Development of a Thickness Design System for Bituminous Concrete Pavements

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THICKNESS DESIGN SYSTEM
FOR BITUMINOUS CONCRETE PAVEMENTS

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INTRODUCTION

A pavement provides a functional surface for the safe operation of a vehicle. The operator or passenger of a vehicle does not particularly care about the material from which the pavement structure is constructed. However, they are sensitive to such factors as speed, safety (skid resistance), and comfort (roughness). One aspect of pavement design is the selection of the thickness of the pavement and its various components sufficient to support vehicular loadings and to transfer those loadings through successive layers of the pavement — surface, base, and subgrade — to the soil on which the pavement rests. The structural design of a highway pavement involves a study of the soils, paving materials, and their behavior under load. The pavement structure must be adequate to support the wheel loads of motor vehicles. Each time a vehicle passes over a pavement, some stressing and straining of the surface and underlying layers occur. If the load is excessive or if the supporting layers are not sufficiently strong, repeated applications of the vehicular loadings will cause rutting and cracking that ultimately lead to a complete structural failure of the pavement. The pavement thickness design scheme suggested in this report provides a procedure by which the load-carrying capabilities of any individual layer or of the soil upon which the pavement rests are not exceeded.

Prior to 1948, pavement thicknesses were based upon curves developed by the California Department of Highways in 1942. The soil support was expressed in terms of California Bearing Ratio (CBR). The first set of thickness design curves for Kentucky (1) was developed in 1948. Extensive laboratory tests of soils were performed using several methods, and the CBR method was chosen as the basis for evaluating soil strength. Extensive evaluations of pavement performance were made; pavements were trenched; and in-place bearing tests were made. Performance correlated best with minimum laboratory CBR’s. Traffic histories were estimated for the pavements. Those pavements with approximately the same traffic histories were grouped, total pavement thickness was plotted versus CBR, and data points were coded “good” and “bad.” Best-fit curves were drawn to separate the failed from the unfailed. The 1948 curves were based on failure boundaries and performance envelopes.

By 1957, a need to update the 1948 curves and to extend the curves to higher traffic loadings became apparent. Another extensive series of field tests and analyses resulted in the 1959 Kentucky design curves (2), which are the curves in use today (September 1981). Those curves were in use prior to the AASHO Road Test, were verified by field tests, and had precedence to the AASHO thickness design system that was developed later.

The need to update the 1959 Kentucky curves resulted in a research study in 1968. By then, computers had been developed with enough sophistication, speed, and memory capacity to permit the development of the N-layer program to analyze pavements using elastic theory (3). That program requires input data for such variables as load, tire pressure, number of layers, layer thicknesses, and material properties; stresses, strains, and deflections are calculated. Some assumptions required modification; ultimately, the 1973 Kentucky design curves (4) were prepared. Experience confirmed the design curve for pavements with 1/3 of the thickness being asphaltic concrete and 2/3 being dense-graded aggregate. Based upon elastic theory, thickness curves were created for other proportions, but they remained unconfirmed by extensive field test data. Experimental pavements were designed, constructed, and tested beginning in 1971. Field test data have been matched with theoretical solutions and confirms both thickness curves and the method of estimating pavement fatigue caused by traffic loadings.

ELEMENTS OF DESIGN PROCEDURE

SUBGRADE SUPPORT

Several procedures were utilized in the 1948 testing program to evaluate the load-carrying capacity of the subgrade and consisted of plate bearing, dutch cone, in-place CBR, and the soaked laboratory CBR tests. The best correlations of pavement performance (1) were found with the soaked laboratory CBR, which differs from the ASTM method in one aspect. ASTM specifies the sample be soaked for three days; the Kentucky method allows soaking until swelling ceases. The soaked CBR was the basis for the 1959 curves (2) and still is the basis for the 1981 curves (5). Correlations have been made with the AASHTO soil support scale and elastic moduli on the basis of field test data and the Chevron N-layer computer program. A literature review (6, 7) indicated that the elastic modulus of clay soils could be estimated from laboratory tests by

$$E = 1500 \times CBR,$$
in which

\[ CBR = \text{value obtained by the Kentucky procedure and} \]

\[ E = \text{elastic modulus of the subgrade.} \]

Chevron N-layer analyses require elastic moduli as input. Research indicates the factor "1500" is valid for clays, but possibly is different for sands, gravels, rock, etc. Thus, the factor of "1500" may be changed and a new scale fitted, but the design thicknesses, based upon elastic theory, remain valid. Soil at the AASHO Road Test was assigned a soil support value of 3.0. Samples of soil from the Road Test were received and subjected to the Kentucky CBR test procedure. The equivalent CBR was 5.2/8).

CHARACTERISTICS OF PAVING MATERIALS

Input to the Chevron N-layer computer program consists of the thickness, elastic modulus, and Poisson’s ratio for each layer.

Asphaltic Concrete – Factors affecting the modulus of asphaltic concrete are percent by weight of asphalt cement, percent voids (density measurements), frequency of applied load, and the temperature distribution in the pavement. Construction practices that meet current standard specifications will produce high-quality pavements and have an elastic modulus equal to 480,000 psi for Kentucky conditions. However, should compaction procedures be neglected, producing voids twice that of the design mix, for example, the elastic modulus will be reduced approximately 40 percent. Likewise, small increases in asphalt cement content will weaken the modulus significantly (9, 10).

Temperature distributions within the pavement significantly affect distribution of elastic moduli in the asphaltic concrete layer. Pavement and air temperature histories have been recorded at the AASHO Road Test, in Kentucky, in upper New York state, in Arizona, and in Maryland. A method was developed (11, 12) to estimate temperature distributions in the asphaltic concrete layer. Laboratory tests (13) provided the relationship of temperature and elastic moduli. Computer runs were analyzed and deflection adjustment factors (11) developed to account for temperature variations. Adjusting surface deflections to a reference temperature (70°F in Kentucky) significantly reduced the scatter of data and permitted rational analyses.

Dense-Graded Aggregate – Kentucky is blessed with high-quality limestone and sandstone aggregates that may be crushed to produce a dense-graded product with very low void contents. Dense-graded aggregate has a very low tensile strength, attributable to a small amount of cementation. A small movement destroys any cementation effects.

The 1968 analyses assigned one modulus value to the dense-graded aggregate without regard to CBR value. Later analyses were made allowing the modulus of the dense-graded aggregate to vary as a function of the moduli of the confining layers. The structures represented by Curve X (1959 curves (2)) were given various crushed-stone moduli and subjected to analyses by the Chevron program. Figure 1 (14, 15) illustrates the relationship of dense-graded aggregate modulus as a function of the asphaltic concrete and subgrade moduli (CBR). Thus, the modulus of the dense-graded aggregate is a function of the confining layers. When the subgrade has a high modulus, the load-carrying capacity of the dense-graded aggregate is very good. However, if the subgrade is very weak, even the highest quality dense-graded aggregate will not develop its full potential modulus because the particles move around relative to each other.

Equivalency Factors – Correlating results of N-layer solutions and design criterion based upon elastic theory have permitted matching field test data to the thickness design curves. Thickness design curves were developed for various ratios of thickness of asphaltic concrete to the total thickness. Maintaining the same strain-repetitions criterion results in equivalent designs based upon an expected behavior. Comparisons of these designs show that “equivalency factors” vary with the design traffic and design CBR. Equivalency factors may vary from 1.25 at a low CBR to over 15 for CBR’s over 15. Equivalency factors may only be used for a specific choice of fatigue life and CBR and is not a constant value.
The AASHTO equivalency factors \(16\) were developed from the AASHO Road Test \(17\) data, but there was only one subgrade involved in the Road Test. Therefore, the "equivalency factor" is valid only at that CBR or soil support value.

**TRAFFIC**

Traffic Stream -- The total traffic on a pavement is a composite of many styles and sizes of vehicles carrying a wide range of loads. Pavements fatigue under loading, whether the load is legal or illegal. Ignoring the existence of illegal loads results in under-designed pavement thicknesses that contribute to "premature failure". In most cases, repair costs will exceed costs that would have occurred at the design and construction stage.

Axle configuration combined with the load on those configurations can greatly reduce or increase the accumulation of fatigue to the pavement \(18\). Adding axles to a given vehicle may, or may not, reduce fatigue life of the pavement. A controlling factor is the suspension system for that configuration. One that can distribute the load equally to all axles of the configuration will reduce the fatigue damage to the minimum for that number of axles. So called "drop axles" have a high probability of causing an uneven distribution of load among the axles in the closely-spaced group. If the axle is lowered so almost no load is carried by that axle, then the damage caused by the remaining axles of that group will be severe compared to the equal loading situation. An investigation of 375 352 (five-axle semi-trailer) trucks listed in the 1977 Kentucky W-6 tables indicated a 40-percent increase in fatigue damage caused by unequally loaded axles within the tandem groups compared to equally loaded axles \(18\).

The axle configuration is significant in inducing fatigue damage caused by the total load on that vehicle. For example, 80,000 pounds on a three-axle single-frame dump truck causes 20 18-kip EAL's of damage. The same load properly distributed on a 352 causes 2.5 18-kip EAL's and 1.3 EAL's on a 353 truck \(18\).

The number of tires on an axle also causes a variation in fatigue damage. Two single wide tires will cause more damage than four regular tires \(18\). One 18-kip EAL is caused by 18,000 pounds on a single four-tired axle or by 14,000 pounds on a single two-tired axle (steering axle). Researchers at the AASHO Road Test observed the same behavior, but the magnitudes might have been different.

Through the years, the size as well as the number of trucks on the highways have been increasing. Development of the interstate system generated more traffic than was dreamed possible. Increasing the legal load limits greatly increased the rate of fatigue damage. In 1974, Congress raised the legal limit from 73,280 pounds to 80,000 pounds. For a 352 vehicle, raising the legal load by approximately 10 percent caused a 70-percent increase in the fatigue damage \(18\).

Economic analyses have been reported by FHWA \(19\) advocating raising the total legal limit in 1985 from 80,000 pounds to 120,000 pounds to conserve fuel. An investigation should be made to specify the style of truck that will minimize the fatigue damage to the highway system -- bridges and pavements -- prior to raising the legal limit.

**Estimating Equivalent Axleloads** -- Knowing only the volume of traffic will not permit adequate forecasting of the expected fatigue life. The number of trucks of a given classification also must be estimated. Yet, some trucks will be empty, and others will be loaded up to and even in excess of the legal limit. Thus, all damage must be expressed in relative terms of a specified axleload. The 18-kip single axleload has been selected as the reference load \(17\), and this has become the most widely used reference load in the United States and abroad.

The 5-kip equivalent wheel load (EWL) was proposed by California and adopted by Kentucky in 1948 \(1\). The 1959 Kentucky design curves \(2\) are also based upon the 5-kip EWL.

The use of the 18-kip equivalent axleload as the reference single axleload at the AASHO Road Test prompted the 1968 Kentucky investigation. The AASHTO fatigue equation is expressed as a log-log equation \(16\), but the Kentucky expression took a semi-log form with repetitions plotted on the log scale. The constants in the AASHTO equation were developed as a correlation of repetitions, load, and a pavement serviceability of 2.5. A pavement reached a serviceability of 2.5 when

- a. the rut depth was measured as 0.375 inches
- b. cracking occurred over the width of the two wheel tracks (40 percent or 5 feet of the 12-foot width).

Input to the AASHTO equation consists of the number of 18-kip single axleloads, the serviceability level \(P_t\), the structural number that describes the pavement thickness, the magnitude of load on the axle configuration, and a value of 1 or 2 as a code for single or tandem axle configurations, respectively. The calculated result is the number of 18-kip EAL's. Damage factor is defined as the calculated number of 18-kip EAL's divided by the reference number of 18-kip EAL's.

Choosing the number of 18-kip single axleloads, the AASHTO equation was solved for the number of
18-kip EAL's with respect to other loads, single axles, and tandem axles. Plots were made of the loads and their respective calculated number of repetitions.

Curve X of the 1959 Kentucky design curves (2) was associated by field experience with 8 x 10^6 18-kip EAL's. Therefore, the semilog relationships chosen by Kentucky were made tangent to the AASHO load-repetition curves for their respective axle groups at P_2 = 2.5.

Fatigue Criteria - The literature (13-15) provided fatigue criteria determined by laboratory testing. The criterion related magnitudes of tensile strains at the bottom of the asphaltic concrete to repetitions of a reference load. Another criterion involved the vertical compressive strains at the top of the subgrade related to repetitions of a reference load. Most design systems reported to date involve these two components of strain at their respective locations.

Strain energy concepts were applied in 1979 to analyses of pavements (18, 20). The displacement of a pavement caused by a load is called WORK. The internal resistance by the pavement to that load is called STRAIN ENERGY. The advantage of this concept lies in that all components of strain, instead of just one, at a specific location in the pavement are taken into account. In the majority of pavements subjected to analyses by the N-layer program, the greatest strain was the tangential strain at the bottom of the asphaltic concrete. However, the radial strain at the same location was almost as large in magnitude but was ignored. The strain-energy approach uses all nine strain components, but four have a value of zero. A minor mathematical operation permits expressing strain energy as "work strain". Direct correlations may be made between "work strain" and any component of strain. Correlations of tensile strain to work strain at the bottom of the asphaltic concrete and vertical compressive strain to work strain at the top of the subgrade provided the relationship between fatigue and work strain.

Variation of load on a particular pavement structure yields respective magnitudes of strains. Damage factors associated with loads were calculated from this relationship of strain and repetitions. Using superposition principles, tire and axle configurations duplicating the AASHO Road Test (21) were analyzed by the N-layer program for pavement thicknesses used at the AASHO Road Test. Computer analyses permitted the development of damage factor relationships for steering axles, two-tired single axles, eight-tired tandem axles, 12-tired tri-axles, 16-tired quad-axles, 20-tired five axles, and 24-tired six axles groups. These damage factor curves allow the analysis of a specific vehicle by determining the load on each group of axles, obtaining that damage factor, and summing all damage factors for that vehicle. This feature is particularly important for the steering axle. The front axleload on Loop 4, Lane 1, was 5,600 pounds and corresponds to a damage factor of 0.045. Most "cab-over" tractors have steering axleloads of 12,000 pounds, or a damage factor of 0.70. Wide flotation tires will carry 20,000 pounds for a damage factor of 1.35. Therefore, modern truck designs require the assignment of appropriate damage factors. Analyses of the test vehicles at the AASHO Road Test showed that the total damage factors for the vehicle determined by "work strain" concepts were almost identical to the damage factors assigned at the AASHO Road Test.

Nonuniform loading of axles within the same group was investigated using "work-strain" principles. The 1976 W-6 tables for Kentucky indicated that only 10 percent of all tandems had the load distributed equally between the two axles. Forty percent of the tandem axles exceeded 4,000 pounds difference between the two axles. The net result showed a 40-percent increase in fatigue damage because of uneven load distribution on the two axles versus evenly distributed loads (18). These suspension systems on most trailers in use today do not divide the load equally over the two axles of that tandem. Data furnished by the Washington Office of FHWA confirm the uneven loading of axles in a tri-axle group.

FIELD PERFORMANCE

EARLY KENTUCKY EXPERIENCE

The 1948 Kentucky design curves (1) were drawn to separate adequately performing pavements from failed pavements and, thus, were field performance curves. That set consisted of five curves for five levels of traffic.

As traffic volumes and vehicular weights increased and more experience was gained, additional curves were needed. A series of pavements were opened, thicknesses determined, in-place soil tests performed, and samples taken for laboratory tests. Increased existing traffic volumes provided three additional curves.

The Kentucky laboratory CBR tests conducted in 1957 on samples from throughout the state yielded a mean value of 7. Curves for three additional traffic
levels were thought to be required, and the thicknesses were determined by extrapolation. Curve X represented $8 \times 10^6$ 18-kip EAL's, and the extrapolated thickness was 23 inches at CBR 7. That thickness consisted of 33 percent asphaltic concrete and 67 percent dense-graded aggregate. The 23 inches were considered to be much too thick, thus Curve X was reduced to 21 inches and Curves VIII and IX proportioned accordingly \( (2) \). By 1968, traffic volumes and vehicular weights had increased until it was thought that Curves XI and XII needed to be added. An extensive series of Benkehn beam deflection tests were conducted at sites throughout the state. Pavement temperature affected the test results dramatically. As an example, the same site was tested from 8 a.m. until 5 p.m. and the deflections varied from 0.015 inch at 8:00 a.m. to 0.045 inch at 2 p.m. to 0.035 inch by 5:00 p.m. A method was developed \( (11, 12) \) to estimate the temperature distribution and adjust the deflections to an equivalent value at a reference temperature. This reduced the scatter of data, but enough scatter remained to cause confusion. Thus, pavement deflection was not the key attribute; only layered-system analyses offered the necessary insights into pavement mechanics.

DEVELOPMENT OF RATIONAL DESIGNS

At this point, Chevron Research Company, a Division of Chevron Oil Corporation, provided a privileged copy of their N-layer computer program \( (3) \) that is based upon elastic theory. Computer simulations of Curve X produced too large a surface deflection. However, the deflection calculated due to a 9-kip wheel load (one circular area representing two-tires) for a 23-inch structure consisting of 33 percent asphaltic concrete and 67 percent dense-graded aggregate on a CBR 7 subgrade did match the deflection associated with 23 inches consisting of 33 percent asphaltic concrete and 67 percent dense-graded aggregate on a subgrade that is based upon elastic theory. Computer simulation is shown in Figure 2. Overlaying the 1959 traffic curves of equal subgrade strain, particularly for Curve X, verified the original extrapolation of 1957 field test data.

Curves of dense-graded aggregate thicknesses were constructed relating asphaltic concrete thickness to surface deflection, horizontal tensile strain at the bottom of the asphaltic concrete layer, and vertical compressive strain at the top of the subgrade. For fixed values of strain or deflection, layer thicknesses for a fixed percentage of asphaltic concrete could be determined for a specific CBR. Solutions for various values of CBR produced families of strains and deflections as shown in Figure 2. Overlaying the 1959 traffic curves indicated the curves "of equal behavior" were not curves of equal surface deflection, but represented curves of equal subgrade strain, particularly for Curve X in the range of CBR from 3 to 12. This was the range encountered in the in-place CBR tests conducted during 1957.

Although the modulus of the dense-graded aggregate was fixed initially at 25,000 psi, this was determined to be an erroneous assumption. Figure 1 illustrates the relationship of $E_2$, the modulus of the dense-graded aggregate, as a function of the moduli of asphaltic concrete and subgrade, the two confining layers. The $E_1$, $E_2$, $E_3$, relationship (Figure 1) was confirmed using the thicknesses of the 1959 Curve X and adjusting $E_2$ to obtain the same vertical compressive strain at the top of the subgrade.

Having determined that Curve X of the 1959 design curves was based upon equal subgrade strains, a literature search disclosed fatigue-strain relationships \( (6) \) determined from laboratory tests. The vertical compressive strain at the top of the subgrade related to fatigue associated with an 18-kip equivalent single axleload \( (6) \) is shown in Figure 3. The horizontal tensile strain at the bottom of the asphaltic concrete was related to fatigue caused by an 18-kip single axleload \( (13) \) and is illustrated in Figure 4/4.

Utilization of the two fatigue criteria was based upon the following assumptions:

a. Farm-to-market roads should have the least asphaltic concrete thickness controlled only by the magnitude of the horizontal tensile strain in the asphaltic concrete -- associated with 7,300 repetitions of an 18-kip single axleload. This assumption allows rutting to occur uncontrolled in the subgrade.

b. For $4 \times 10^6$ or more repetitions of an 18-kip single axleload, pavement thicknesses should be controlled only by the magnitude of the vertical compressive strain at the top of the subgrade. This assumption provides a high assurance that there will be no rutting in the subgrade.

c. Figure 5 illustrates the fatigue relationship between the two criteria for the intermediate range between Conditions a and b above. This assumption allows rutting to occur in the subgrade between the extremes of Conditions a and b as a function of traffic.

The matrix of iso-dense-graded aggregate thickness curves for strain versus thickness of asphaltic concrete were combined with the fatigue criteria to obtain the thicknesses for a constant modulus of asphaltic concrete and a constant ratio of asphaltic concrete thickness to the total thickness to produce a set of charts of the form shown in Figure 6.

Analysis of Kentucky weather records indicated the mean annual temperature was 64°F, but the influence of summer temperatures raised the "design temperature" to 70°F. Figure 7 gives the relationship of temperature and moduli of asphaltic concrete for a frequency of 0.5 Hz -- the approximate speed for a
Figure 2. Total Thickness versus Kentucky CBR and Subgrade Modulus of Elasticity, Illustrating the Change in Thickness for Constant Strain and Deflection Values.
Figure 3. Limiting Subgrade Vertical Compressive Strain as a Function of Number of Repetitions and Equivalent Single Axle-load.

Figure 4. Limiting Asphaltic Concrete Tensile Strain as a Function of Number of Repetitions and Asphaltic Concrete Modulus of Elasticity.

Figure 5. Adjustment of Design Thickness for Rutting as a Function of Repetitions of 18-kip EAL's.

Figure 6. Total Thickness versus Kentucky CBR and Subgrade Modulus for Pavement Structures Consisting of 33 Percent Asphaltic Concrete and 67 Percent Dense-Graded Aggregate for Asphaltic Concrete Modulus of 600 ksi.
Dynamic tests before and after overlays have provided significant data to verify three-layer pavement designs varying from very thin to thick interstate pavements. Overlay thicknesses have ranged from 1 to 6 inches. Recent verifications have resulted from testing full-depth asphaltic concrete pavements on US 60 in Boyd County, on a portion of the west end of the

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**Figure 7.** Asphaltic Concrete Modulus versus Average Pavement Temperature for Frequency of 0.5 Cycles per Second.

creep-speed Benkelman beam test. For 70°F, the corresponding modulus is 480 ksi. Thus, for a fixed ratio of asphaltic concrete thickness and a constant CBR, Figure 8 is a typical example relating total thickness and asphaltic concrete moduli for a fixed value of fatigue. Interpolating for a modulus of 480 ksi gave the required thickness for that level of fatigue, and combining thicknesses for other levels of fatigue produced thickness design charts. Figure 9 is an example for structures comprised of 33 percent asphaltic concrete thickness and 67 percent dense-graded aggregate thickness. Thus, Figure 9 relates elastic theory to past experience for a pavement structure consisting of 33 percent asphaltic concrete.

Kentucky had limited experience with full-depth asphaltic concrete pavements. Thus, three sets of thickness curves were created for full-depth pavements. Each was associated with a pavement temperature and the corresponding modulus. Figure 10 relates temperature, percentage asphaltic concrete thickness of total thickness, and modulus of asphaltic concrete. Using Figure 10, thickness design curves were prepared for fixed ratios of asphaltic concrete thickness to total thickness. The modulus of the asphaltic concrete is determined by the line ending at the full-depth modulus. That modulus is used to enter the series of charts (Figure 8) to obtain the thicknesses for corresponding CBR's associated with a value of repetitions. A thickness design guide (4) presented the three sets of thickness designs as tables of thicknesses for fixed levels of fatigue.

The three sets of curves were created because of the uncertainty about the relationship between temperature and rutting. Comments had been made to the effect that higher temperatures would soften the asphaltic concrete, pavements would rut, and damage would extend into the subgrade. Thus, choosing a weaker design modulus had the effect of producing thicker pavements.

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**Figure 8.** Total Thickness versus Asphaltic Concrete Modulus for Fixed Level of Fatigue, 33 Percent Asphaltic Concrete, and Iso-CBR Values.
Cumberland Parkway, and on the Daniel Boone Parkway. Deep trenches were cut into the 17-inch full-depth asphaltic concrete pavement on the Daniel Boone Parkway, an 18-inch full-depth section of US 60 in Boyd County, and the three-layer pavement on I 64 in Boyd County (7.5 inches of asphaltic concrete over 14 inches of dense-graded aggregate). In all cases, rutting never exceeded a depth of 6 inches below the surface. Therefore, rutting in thick asphaltic concrete pavements is a mix-stability problem and not a structural problem as had been thought.

Elastic theory has been applied in simulating dynamic testing (22-24). Ten years of dynamic testing using Kentucky’s Road Rater and Benkelman beam tests have confirmed that in Kentucky pavements have according to the thickness design curves having a modulus of 480 ksi for all ratios of asphaltic concrete thickness to the total thickness. Another report (25) presented all equations required for computerization of a method to automate the analyses of dynamic test data. Equations are presented to adjust dynamic test data recorded by Dynaflect testers to equivalent Road Rater results. A limited number of Dynaflect test data have been analyzed successfully by this procedure. Thus, matching of field test data using elastic theory with the thickness design curves developed from the same theory provides solid proof and verification of the thickness design curves based upon a modulus of 480 ksi.

Figure 9. Thickness Design Curves for Pavement Structures Having 33 Percent Asphaltic Concrete Thickness of the Total Thickness.
Another source of verification for the 480-ksi curves has been data from the AASHO Road Test (17, 26). Applying Kentucky damage factors to the repetitions recorded at the Road Test and plotting them versus their respective AASHTO Structural Number indicated the following results:

a. Converting Kentucky thickness design curves to an equivalent Structural Number indicated the Kentucky curves were essentially parallel to the AASHO equation (16) (Figure 11) but required greater thickness.

b. The AASHTO regression equation is a least-squares fit and as such passes through the middle of the Road Test data. Because the equation is a relationship between fatigue and pavement thickness expressed as "Structural Number," then 50 percent of all pavements will fail prematurely. Kentucky's design curves require greater thicknesses and will perform longer than designs obtained by the AASHTO Interim Guide (16). Proof of this fact is that Kentucky's interstate pavements have performed adequately for at least 16 to 18 years while those pavements designed by the AASHTO Interim Guide have reached their design fatigue life in 10 to 12 years.

OVERLAY DESIGNS

Kentucky's overlay design method (27) was used first in the 1977 RRR estimate (28). For interstate pavements, it was assumed that the pavement would be scheduled for overlaying when the serviceability index, obtained from roughness measurements, reached 3.5 so that actual paving would occur at least by the time the serviceability index had deteriorated to a level of 3.25. Other highway segments would be allowed to reach lower serviceability indices compatible with the highway system of which it is a part.

Kentucky's overlay design method has been developed using elastic theory as expressed by the Chevron N-layer program (3). The expected traffic expressed as repetitions of an 18-kip equivalent axle-load is used to determine a design thickness from the design curves for several ratios of asphaltic concrete thickness to the total thickness as shown by Curve A of Figure 12. The existing thickness of crushed stone base is used to determine the total thickness to be required and is identified as Curve B of Figure 12. The intersection of Curves A and B yields the total design thickness consisting of the existing crushed stone base and the remainder as asphaltic concrete. Dynamic
testing by the Road Rater provides the necessary data to determine the behavioral thickness of existing asphaltic concrete layers in terms of "reference-quality" material. The effective total thickness of the in-place pavement is subtracted from the total design thickness; the difference is the overlay design thickness. This method has been verified by Road Rater tests of pavements both before and after overlays were constructed and again confirms that pavement performance matches the 480-ksi curves.

Strain energy principles have been applied to the development of "damage factors" for various groups of axles \((18, 20)\) in use today. Those factors were developed for the same levels of loadings, the same axle configurations, the same pavement thicknesses, and the same material parameters as those at the AASHO Road Test \((21)\). The method allowed the development of damage factors for the "steering axle" as well as other groups of axles \((18, 20)\). Analyses have developed additional damage rates for axles in a tandem group but which carry unequal loads. An analysis of 375 3S2 trucks for which weight was recorded by axle were taken from a Kentucky W-6 table, and fatigue damage was calculated for each vehicle and accumulated. Damage factor relationships by strain energy analyses and by the AASHTO Interim Guide \((P_t = 2.5, SN = 5.0) / 16/6\) were compared. The accumulated fatigue by strain energy was approximately 90 percent of the fatigue calculated using the AASHTO damage factors.

**FUTURE WORK**

Laboratory fatigue tests of asphaltic concrete \((29)\) confirmed that total work of an asphaltic concrete pavement is expressed by

\[
W_t = N(1/2 \sigma e),
\]

in which

- \(W_t\) = total work and
- \(N\) = number of repetitions causing the stress, \(\sigma\), and strain, \(e\).

Substituting \(Ee\) for \(\sigma\), Equation 1 becomes

\[
W_t = N(1/2 Ee^2),
\]

in which

- \(E\) = Young's modulus of elasticity.

Dividing Equation 2 by \(N\) gives the amount of work per cycle:

\[
Work/cycle = 1/2 Ee^2
\]

or

\[
e = (2 work/cycle)^{0.5} \div E.
\]

Strain energy density \((30)\) is defined by

\[
SE = (\lambda \nu^2 / 2 + \mu (e_{11}^2 + e_{22}^2 + e_{33}^2) + 2e_{12}^2 + 2e_{23}^2 + 2e_{13}^2),
\]

in which

- \(SE\) = strain energy,
- \(E_{ij}\) = \(i\)th, \(j\)th component of the strain tensor,
- \(\mu\) = \(E/(2(1+\nu))\), the modulus of rigidity or the shear modulus,
- \(\lambda = Eo/(1+\nu)(1-2\nu)\),
- \(\nu\) = Poisson's ratio, and
- \(\nu = e_{11} + e_{22} + e_{33}\)

Inspection of Equation 4 shows that the factor \(E/(2(1+\nu))\) is contained directly or through \(\lambda\) and \(\mu\). Also, it is noted that the strain components are squared. Having calculated strain energy density, work strain may be obtained from
\[ \varepsilon_w = (2SE/E)^{0.5} \]

in which

\[ \varepsilon_w = \text{work strain}. \]

The associated "work stress" is given by

\[ \text{Work Stress} = E\varepsilon_w. \]

Admittedly, work strain is not a true strain because Poisson's ratio has not been eliminated prior to taking the square root; however, it is of the same order of magnitude as any of the strain components. Work strain is also the composite, or net effect, of all strain components and thus is an indicator of the total strain behavior.

Some thickness design systems for flexible pavements are based partially upon tensile strain criteria at the bottom of the asphaltic concrete layer. Kentucky's proposed system (5) is based partially upon the tangential strain component. The tangential component is generally the largest in magnitude, but the radial component often is nearly as large. Only the tangential component has been utilized because laboratory test data yield only one component of tensile strain. The net effect of all components of strain can be correlated with any component of strain. Thus, design systems based upon one component of strain may be converted to a design system that utilizes the net effect of component strains. All comments concerning component strains also apply to component stresses.

The concept of total work (29) was applied in the development (5, 31) of the proposed Kentucky thickness design system. The total work was expressed as a function of limiting tensile strains for all values of Young's modulus. Unfortunately, at the time, no way was found to combine a single limiting strain criterion with a single limiting stress criterion. Since then, work with the concepts of strain energy and work strain has provided a key to successfully combining both concepts of limiting strains and stresses and total work. Applying this concept will modify the proposed Kentucky thickness design method for asphaltic concrete pavements and provide a theoretically based design method that will be far superior to any existing method. Funding should be authorized to accomplish this task.

Thickness design procedures based upon concepts of equal work and strain energy should apply equally to portland-cement concrete pavements. Development of design procedures for such pavements may require a high priority. The thickness design method proposed by the Portland Cement Association (32) has two major restrictions:

a. axle configurations are limited to four-tired single axles or eight-tired tandem axles, and
b. the maximum thickness of concrete pavements provided for is 10.5 inches. Kentucky has built 11-inch portland cement concrete pavements.

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