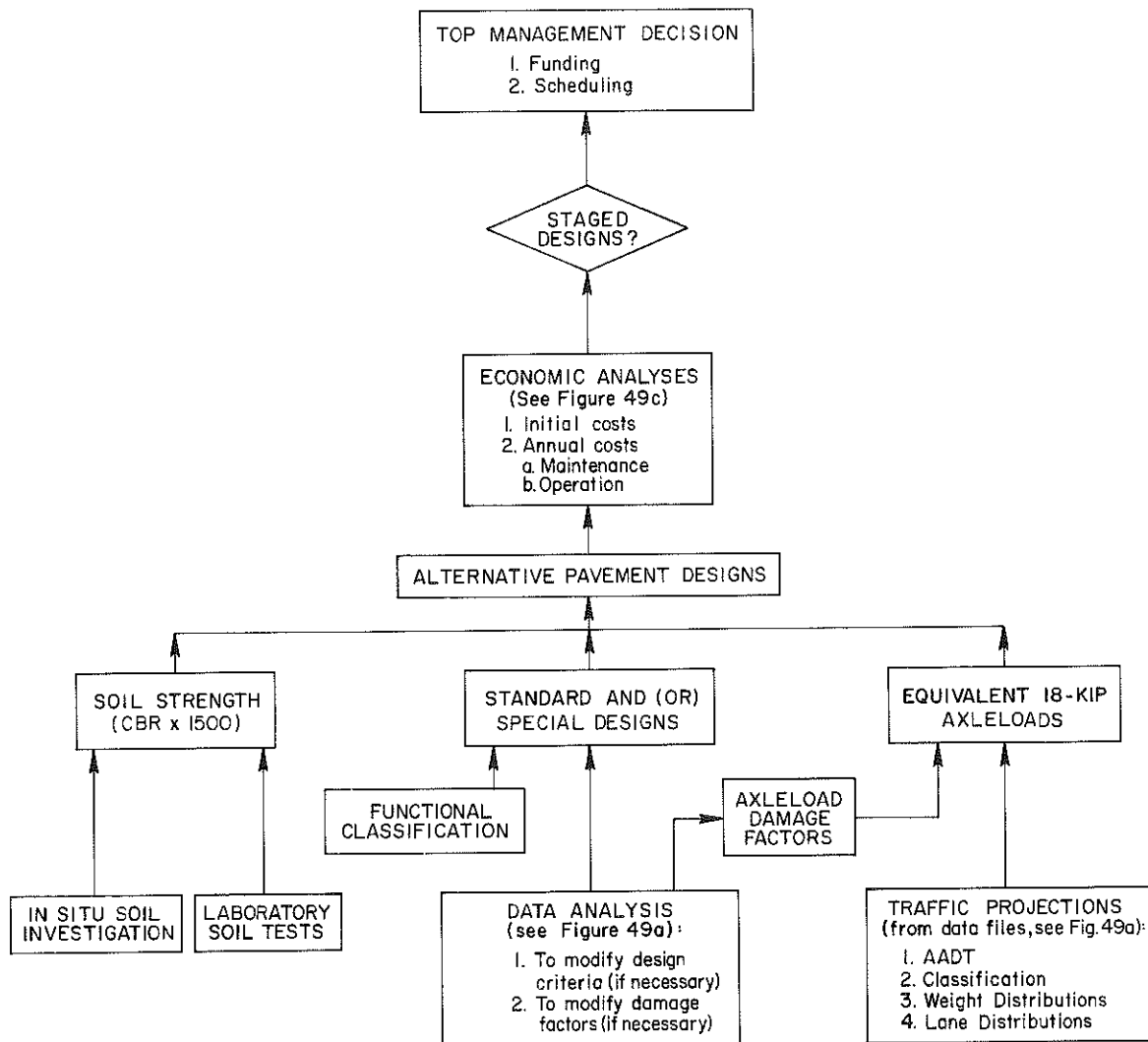


Figure 49b. Flow Diagram for Pavement Design for New Facilities.

Intermediate management will decide which recommended actions should be considered for final action and those which should be placed in a *Schedule for Future Needs File*. The latter group would include those pavements which have reached a critical level of the designed fatigue life and should be scheduled for action during some recommended year.

Factors which bear upon decisions of top management include the submitted list of recommended actions by intermediate management, designs for new facilities, functional classification of each recommended action, available funds, and political considerations of all types. This list is then subjected to a dynamic programming analysis (49) which indicates those projects that will produce the highest benefit-cost ratio coupled with needs. Top management then assigns priorities, matches the funds, and the construction process begins. Those projects not funded should be immediately analysed to determine if some possible future complication compels a rejected project to be reconsidered. Upon reconsideration, priorities are reassigned, or the project is placed in the *Needs File* for future action. Upon construction, pertinent data is added to the appropriate data files of existing facilities, thus completing the cycle.

The *Needs File* should be one of the most important files and sources of information for all levels of management. All pavements requiring overlays for any reason, the rebuilding or expansion of any existing facility, and scheduling of new construction should be

contained in this file. Since Kentucky has more than five functional classifications of highways, proposed actions should be sorted into their respective classifications. Future inquiries to this file could be made for many reasons, some of which are

- (1) backlog of individual needs and each respective cost estimate,
- (2) backlog of cost and effort within each functional classification,
- (3) information for normal budgetary efforts, and
- (4) background for financial requests submitted to the legislature for possible changes in tax laws and funding programs.

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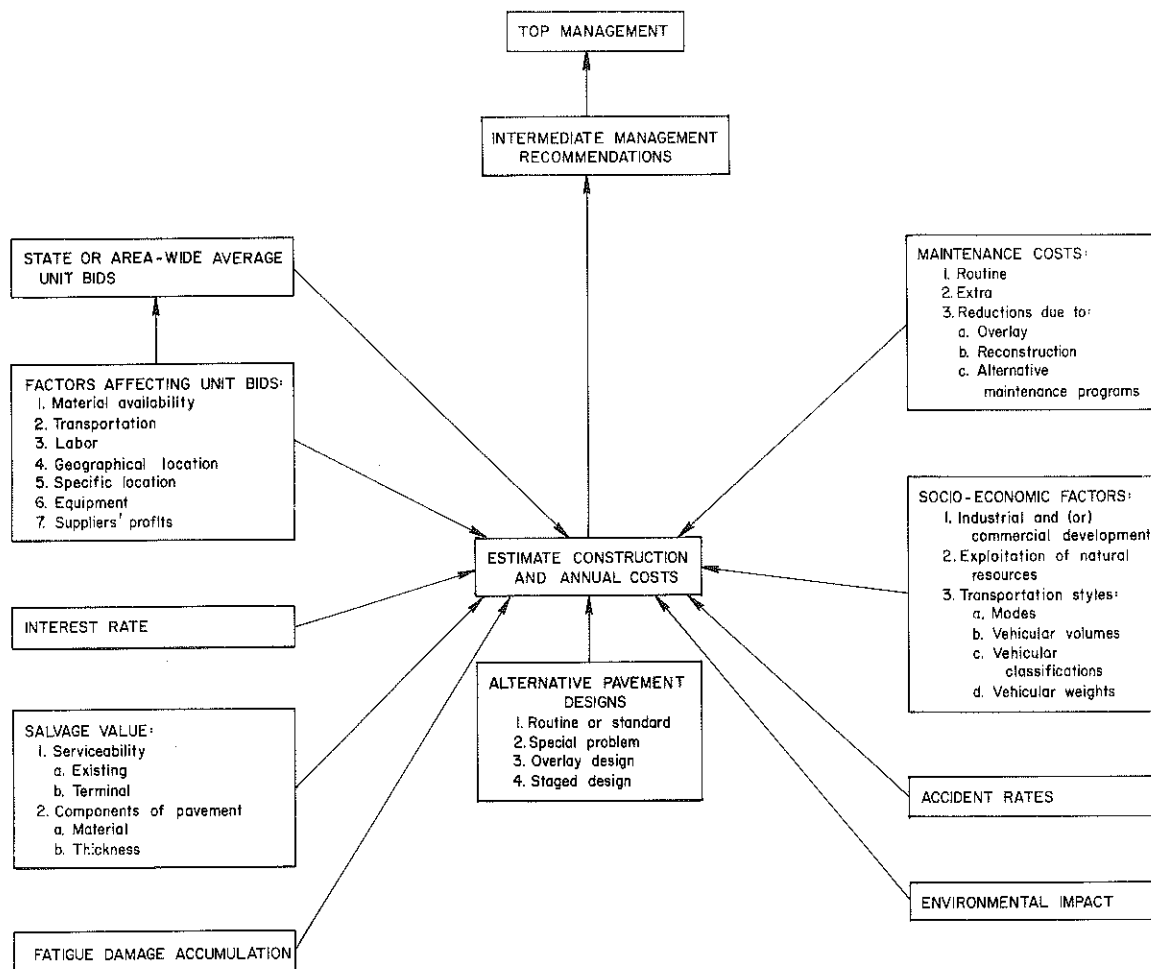


Figure 49c. Flow Diagram for Economic Analysis of Recommended Action for Existing and New Facilities.

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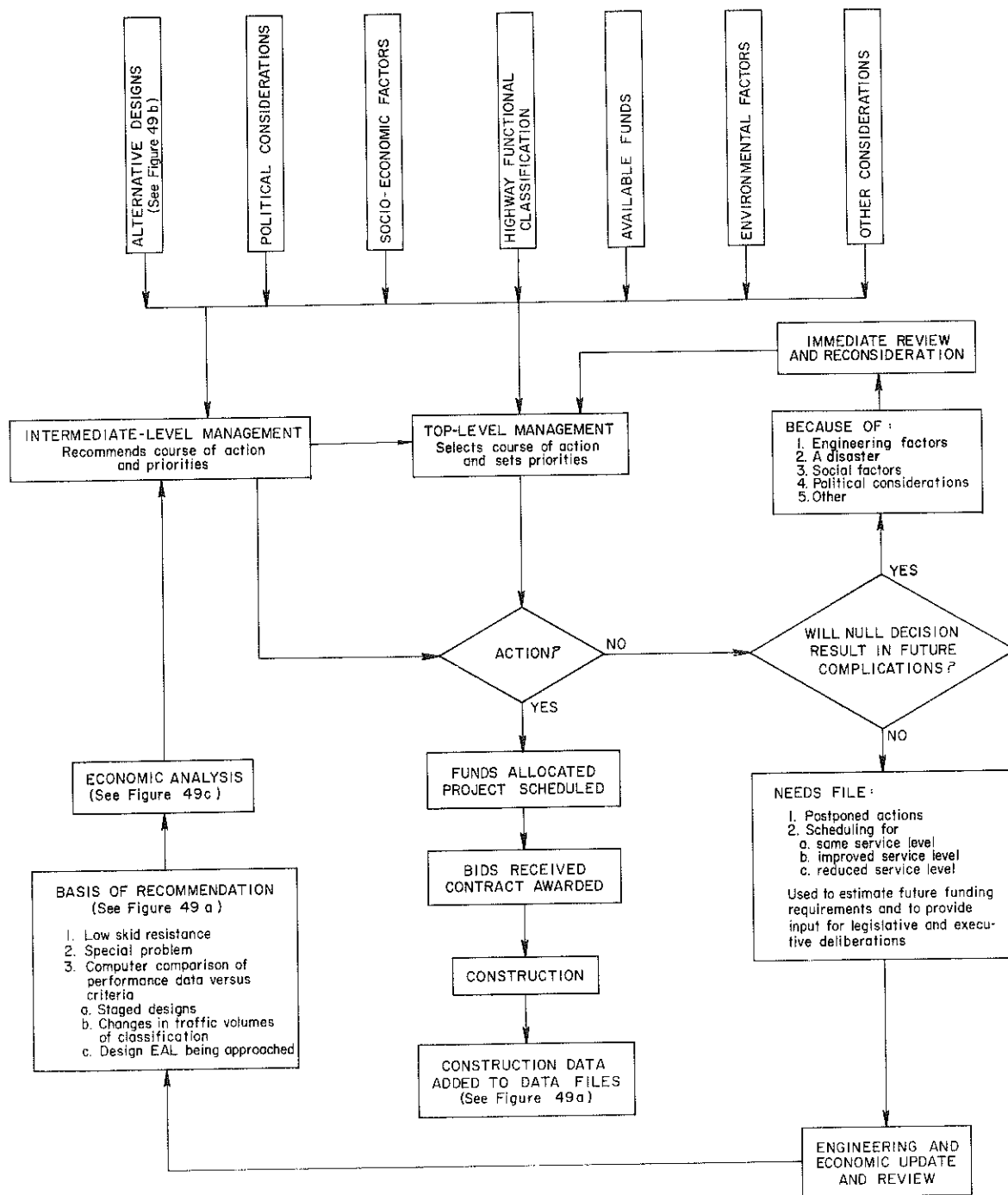


Figure 49d. Flow Diagram for Intermediate and Top Level Management Decisions Related to Pavement Management Problems.

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