Elastic theory and 40 years of empirical flexible pavement design in Kentucky have been joined into the design system presented in this paper. A brief discussion is presented of the coupling mechanisms relating experience to theoretical analyses. An annotated design procedure is presented as a guide for pavement designers. Design nomographs account for a wide range of input parameters and permit the designer a wide choice of alternative thickness designs.

Pavement Design Schema

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The approach to a structural engineering problem is to resolve an equation of equilibrium and an equation of failure. The simplest equilibrium equations are found in elastic theory. The simplest failure equations are statements of phenomenological strengths. A rational design criterion for pavements must be compatible with all past experience and performance histories. In fact, collectively, those experiences are the best available equations of failure. Empirical design systems qualify abundantly in this way.

Many logic statements may be needed to transform empirical parameters into classical units and to bring experiences into conformity with strict mechanistic disciplines. When they are so transformed and anomalies are resolved, the predictive capabilities of the mechanistic theory stand confirmed; and the schema is claimed to be rational. Indeed, an enabling element in this venture was the Chevron computer program (1) to solve N-layered, elastic theory problems. The empirical resources were contained in a well-developed, experience-tested, equivalent wheel load (EWL)-California bearing ratio (CBR)-thickness design criterion or system (2, 3, 4).

From the mechanistic point of view, load-deflection relations outwardly portray the composite stiffness or rigidity of pavement systems. Contrary to general impressions, surface deflection is not a discrete, limiting parameter. Stresses and strains in the subgrade soil and in the extreme fibers of bituminous concrete layers constitute overriding, fundamental limits. Therefore, thickness design criteria cannot be based directly on deflection spectra.

It is historically evident that many pavements fail through fatigue and creep. In the fatigue domain, the state of strain and stress or both is computable from elastic theory. Obviously, it is necessary to resolve a suitable fatigue diagram. Customarily, fatigue diagrams are in terms of either controlled strains or controlled stresses. Creep alludes to the mechanism of rutting and is most easily handled in a separate analysis. In that instance, creep, or rutting, is handled empirically.

DEVELOPMENT OF DESIGN PROCEDURE

The controlling, empirical model in this instance was the 1958 Kentucky design curves shown in Figure 1 (4). It involved 3 parameters and 3 layers. By convention, the total thickness has been proportioned to be approximately ⅓ asphaltic concrete and ⅔ crushed rock base. Control points were selected for matching and balancing the elastic theory and fatigue analyses. Analysis of computer (Chevron program) results prevailed in the rightward portion of Figure 1, that is, to correct earlier errors in judgment in placing the design curves. Of course, the objective was to reconstitute those curves through theory (5, 6). Layer moduli and thicknesses were arrayed, and many solutions were obtained; numerous influence graphs were plotted. The necessary input assumptions are given below.

1. E_1 (modulus of elasticity of layer 1) ranged from 150,000 to 1,800,000 psi. The effective moduli of asphalt-bound layers depend on 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.
2. Poisson's ratio of layer 1 = 0.40. Dormon and Edwards (7) have reported that Poisson's ratio of such materials varies from 0.35 to 0.45.
3. E_2 (modulus of elasticity of layer 2) = F × CBR × 1,500, where F is found from curves shown in Figure 2 (5, 6, 8); note that F = 1 when $E_1 = E_2 = E_3$. Heukelom and Klomp (9) have shown that the effective elastic moduli E_2 of granular base courses tend to be related to the modulus of the underlying subgrade soil. 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 that ratio is generally between 1.5 and 4.0. A value of 2.8 was selected in this study as being typical at a CBR of 7 (Fig. 2). Comparison of the 1958 Kentucky design curves and field data (4) indicated that this assumption was reasonable. It was further assumed that the ratio of E_2 to E_3 would be equal to one when $E_1 = E_2 = E_3$. The curves shown in Figure 2 were then obtained by assuming a straight-line relation on a log-log plot. A review of the literature (10, 11) indicated that curves shown in Figure 2 give reasonable values for good quality granular bases within a range of practical design situations (CBR < 20); and, therefore, they were used throughout the analysis here to relate the modulus of the granular base to the subgrade support values. E_2 values are a function of E_1 and E_3 only.
4. Poisson's ratio of layer 2 = 0.40. Again, Dormon and Edwards (7) have reported Poisson's ratio of 0.35 to 0.45.
5. E_3 (modulus of elasticity of layer 3) = CBR × 1,500. Conversion from laboratory soil strength values to theoretical moduli of subgrades was aided by Heukelom and Foster (12), who developed a relation suggesting the subgrade modulus (in psi) is approximately equal to the product of the CBR and 1,500. Heukelom and Klomp (9) also indicated that this relationship is an acceptable approximation for evaluating subgrade moduli and provides a simple and practical approach to this estimation, at least for CBR's up to about 20.
6. Poisson's ratio of layer 3 = 0.45. Dormon and Edwards (7) indicated Poisson's ratio for subgrade materials on this order.
7. Tire pressure = 80 psi. Many firms in Kentucky indicated that they operated their trucks with a tire pressure of 80 psi.

A summary of the derivation of the fatigue criterion follows.

1. Kentucky EWL's (equivalent 5,000-lb wheel loads) were transformed into EAL's (equivalent 18-kip axle loads) (5, 6) by EAL's = 2-directional Kentucky EWL's/32.
2. The criterion concerning limiting strains in the asphaltic concrete was based on interpretative analyses of other work (13). Van der Poel (14, 15) indicated that a safe limit for asphalt was in the order of 1×10^{-3} at 30 F. Because asphaltic concrete consists of approximately 10 percent binder by volume, that fixes the safe strain level of asphaltic concrete at 30 F in the order of magnitude of 1×10^{-4}. Others (7, 13, 16, 17) have established (by interpretative analyses of pavements and fatigue test data) that the

magnitude of asphalt strain ϵ_A to ensuring 1×10^6 repetitions at 50 F was 1.45×10^{-4}. Limiting values of strain (all at 50 F) as a function of number of repetitions N of the base load (18-kip axle load in EAL computations) as given by Dormon and Metcalf (17) 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 3 and 4. The relations given by Kallas (18) between temperature and E_1 provided guidance at this stage.

Some investigators suggest a fatigue diagram of the load-log N type. Fatigue theorists (19, 20, 21) have suggested and shown in certain instances that a log load-log N plot is more realistic. Pell (20) suggested an equation of the form $N = K'(1/\epsilon_A)^n$, where n is the slope of the log ϵ_A-log N plot and K' is a constant. Pell (20), Deacon (19), and others have suggested that the value of n lies between 5.5 and 6.5 and is a function of the modulus of the asphaltic concrete. Pell's work further suggested that the family of curves relating log ϵ_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 relation was sought.

By plotting (to a log-log scale) the 18-kip tensile strain versus the tensile stress at the bottom of the asphaltic layer, we 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} (Fig. 3). By extrapolating Dormon and Metcalf's data (17), represented by the equation given above, to a value of N = 1, we found the asphaltic tensile strain 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 wheel load. By constructing lines tangent to the strain versus stress curves at a strain of 2.24×10^{-3}, we obtained modulus lines representing the limiting relations for asphaltic strain versus stress—independent of structural influences. The stress-strain ratios shown in Figure 3 are in terms of bulk moduli ($E_1 = 0.6K_1$, 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, 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 in. (control pavement) was 1.490×10^{-4}. The traffic associated with that control point was 8×10^6 EAL's. In Figure 3, a line drawn perpendicular to the line for an asphaltic concrete modulus of 480 ksi, 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 ϵ_A and log N, the curves shown in Figure 4 were obtained as representing the critical asphaltic concrete strains.

The limiting asphaltic stress-strain curves shown in Figure 3 are shown again in Figure 5. 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 axle load in kips (5, 6), is taken to be 2.24×10^{-3}. As shown in Figure 4, another known 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 2 points, the lines of equal numbers of repetitions shown in Figure 5 are obtained. The limiting asphaltic concrete tensile strain for any combination of number of repetitions and modulus of the asphaltic concrete can be read from curves shown in Figure 5 and are the same as those shown in Figure 4. The curves shown in Figure 5 converge to a common strain value at N = 1. That is a unique feature in the development of the schema. The convergence allows stress to proportionalize according to modulus when a limiting catastrophic strain is respected, regardless of modulus.

3. It was observed from computations and analysis (5) that the vertical strain at the top of the subgrade ϵ_S for the control pavement (CBR 7, 23-in. total pavement thickness, i.e., 7.7 in. of asphaltic concrete and 15.3 in. of crushed stone base) was 2.400×10^{-4}. A review of other work (10, 17) also indicated that an ϵ_S of 2.400×10^{-4} for 8×10^6 18-kip axles would provide a high degree of assurance against rutting; that value was thus assigned to ϵ_{S9} at 8×10^6 repetitions and a wheel load of 9 kips. Analysis of elastic theory computations throughout a spectrum of pavement structures resulted in the curves shown in Figure 6 (5, 6). Figure 7 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.

Figure 1. 1958 Kentucky flexible pavement design curves.

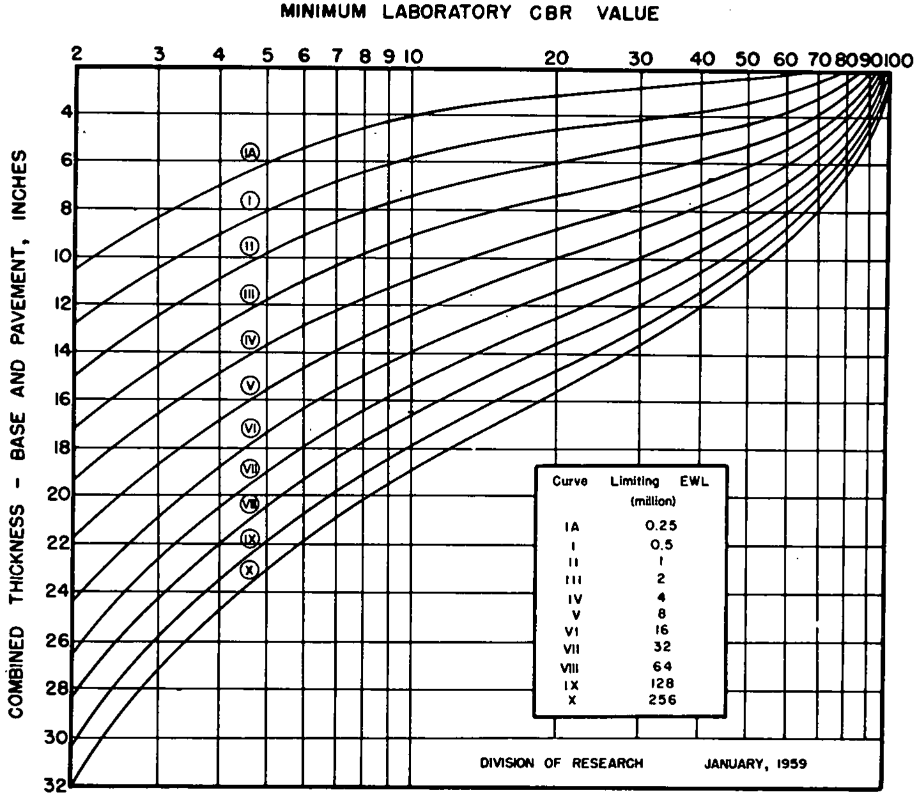


Figure 2. Relation of moduli of subgrade and moduli of granular base.

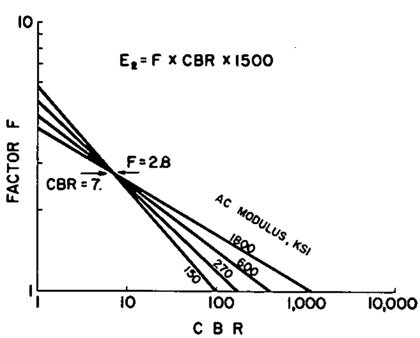


Figure 3. Asphaltic tensile strains-stresses for various structure CBR, and asphaltic concrete moduli of elasticity.

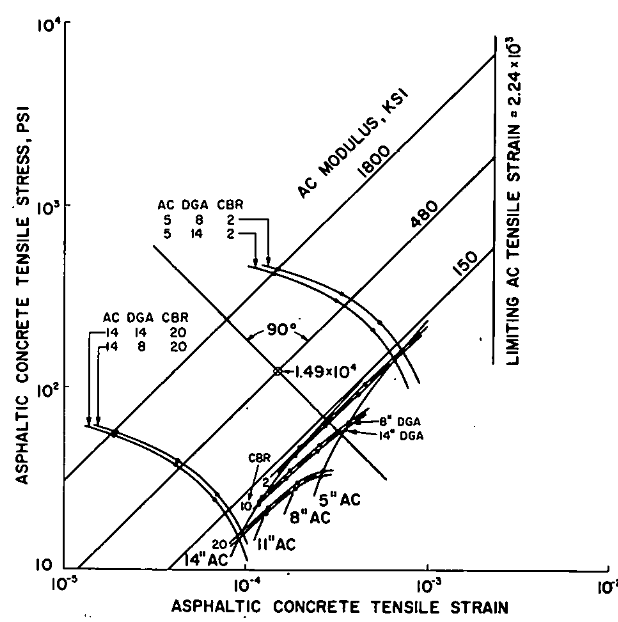


Figure 4. Asphaltic stress-strain curves showing application of strain-control criterion.

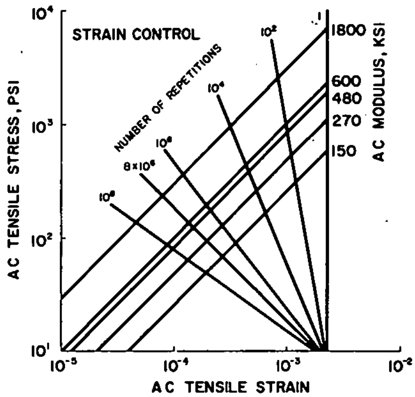


Figure 5. Limiting asphaltic concrete tensile strain as function of number of repetitions and asphaltic concrete moduli of elasticity.

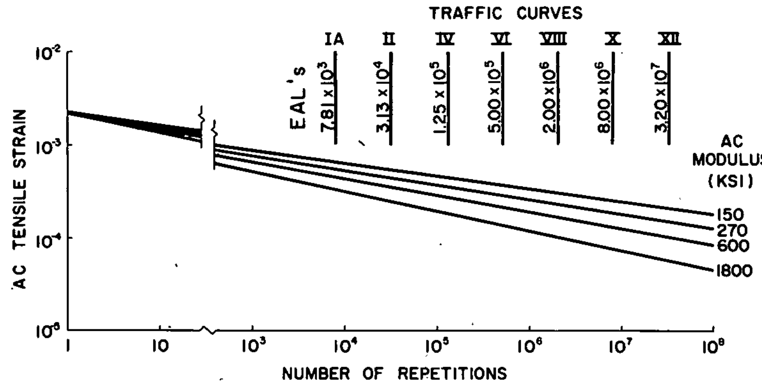


Figure 6. Ratio of subgrade strain to strain under 9-kip wheel load as function of equivalent, hypothetical wheel load.

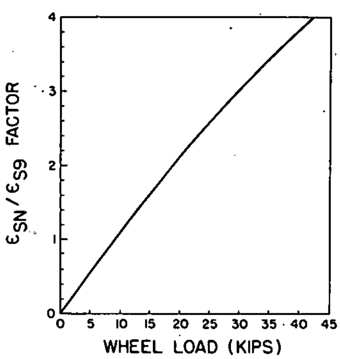
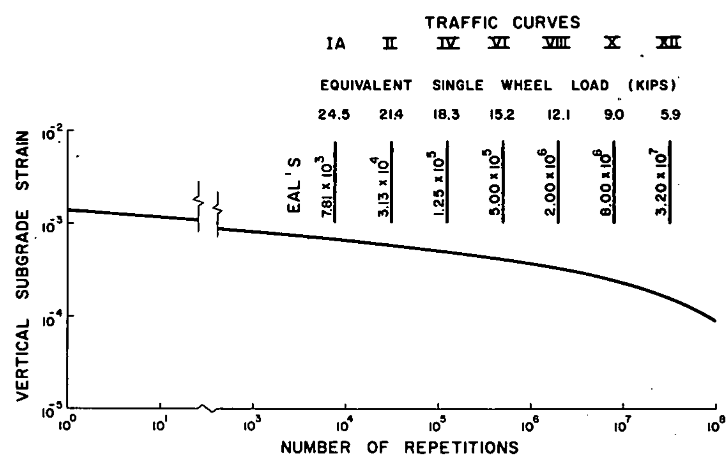


Figure 7. Limiting subgrade vertical compressive strain as function of number of repetitions and equivalent single axle load.



4. To complete the fatigue analysis required that results be plotted in terms of modulus values, layer thicknesses, and so on from influence graphs, satisfying limiting strains. That was done for the following proportions of T_1 and T_2: $T_1 = \frac{1}{3}T$ and $T_2 = \frac{2}{3}T$, $T_1 = \frac{1}{2}T$ and $T_2 = \frac{1}{2}T$, $T_1 = \frac{3}{4}T$ and $T_2 = \frac{1}{4}T$, and $T_1 = T$ and $T_2 = 0$. T_1 = thickness of layer 1, T_2 = thickness of layer 2, and T = total pavement thickness. Coaxial graphs, shown in Figures 8, 9, and 10, 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 Volume Information

Normally, traffic volumes are forecast in connection with needs studies and in the planning stages for all new routes and for major improvements of existing routes. Whereas 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. Historically, actual growths of traffic have exceeded the forecasts in the majority of cases. Overriding predictions of traffic volumes may be admissible for purposes of EAL estimates when properly substantiated. Moreover, the design life of the pavement may differ from the geometric design period.

If only the beginning and twentieth-year AADT is furnished, it may become necessary to request a listing of AADT estimated for each calendar year; otherwise, a normal growth curve must be assumed. In the absence of specific guiding information, a constant yearly increase factor may suffice, typified by the compound interest equation

$$A = P(1 + i)^n$$

where

A = AADT in the nth year,
P = 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 (P + A)/2, which, when multiplied by the number of years, yields a cursory estimate of the total design-life traffic.

Design EAL'S

Heretofore, the Kentucky design system was based on EWL's. The present system is based on EAL's. That transformation was made for the sake of unifying design practices and standardizing definition of design terms. EAL's are defined here as the number of equivalent 18-kip axle loads (22).

Basically, the computation of EAL's involves, first, forecasting the total number of vehicles expected on the road during its design life and, second, multiplying by factors to convert total traffic to EAL's (23). Of course, that is obviously an extreme simplification. More ideally, the yearly increments of EAL's could be calculated and summed; that approach would permit consideration of anticipated changes in legal weight

Figure 8. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 33 percent of total pavement thickness.

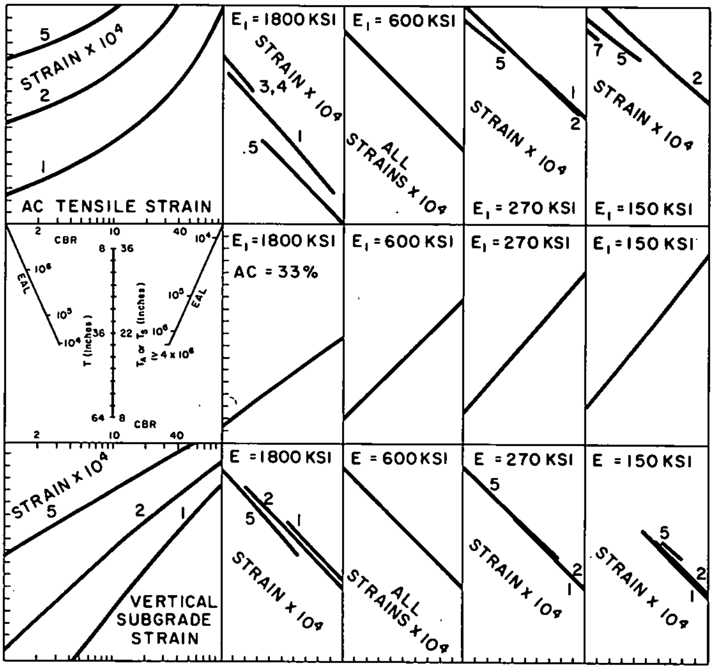
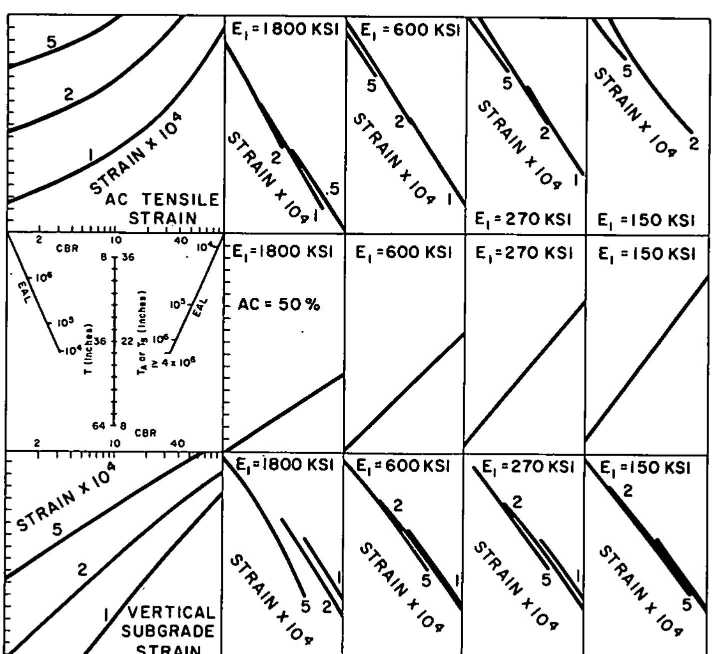


Figure 9. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 50 percent of total pavement thickness.



limits, changes in style of cargo haulers, and changes in routing. If a design life of fewer than 20 years is to be considered or if staged design and construction is foreseen, the EAL value for the respective design period is determined.

The EAL's so determined are gross, 2-directional values that must be reduced to 1-directional values. When more than 2 lanes in each direction are involved, additional factors appropriating EAL's among the lanes will be necessary. No guiding values may be cited, but such values should be available from the planning study report. The necessity of those factors is apparent: It is customary to design all lanes like the most critical one; adjacent lanes of different thicknesses might result in complicating construction procedures. The validity of such a line of argument, however, may be subject to question in the future (24).

Design CBR

CBR test values (3) reflect the supporting strength of the subgrade. Moreover, the test procedure intentionally conditions the soil (by soaking) to reflect its least or minimum supporting strength; that is presumed to be representative of the 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 thereon would be capable then of withstanding heavier loads. If pavements were not designed for the minimum capabilities of the foundation soil, it might be necessary to impose further restrictions seasonally with respect to single axle loads in order to prevent premature and catastrophic failures. However, a pavement should be designed so that it will perform adequately throughout the design period when seasonal variations are considered. To the extent that such performance is represented, the empirical curves shown in Figure 1(and thus the corresponding empirical expressions of failure criterion) represent such designs.

The CBR value does not ensure immunity against frost heave even though it may have a compensating effect in the design of the pavement structure. Greater pavement depths are required for low-CBR soils than for high-CBR soils, and it is usually the low-CBR soils that are more sensitive to frost. A high type of pavement is normally of sufficient thickness that the supporting soil lies below the freezing line (in Kentucky). However, because of the thermal properties of the constituent materials of the pavement, frost penetration in the pavement may be greater than in the adjacent soil mass. For thinner pavements, the supporting soil is well within the frost zone; therefore, the pavement structure providing the greatest template depth is preferred. Pavements less than 6 in. in thickness or having less than 4 in. of asphaltic concrete should be regarded dubiously from this point of view. It is recommended that soil having a CBR of less than 2 be considered ineligible and unsuitable for use as pavement foundation.

Soil surveys may indicate wide variations in CBR along the length of a specific 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. The designer must respect all minimums, or else some sections of pavement will be underdesigned; overdesigns must be admitted as a natural consequence therefrom. The designer is privileged to decide whether to require an intervening low-CBR section to be upgraded to the same quality as that of abutting high-CBR sections or make a separate design for the low-CBR section. Of course, the designer should consider the relative economics of the 2 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.

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 mixture becomes more and more significant.

Initial and preliminary analysis of the performance of Kentucky flexible pavements (thickness being ⅓ asphaltic concrete and ⅔ 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; that corresponds to the modulus at 64 F (the mean annual pavement temperature) obtained from an independent correlation between modulus and average pavement temperature. Weighting distributions of pavement temperature of more than 64 F for various thicknesses of asphaltic concrete suggest that 76 F 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 of the asphaltic concrete according to Southgate and Deen (25) makes it possible to determine and plot (Fig. 11) the moduli corresponding to 64 F (thickness being ⅓ asphaltic concrete) and 76 F (full-depth asphaltic concrete). Based on a straight-line relation, the change in asphaltic concrete modulus as the temperature sensitivity to rutting varies is described as shown in Figure 11. Designs obtained by the use of modulus values shown in Figure 11 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.

Alternative Pavement Thicknesses

1. If the design EAL is known, the limiting subgrade strain can be determined from curves shown in Figure 7. Likewise, Figure 5 shows the limiting asphalt tensile strain values. If a design is desired for an asphaltic concrete with a modulus other than the 4 specifically shown in Figures 8, 9, and 10, it will be necessary to know the limiting asphaltic concrete strain for each of the 4 modulus values so that interpolations can be made later.
2. Enter the top portion (for asphaltic strain control) of Figure 8 at the design CBR. Draw a line vertically to limiting strain values (from Fig. 5) for each E_1; mark each point (Fig. 12).
3. Draw horizontal lines from each of the points obtained above to the respective E_1 modulus quadrants, and 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 8. Only one value of limiting subgrade strain is given for a fixed value of repetitions and is independent of E_1 moduli.
7. Draw a horizontal line to the right through all 4 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 modulus, and fit a smooth curve to the points as shown in Figure 13.
10. Repeat steps 1 through 8 and use Figures 9 and 10.
11. From Figure 13, 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 14. Repeat this step and use T_S from Figure 13.
12. Select from Figure 14 the final design total thickness values for T_A and T_S for the desired ratio of asphaltic concrete thickness to total thickness.

Figure 10. Nomograph for analysis of vertical compressive strains at top of subgrade and tensile strains at bottom of asphaltic concrete layer comprising 100 percent of total pavement thickness.

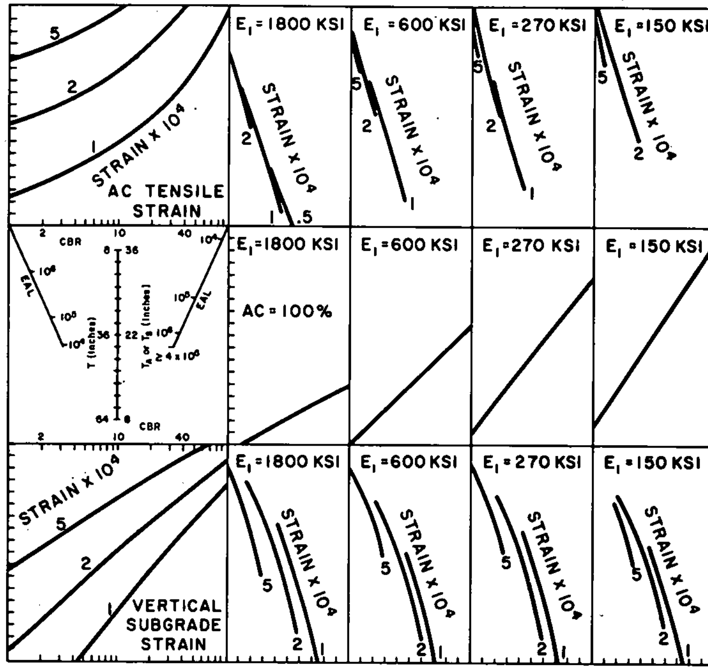


Figure 11. Weighting of asphaltic concrete modulus for ratio of thickness of asphaltic concrete to total pavement thickness.

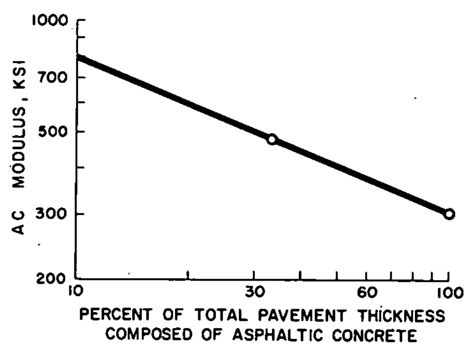


Figure 12. Illustration of use of Figure 8.

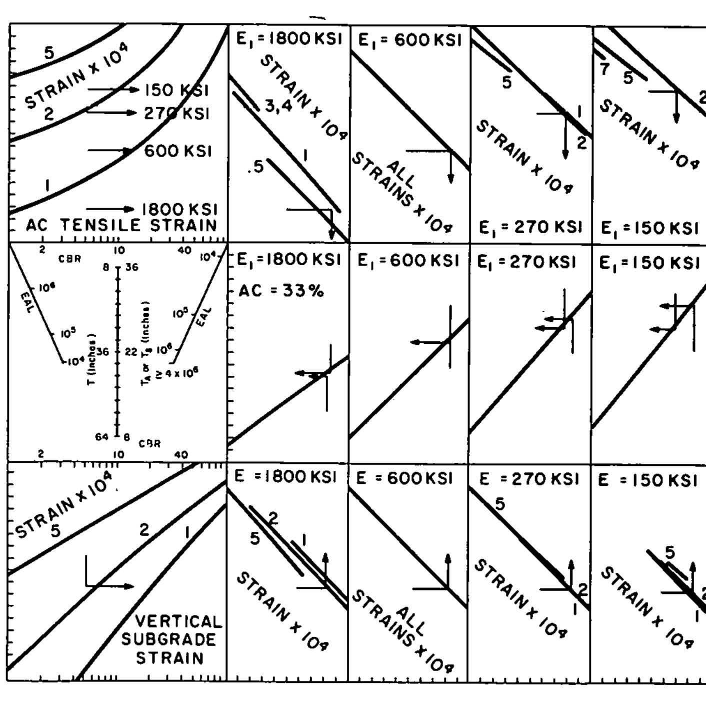


Figure 13. Total pavement thickness, T_A and T_S, as function of asphaltic concrete modulus.

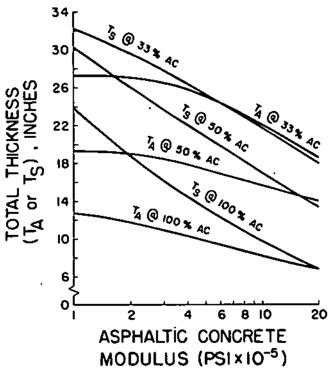


Figure 14. Total pavement thickness, T_A and T_S, as function of ratio of thickness of asphaltic concrete to total thickness.

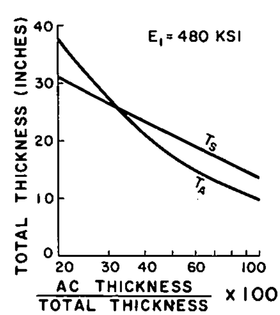


Figure 15. Nomograph to adjust design thicknesses for rutting.

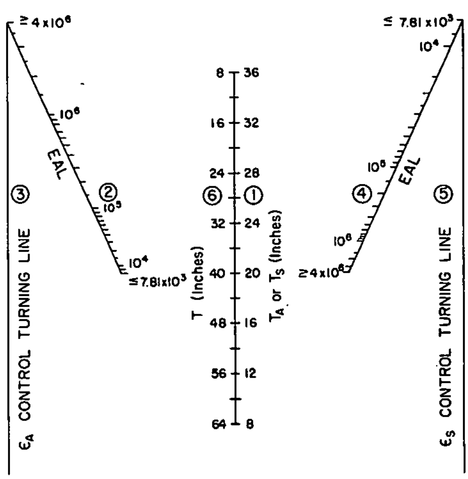
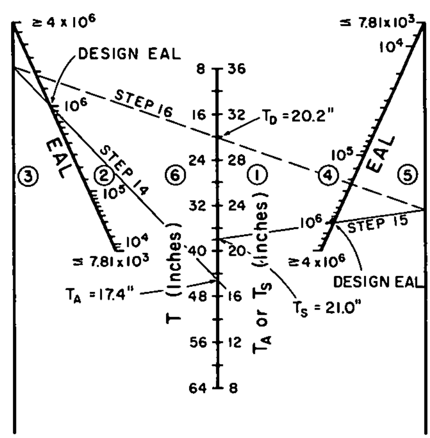


Figure 16. Illustration of use of Figure 15.



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.

Rutting of Subgrade

Whereas the respective design curves provide equal assurances against rutting throughout all ranges of EAL's, greater rutting is tacitly and progressively admissible in some inverse relation to EAL's. It has been presupposed that no additional rutting should be allowed in pavements having design EAL's equal to or greater than 4×10^6. On the other hand, it seemed that a pavement having a design EAL equal to or less than 7.81×10^3 might be allowed to rut in a completely uncontrolled manner. Weighting the intervening curves in relation to EAL's permitted construction of a nomograph (Fig. 15) for those designs where rutting criteria control. It is suggested that this weighting be respected in an advisory way. It may be violated permissively in either direction, provided the fatigue limit of the asphaltic concrete layer is respected.

Figure 15 is used to adjust for rutting when the design EAL is more than 7.81×10^3 and less than 4×10^6. The final design thickness adjusted for rutting is obtained from the following procedure:

14. For the desired ratio of asphaltic concrete thickness to total thickness shown in Figure 14, read the total thickness (T_A for asphaltic concrete strain control), and mark on scale 1 in Figure 16. Draw a straight line from T_A on scale 1 through the design EAL value on scale 2, and mark the intersection point on line 3.
15. For the desired ratio of asphaltic concrete thickness to total thickness shown in Figure 14, read the total thickness (T_S for subgrade strain control), and mark on scale 1. Draw a straight line from T_S on scale 1 through the design EAL value on scale 4, and 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.

CONCLUDING REMARKS

To determine pavement thicknesses from the nomographs similar to those shown in Figures 8, 9, and 10, one must know design EAL's, CBR of the subgrade soil, and modulus of elasticity of the asphaltic concrete. Such a set of nomographs permits selection of pavement structures employing alternative proportions of bituminous concrete and crushed stone base. Total thickness varies according to the proportion chosen. However, the choice may not be made arbitrarily or trivially. It is implicitly intended that the final selection also be based on additional engineering considerations such as estimates of comparative construction costs, compatibility of cross-sectional template and shoulder designs, uniformity of design practices, highway system classifications, engineering precedence, and utilization of indigenous resources. Designs based on 33 percent and 67 percent proportions of bituminous concrete (asphaltic concrete modulus of 480 ksi) and crushed rock base respectively conform with the department's current design chart, representing current, conventional, or precedential design. The nomographs (Figs. 8, 9, and 10) represent theoretical extensions of conventional designs and, from a theoretical standpoint, provide equally competent structures.

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