MEMO TO: J. R. Harbison 
State Highway Engineer 
Chairman, Research Committee


The report enclosed herewith issued from a meeting held in the Research Division, February 27, 1974, for the purpose of reviewing Report No. 376 (Design Guide for Bituminous Concrete Pavement Structures) with Design and Planning staffs concerned with pavement design procedures. Several questions emerged in regard to computations of EWL's and EAL's as defined by AASHO and as modified in the proposed criterion. There were also questions concerning relationships and comparisons between the proposed criterion and the 1972 AASHO Interim Guide. The several points, then and afterwards, seemed so inter-related that one aspect could not be disassociated from another. Our efforts to reconcile those issues proved to be so rewarding that we are submitting the findings as a research report.

Respectfully submitted,

J. C. Havens
Director of Research

cc's: Research Committee
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Adaptation of AASHO Interim Guide to Fundamental Concepts

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Considering an analysis of the combination of equivalent repetitions of different loads and variable load survivability values, many apparent discrepancies between results of designs obtained by the 1979 Kentucky design curve, the 1973 Kentucky design guide, and the AASHO nomographs were accounted for and explained. Several important conclusions were noted:

1. The theoretically based (elastic theory) 1973 Kentucky design criterion for an asphaltic concrete modulus of 3.31 GPa (480 ksi) requires thicknesses which would encompass 95 percent of the AASHO Road Test data points.
2. The same line is essentially parallel to the band of data points.
3. The assumed cutting criterion incorporated in the 1973 Kentucky design guide combined with the variable terminal survivability concept is validated by AASHO Road Test data.
4. The basic AASHO equation produced solutions that lie approximately in the middle of the AASHO Road Test data points. This implies a 50-percent probability of premature pavement failure.
5. The basic AASHO Road Test equation lies in the middle of the family of asphaltic concrete elastic moduli lines, and its position corresponds to a Young's modulus of elasticity of approximately 4.81 GPa (700 ksi).
6. The AASHO Road Test data confirm the validity of the 1973 Kentucky design criteria and, in turn, the 1975 Kentucky design criteria illustrate that elastic theory explains much of the behavior observed at the AASHO Road Test and allows the extension of the AASHO Road Test results with a considerable increase in confidence.
7. The independent analysis of the AASHO Road Test data indicates that reasonable levels of design can be reached with a 3.31 GPa (480 ksi) design criterion. The lower value of 2.59 GPa (375 ksi) would provide a very safe design system and would encompass approximately 98 percent of the AASHO Road Test data.

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ADAPTATION OF AASHO INTERIM
GUIDE TO FUNDAMENTAL CONCEPTS

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Bureau of Highways. This report does not constitute a standard, specification, or regulation.

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EXECUTIVE SUMMARY

Using 1971 vehicle classification counts and truck weights from nine Kentucky locations, equivalent axleloads (EAL's) were calculated by several methods. Apparent discrepancies led to a review of axleload equivalency factors used to estimate either EWL's (equivalent wheel loads) or EAL's.

Axleload equivalencies are determined as the ratio of the number of repetitions of a standard or reference load to the number of equivalent (damage-wise) repetitions of the load in question. The choice of equivalency factors can result in as much as a 40-percent difference in calculated EAL's. Most of Kentucky's traffic is made up of axleloads less than 80 kilonewtons (18 kips). The 1973 Kentucky design guide axleload factors are more severe than either the 1959 Kentucky or 1972 AASHO Interim Guide factors for axleloads less than 80 kilonewtons (18 kips). An extensive effort has been made herein to explain these differences. The AASHO Road Test has provided an independent source of data.

In 1970, the Kentucky laboratory CBR of the AASHO Road Test subgrade soil was reported to be 5.2, corresponding to the AASHO soil support value of 3.0. For this study, thickness requirements for a CBR of 5.2 were converted to structural numbers using appropriate AASHO coefficient values. Thus, comparisons of data and designs could be easily accomplished.

An investigation of the AASHO "design" nomographs and equations indicated values from the equations which produced a line that either passed through the midst of the data points or else had a rather severe rotation and departure from the data. This implies that any pavement thickness design based on that equation has approximately a 50-percent probability of premature fatigue failure. A system for pavement thickness designs should, however, be predicated upon a line that encompasses a higher percentage of the test data, even as much as 90 to 95 percent of the data points. This would assure a reasonably high probability of successful performance of the pavement during its design life.

Kentucky pavement designs have, for years, been predicated on the premise that high volume roads should be thick enough to minimize rutting so as to minimize the costs, inconvenience, and hazards associated with maintenance activities. On the other hand, low volume roads can tolerate considerable rutting and pavement cracking because geometries generally reduces the allowable speed limit to the point that rutting is not a dangerous factor. These notions of adequate pavement performance can be embodied in the AASHO Road Test concept of terminal serviceability.

Considering an analysis of the combination of equivalencies in repetitions of different axleloads and variable terminal serviceability values, many apparent discrepancies between results of designs obtained by the 1959 Kentucky design curves, the 1973 Kentucky design guides, and the AASHO nomographs were accounted for and explained. Several important conclusions were noted:
1. The theoretically based (elastic theory) 1973 Kentucky design criterion for an asphaltic concrete modulus of 3.31 GPa (480 ksi) requires thicknesses which would encompass 93 percent of the AASHO Road Test data points.

2. The same line is essentially parallel to the band of data points.

3. The assumed rutting criterion incorporated in the 1973 Kentucky design guide combined with the variable terminal serviceability concept is validated by AASHO Road Test data.

4. The basic AASHO equation produced solutions that lie approximately in the middle of the AASHO Road Test data points. This implies a 50-percent probability of premature pavement failure.

5. The basic AASHO Road Test equation lies in the midst of the family of asphaltic concrete elastic moduli lines, and its position corresponds to a Young's modulus of elasticity of approximately 4.83 GPa (700 ksi).

6. The AASHO Road test data confirms the validity of the 1973 Kentucky design criterion and, in turn, the 1973 Kentucky design criterion illustrates that elastic theory explains much of the behavior observed at the AASHO Road Test and allows the extension of the AASHO Road Test results with a considerable increase in confidence.

7. The independent analysis of the AASHO Road Test data indicates that reasonable levels of design can be reached with a 3.31-GPa (480-ksi) design criterion. The lower value of 2.59 GPa (375 ksi) would provide a very safe design system and would encompass approximately 98 percent of the AASHO Road Test data.
INTRODUCTION

Thicknesses of flexible pavement structures have been determined by several systems, among which are Kentucky's 1948 and 1959 curves (1, 2), the 1972 AASHO Interim Guide (3), and Kentucky's proposed 1973 design guide (4). The objectives of this paper are (1) to compare results obtained from the above methods and (2) to resolve apparent discrepancies of EWL-EAL calculations as made in each of the methods.

BACKGROUND

1959 Kentucky Method

The 1959 Kentucky design curves (Figure 1) were based upon empirical tests and observations of pavements in 1948 and 1957 and were generally drawn to separate points representing pavements which were performing satisfactorily from those representing unsatisfactory performance. The curves were an extension of Kentucky's 1948 curves to account for increased traffic volumes. The data indicated that the curves should have been positioned to provide slightly thicker pavements, but engineering judgement could not justify these increases at that time.

1973 Kentucky Design Guide

The 1973 Kentucky design guide is based upon elastic theory, and the curves (5, 6) were drawn to provide structural thicknesses that resulted in the same strain values (Figure 2). The limiting values of strain were obtained by matching theoretical strain values with field performance data. The matching was primarily based upon the performance of a 0.58-meter (23-inch) pavement thickness for a CBR of 7 under a field loading of 8 x 10^6 applications of 80-kilonewton (18-kip) axiloads.

AASHO Road Test

A stated objective of the AASHO Road Test (7) was to determine significant relationships between the numbers of repetitions of specified axleloads of different magnitudes and arrangements and the performance of different thicknesses of uniformly designed and constructed asphaltic concrete surfaces on different thicknesses of bases and subbases when on a basement soil of known characteristics. Special studies were also made of pavement fatigue. It should be noted that none of the objectives of the AASHO Road Test mentioned the development of a pavement thickness design system. The emphasis was upon pavement structural fatigue analysis and comparison of load-damage effects.

The following equation was derived (3) to express the relationship between repetitions (N), axleload (L), structural number (SN), and terminal serviceability (P):

\[
\log W_t = 5.93 - 9.36 \log (SN + 1) - 4.79 \log (L_x + L_2) - 4.33 \log L_2 + G/Bx,
\]
where \( G_t = \log \left\{ \frac{(4.2 \cdot P_t)}{(4.2 \cdot 1.5)} \right\} \), and
\[ P_X = 0.40 + \frac{[0.081(L_X + L_2)^{3.23}]/[(S + 1)^{5.19}(L_2)^{3.23}]}{[(S + 1)^{5.19}(L_2)^{3.23}]} . \]

For single axles, this equation reduces to
\[ \log W_t = 5.93 \cdot 9.36 \log (S + 1) - 4.79 \log (L_x + 1) + G_t/B_x . \]

Equations 1 and 2 were used to develop nomographs ("design charts") similar to that shown in Figure 3.

**FATIGUE CONCEPTS**

Fatigue testing of such materials as steel indicates that materials which will withstand a given number of repetitions at one stress level will support another number of repetitions if the stress level were changed. This concept has been applied by AASHO (3, 7) and others (1, 2, 4), and the ratios of the number of repetitions at one stress level to the number of repetitions at another stress level causing the same damage has been labeled as "traffic equivalency factors" -- also "axleload equivalencies" or "traffic damage factors".

Bradbury (8) performed a few laboratory fatigue tests on reinforced concrete samples in the 1930's. His results were used by Grumm (9) to develop axleload equivalencies used in the 1942 California pavement design system and the 22-kilonewton (5-kip) wheel load was chosen as the base value. These same factors \( f_k \) were used in the development of the 1948 and 1959 Kentucky design curves and can be represented by (5)
\[ f_k = a(2P')^a - 5 = a(\sqrt{2}P)^a - 10 , \]
where \( P' = \) axleload in tons or wheel load in kips,
\( P = \) axleload in kips, and
\( a = 1 . \)

Thus, a 22-kilonewton (5-kip) wheel load, or a 44-kilonewton (10-kip) axleload, was used as the base value by Grumm and for the 1948 and 1959 Kentucky design methods inasmuch as Bradbury’s test results were the best data available for determining such equivalency factors for the calculation of EWL's (equivalent wheel loads) until publication of the WASHO and AASHO Road Test data and analyses.

The AASHO EAL (equivalent axleloads) system was based upon one-directional traffic. The EWL system used by Kentucky was based upon two-directional traffic. An approximate conversion of the EWL system from a 22-kilonewton (5-kip) wheel load, two-directional traffic basis to 80-kilonewton (18-kip) EAL's for two-directional and one-directional traffic, respectively, is given by the following:
\[ \text{EAL (two-directional)} = \frac{\text{EWL (two-directional)}}{16} \] and
\[ \text{EAL (one-directional)} = \frac{\text{EWL (two-directional)}}{32} . \]
Flexible pavement sections at the AASHO Road Test provided for the first really sound field fatigue-testing program. Analysis of the AASHO Road Test data resulted in Equations 1 and 2. Solutions of these equations indicated the number of repetitions for given axleloads on a given pavement structure which would produce a selected level of terminal serviceability (damage). Figure 4 shows the AASHO Road Test relationship between axleload and repetitions for a pavement structure having a SN of 5.0 and a $P_1$ of 2.5. Superimposed is the 1959 Kentucky relationship defined by Equation 3. Note that 256 million 22-kilonewton (5-kip), two-directional EWL's converts to 8 million 80-kilonewton (18-kip), one-directional EAL's by Equation 5 and that the AASHO and 1959 Kentucky curves intersect at this value. This intersection was retained as one criterion point in developing the 1973 Kentucky repetitions - axleload relationship. The other criterion point was obtained by the line that was tangent to the AASHO curve at 8 million EAL's and 80-kilonewton (18-kip) axleload as expressed by

$$EAL = N_1(1.25)^{P_1} - 18$$

where EAL is in terms of 80-kilonewton (18-kip) axleloads and $N_1$ is the number of repetitions of axleloads $P_1$ (in kips).

Equation 6 can be used to determine an equivalent axleload which in one repetition produces catastrophic failure. This axleload is 397 kilonewtons (89.2 kips) and is the other criterion point for the axleload - repetitions line identified as 1973 Kentucky modified AASHO in Figure 4. Figure 4 emphasizes a basic difference in shapes of the single axleloads - log repetitions relationships for the three procedures. Inspection shows that, in the axleload range of 44 to 89 kilonewtons (10 to 20 kips), there is fairly good agreement between the 1959 Kentucky curve and the 1972 AASHO curve for a $P_1$ of 2.5 and SN of 5.0. In this same range, the 1972 AASHO curve could be reasonably approximated by a straight line. The 1973 Kentucky curve lies between the 1959 Kentucky curve and the 1972 AASHO curve for axleloads greater than 80 kilonewtons (18 kips). For axleloads less than 80 kilonewtons (18 kips), the 1973 Kentucky curve lies below both the 1959 Kentucky and 1972 AASHO curves, showing a more significant effect of light axleloads.

Traffic equivalency factors is another term used for damage equivalency factor and is defined by

$$\text{Traffic Equivalency Factor} = \frac{\text{number of repetitions of 80-kilonewton (18-kip) axleload}}{\text{number of repetitions of axleload in question}}$$

Equation 7 was used to transform the three relationships shown in Figure 4 into traffic equivalency factors - axleload relationships shown in Figure 5. Note that any axleload and its associated repetitions is "equivalent" (damage-wise) to the reference axleload at its base or reference number of repetitions. The equivalency factor varies significantly from one relationship to another. These equivalency factors are given in Table 1; differences between equations are apparent.

Most traffic on Kentucky pavements have axles equal to or less than 80 kilonewtons (18 kips).
Traffic equivalency factors for axles less than 80 kilonewtons (18 kips) are indicated as more severe under the 1973 Kentucky method than either the 1959 Kentucky or 1972 AASHO methods. The higher EAL determinations obtained by the 1973 Kentucky method match past experiences of Kentucky (1, 2, 4, 5, 6) and the AASHO Road Test (7) in that many pavements have reached performance limits earlier than expected – even when adjusted for increased traffic volumes.

ANALYSIS

AASHO Traffic Equivalency Factors

Table 2 summarizes the AASHO traffic equivalency factors as a function of SN (measure of pavement thickness) and axleload. Inspection of Table 2 raises the question: For a given axleload, why do traffic equivalency factors vary as a function of SN? Equation 2 was used to investigate the effect of and to verify Table 2 values. Ranges of variables of SN from 1.0 to 7.0, Pt from 1.5 to 3.5, and axleloads from 27 to 160 kilonewtons (6 to 36 kips) were investigated. Families of Pt curves for given values of SN's are shown in Figure 6, and the 1973 Kentucky axleload–repetitions relationship is superimposed for comparison. Inspection of Figure 6 indicates that:

1. more repetitions of a given axleload can be tolerated if the allowable damage is permitted to increase (Pt decreases),
2. the number of repetitions for a given terminal serviceability level is dependent upon the axleload,
3. a relationship between axleload and repetitions having a variable terminal serviceability could be developed,
4. levels of terminal serviceability tend to become insignificant at a combination very low numbers of repetitions and very high axleloads,
5. the families of terminal serviceability curves for given structural numbers tend to rotate from near vertical for low structural numbers to near 45 degrees for high structural numbers, indicating reduced sensitivity to load magnitude in stronger pavements,
6. variation in the spread of allowable repetitions between terminal serviceability levels of 1.5 to 3.5 increases correspondingly with increasing structural numbers, and
7. a parallel shift of the 1973 Kentucky relationship between repetitions and axleload appears to be almost tangent at 80 kilonewtons (18 kips) to the terminal serviceability level of 2.5 for each structural number except 1.0 and has essentially the same tangent slope there.

From a plot of SN as a function of axleload for 8 x 10^6 EAL's (Figure 7), it is seen that 80-kilonewton (18-kip) axleloads require a structural number of approximately 5.4 for 8,000,000 repetitions at a terminal serviceability level of 2.5. Iterative calculations determined a more precise value of SN to be 5.36, and the family of curves is shown in Figure 6.
Maximum Terminal Serviceability

Figure 8 shows that Equation 2 gives reasonable solutions for 80-kilonewton (18-kip) axleloads and \( P_t \) of 1.5 to 2.5, but then degenerates to a S-shaped curve for \( P_t \)'s greater than 3.75. This may in part be due to the choice of a maximum terminal serviceability of 4.2. Figure 9 indicates that a maximum terminal serviceability value of 4.5 to 5.0 might not be unreasonable (note convergence of extrapolated curves at \( P_t \) of this approximate value). Figure 10 is the same as Figure 8 except the \( G_t \) term in Equation 1 was redefined as

\[ G_t = \log[(5.0 - P_t)/(5.0 \cdot 1.5)]. \]

The same basic comments for Figure 8 are true for Figure 10 except that the reasonable range is extended to a \( P_t \) of 3.5, but degenerates to a S-shaped curve for a \( P_t \) greater than 4.0. Therefore, the validity of Equation 1 appears to be a function of \( P_t \).

AASHO Terminal Serviceability Ratings

Seasonally adjusted AASHO Road Test data (from Appendix A of Reference 7) was chosen to help explain some of the difficulties mentioned previously. The 80-kilonewton (18-kip) axleload data are presented for specific terminal serviceability levels in Figures 11 through 15. The short dashed lines are solutions of Equation 2 and were apparently intended to be the best-fit curves of the data points. Use of these lines as a criterion for thickness design (Figure 3) implies that approximately half of the pavements would be underdesigned and would fail prematurely. A higher probability of success in a pavement design criterion could be assured by a curve encompassing more of the observed data.

The AASHO Road Test was constructed on a soil assigned a soil support value of 3.0, corresponding to a Kentucky CBR \((12)\) of 5.2. This permits a direct comparison of AASHO structural numbers with the 1959 and 1973 Kentucky designs. The 1973 Kentucky design guide thicknesses at a CBR of 5.2 and an asphaltic concrete modulus of 3.31 GPa (480 ksi) were converted to equivalent SN's, using values of \( a_1, a_2, \) and \( a_3 \) of 0.44, 0.14, and 0.11, respectively, by the equation \((3)\)

\[ SN = a_1 d_1 + a_2 d_2 + a_3 d_3, \]

where \( d_1, d_2, \) and \( d_3 \) are thicknesses of various layers in the pavement structure. These SN's were plotted on Figures 11 through 15 and are shown as the solid line. Note that the 3.31-GPa (480-ksi) curve encompasses all of the data points in Figures 11 and 12, all but one and two data points in Figures 13 and 14, respectively, and passes through the middle of the data points in Figure 15.

Analysis of the other AASHO axleload data (from Appendix A of Reference 7) was attempted to extend the range of equivalent 80-kilonewton (18-kip) axleload repetitions. Plots of SN vs number of repetitions for \( P_t \) of 1.5, 2.0, 2.5, 3.0, and 3.5 were made (similar to Figures 11 through 15) for each of the 27-, 53-, 100-, and 133-kilonewton (6-, 12-, 22.4-, and 30-kip) axleloads. Using the 1973
Kentucky traffic equivalency factors in Table 1, graphs for different axleloads for fixed $P_t$ values could be equated by shifting the graphs to equivalent 80-kilonewton (18-kip) repetition values as displayed by Figures 16 through 20. The 1973 Kentucky 3.31-GPa (480-ksi) asphaltic concrete modulus curve (in terms of SN and repetitions) are superimposed on Figures 16 through 20; this curve encompasses a portion of the data in the following ways:

1. in Figure 16, the curve fits the 27-kilonewton (6-kip) axleload data best at $P_t = 1.5$, yet the remainder of the axleload data points fall further below this line as the axleload increases;
2. in Figures 17 and 18 ($P_t = 2.0$ and 2.5, respectively), the curve best fits the 53-kilonewton and 80-kilonewton (12-kip and 18-kip) data, respectively, diverges less from the increasingly higher axleload data, but comes closer to the 27-kilonewton (6-kip) data;
3. in Figure 19 ($P_t = 3.0$), the curve fits the 100-kilonewton (22.4-kip) data best, lies just above the 133-kilonewton (30-kip) data, but penetrates into the midst of the 27-, 53-, and 80-kilonewton (6-, 12- and 18-kip) data.
4. in Figure 20 ($P_t = 3.5$), the curve fits the 133-kilonewton (30-kip) data best, but significantly penetrates into the midst of the other axleload data.

The relationship of increasing axleload with increasing $P_t$, displayed in Figures 16 through 20, coincides with the concept of "rutting criteria" adopted in the 1973 Kentucky design criteria. The 1973 Kentucky thickness design curves provides for variable terminal serviceabilities which are dependent upon expected equivalent axleloads repetitions. Higher values of terminal serviceability are used for heavily traveled routes (because of the need for 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 speed limits and rutting is not such a prominent and dangerous factor.

The trend of increasing axleloads with increasing terminal serviceability levels noted in Figures 16 through 20 were combined to produce Figure 21. The following observations need to be emphasized:

1. the 3.31-GPa (480-ksi) asphaltic concrete modulus Kentucky curve encompasses approximately 93 percent of all AASHO data points (Kentucky moduli lines for 1.86, 2.10, 2.59, and 4.14 GPa (270, 305, 375, and 600 ksi) have also been superimposed);
2. the 80-kilonewton (18-kip) single-axle curve for $P_t = 2.5$ from Figure 22 (7) was superimposed onto Figure 21, showing that the curve encompasses 59 percent of the data points, has the same general shape suggested by the data points except for the 27-kilonewton (6-kip) 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 (4) 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 Road Test;
5. one level of terminal serviceability should not be used for the wide range of EAL's, as suggested by AASHO thickness design nomographs such as Figure 3; and
6. AASHO nomographs provide for only approximately a 60 percent probability of successful pavement performance throughout the design life.

Comparison of Three Design Methods

Vehicle classification and truck weight data for 1971 from nine locations were used to compute one-directional 80-kilonewton (18-kip) EAL's by the following methods:
1. Method 1 used the 1959 Kentucky traffic equivalency factors given in Table 1,
2. Method 2 used the AASHO axle-equivalency factors in Table 1 for \( P_t = 2.5 \) and \( SN = 5.0 \), and
3. Method 3 used the traffic equivalency factors based upon the 1973 Kentucky axleload-repetitions relationship as shown on Figure 4 and summarized in Table 1.

EAL computations are summarized in Table 3 and should be reviewed with the following facts in mind:
1. 1973 Kentucky pavement thickness graphs (4) and(or) tables are based upon equivalency factors (6) of Table 1, and these factors must be used in EAL computations.
2. Most traffic on Kentucky pavements have axles equal to or less than 80 kilonewtons (18 kips). Traffic equivalency factors for axles less than 80 kilonewtons (18 kips) are presumed to be more severe under Method 3 than either of Methods 1 or 2. Higher EAL values obtained by Method 3 match past experiences of Kentucky and the AASHO Road Test in that the service life of many pavements has been reached earlier than expected -- even when adjusted for increased traffic volumes.
3. The basic relationship of log repetitions of axleloads versus a single, equivalent axleload (arithmetic scale) matches past Kentucky experience (Figure 4) and the AASHO Road Test (Figure 21).
4. Variations in vehicle classifications and axleload distributions can cause significant changes in EAL determinations for the same pavement structure.
5. The 1959 Kentucky flexible pavement design curves (2) had ranges of EAL's for each "Traffic Curve," and the associated nominal values were the midpoints of these ranges.

The majority of the Kentucky and AASHO Road Test experience has been related to a three-layer pavement system, and pavement thickness designs would best be compared on this basis. Table 3 is a compilation of the thicknesses for a three-layered pavement system for the respective EAL's; Figure 23 is a graphical presentation of the data in Table 3. Several observations are worthy of note:
1. the 1973 Kentucky design guide will always produce thicker pavements than either of the other two systems;

2. in developing the 1973 Kentucky design guide, the addition of 5 centimeters (2 inches) of thickness, alluded to previously under BACKGROUND, was made at a CBR of 7; the increase in pavement thickness at a CBR of 5.2 is a little greater because of elastic theory requirements;

3. for 80-kilonewton (18-kip) axleload repetitions less than 8,000,000, the 1959 Kentucky curves required more thickness than the AASHO nomographic solutions; and

4. for 80-kilonewton (18-kip) axle load repetitions greater than 8,000,000, the AASHO nomographic solutions required more thickness than the 1959 Kentucky curves but less thickness than the 1973 Kentucky design guide.

COMMENTS ON THE 1972 AASHO INTERIM GUIDE

Use of the 1972 AASHO Interim Guide nomographs (Figure 3) will not provide a high probability of adequate pavement performance for the following reasons:

1. Errors were apparently made in drafting the AASHO thickness design nomographs. Unit intervals on the arithmetic soil support scale are not the same length.

2. The equation used to develop the AASHO nomographs, is a best fit line through the data. Equation 1 and Figure 21 indicate that approximately 40 percent of the actual designs have a probability of premature failure. For design purposes, the equation should have encompassed a higher percentage of the data points.

3. One level of terminal serviceability, as suggested by the AASHO nomographs should not be used over a wide range of EAL’s.

4. Additional nomographs for higher terminal serviceability values should have been provided so that the designer could have chosen a higher level of design for larger volume facilities.

5. Existing AASHO nomographs are probably being misused by many designers because of a combination of the following reasons:

   a) the designer must assume some structural number in order to select traffic equivalency factors for use in calculating EAL’s;

   b) using the "design" nomographs, the designer obtains a "Structural Number";

   c) since traffic equivalency factors vary according to structural number (Figure 6), EAL’s should be recalculated according to the new structural number; and

   d) this iterative procedure should be followed until the assumed and resulting structural
numbers are essentially the same.

A reanalysis of the AASHO data (Appendix A of Reference 7) in a manner similar to that used in this report in developing Figures 11 through 21 could result in a new AASHO "design" nomograph which could include scales for a variable terminal serviceability and variations in the modulus of elasticity of the asphaltic-bound layer and a repositioning of the soil support scale to coincide with the correlation shown in Figure 24 (3).

SUMMARY

1. Figures 21 and 23 and Tables 1 and 3 clearly show that (a) using the 1973 Kentucky modified AASHO traffic equivalencies (4, 6) has the net result of increasing EAL determinations and (b) increased thicknesses of the 1973 Kentucky designs over those obtained from the 1959 Kentucky curves (2) are justified, as the 1959 analyses indicated.

2. EAL calculations using the 1973 Kentucky modified AASHO traffic equivalency factors (4, 6) requires no more work, indeed probably less work, than that required by using the 1959 Kentucky EAL (2) or the 1972 AASHO Interim Guide traffic equivalency (3) methods.

3. Table 3 shows that approximately a 40-percent increase in calculated EAL values will result using the 1973 Kentucky traffic equivalencies (4, 6) given in Table 1 and shown in Figures 4 and 5.

4. It must be emphasized that use of the 1973 Kentucky design guide (4) is based upon the "1973 Kentucky Modified AASHO Traffic Equivalencies" (6) as shown in Figure 4 and Table 1. EAL's calculated by any other relationship MUST be converted to equivalent 1973 design guide EAL's. Failure to make the necessary conversion will result in misuse of the 1973 design guide tables and graphs.

5. EAL's calculated by any equation or relationship of repetitions versus equivalent axleloads can be readily converted to, and compared with, the 1973 Kentucky design guide EAL values.

6. The semi-logarithmic relationship between single axleload and repetitions used in the 1973 Kentucky design guide is not only a reasonable assumption but has been confirmed by the analysis of the AASHO Road Test data.

7. The 1973 Kentucky design guide is predicated upon empirical, Kentucky experience (4, 5, 6) and has been strengthened by a matching of the AASHO Road Test data (7).

8. This independent analysis of the AASHO Road Test data (7) merged with the 1973 Kentucky design guide (4), which is predicated upon elastic theory, indicates that the Road Test load - fatigue behavior is also explainable by elastic theory. Therefore, AASHO Road Test results can be confidently extended by use of elastic theory and the 1973 Kentucky design guide.

9. A reanalysis of the AASHO Road Test (7) data shows that Equation 1 (3) is based upon a
best fit line through the middle of the data. Use of Equation 1 to create the AASHO "design" nomographs (Figure 3) (3) results in pavement designs having a 40-percent probability of premature failure.

10. Had AASHO positioned a line that encompassed 90 to 95 percent of the Road Test data (4, 10) and used the 1973 Kentucky traffic equivalency factors (4), there would be negligible differences between the design methods.

11. The cursory analysis of the AASHO Road Test (7) data herein suggests that the AASHO flexible pavement design system (3) requires reanalysis and that the existing AASHO nomographs be modified or completely redesigned. A new nomograph could be designed with scales to adjust for desired variations in terminal serviceability values and to relate the AASHO system to elastic theory (by means of variations in modulus of the asphaltic-bound layers). The soil support scale should be modified to agree with Figure 24 (3).

REFERENCES


2. Drake, W. B. and Havens, J. H., Kentucky Flexible Pavement Design Studies, Bulletin No. 52, Engineering Experiment Station, University of Kentucky, 1959.


Figure 1. 1958-59 Kentucky Flexible Pavement Design Curves.
Figure 2. Theoretical Vertical Compressive Subgrade Strains, Asphalt Tensile Strains, and Surface Deflections as a Function of CBR and Total Thickness.
Figure 3.
AASHTO Design Chart for Flexible Pavements, P* = 2.5
(Reference 3, page 19).
Figure 4. Repetitions versus Single Axleload Equivalencies.
Figure 5. Traffic Equivalencies Related to Single Axleloads.
Figure 6. Relationship between Repetitions and Single Axle load for AASHTO SN and $P_t$ Variations.
Graphical Solution of Design Equation for $8 \times 10^6$ Repetitions of 80-kilonewton (18-kip) Single Axleload.

<table>
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<tr>
<th>$P_t$</th>
<th>Equivalent Axleload</th>
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(Graph) (Equation)
Figure 8. Solutions of Design Equation for Maximum Terminal Serviceability of 4.2.
Figure 9. Relationship of AASHO SN Values, Repetitions of 80-kilonewton Single Axleload, and Terminal Serviceability, $P_t$. 
Figure 10. Solutions of Design Equation for Maximum Terminal Serviceability of 5.0.
Figure 11. Relationship between Repetitions of a 80-kilonewton (18-kip) Single Axleload and Structural Number for Terminal Serviceability of 1.5, AASHO Road Test Seasonally Weighted Data (Reference 7, Appendix A).

Figure 12. Relationship between Repetitions of a 80-kilonewton (18-kip) Single Axleload and Structural Number for Terminal Serviceability of 2.0, AASHO Road Test Seasonally Weighted Data (Reference 7, Appendix A).
Figure 13. Relationship between Repetitions of a 80-kilonewton (18-kip) Single Axleload and Structural Number for Terminal Serviceability of 2.5, AASHO Road Test Seasonally Weighted Data (Reference 7, Appendix A).

Figure 14. Relationship between Repetitions of a 80-kilonewton (18-kip) Single Axleload and Structural Number for Terminal Serviceability of 3.0, AASHO Road Test Seasonally Weighted Data (Reference 7, Appendix A).
Figure 15. Relationship between Repetitions of a 80-kilonewton (18-kip) Single Axleload and Structural Number for Terminal Serviceability of 3.5, AASHO Road Test Seasonally Weighted Data (Reference 7, Appendix A).
Figure 16. Relationship between Repetitions of Single Axleloads and Equivalent Repetitions of 80-kilonewton (18-kip) Single Axleloads for Terminal Serviceability of 1.5.

Figure 17. Relationship between Repetitions of Single Axleloads and Equivalent Repetitions of 80-kilonewton (18-kip) Single Axleloads for Terminal Serviceability of 2.0.
Figure 18. Relationship between Repetitions of Single Axleloads and Equivalent Repetitions of 80-kilonewton (18-kip) Single Axleloads for Terminal Serviceability of 2.5.

Figure 19. Relationship between Repetitions of Single Axleloads and Equivalent Repetitions of 80-kilonewton (18-kip) Single Axleloads for Terminal Serviceability of 3.0.
Figure 20. Relationship between Repetitions of Single Axleloads and Equivalent Repetitions of 80-kilonewton (18-kip) Single Axleloads for Terminal Serviceability of 3.5.

Figure 21. Relationship between Repetitions of Single Axleloads of Various Terminal Serviceabilities and 80-kilonewton (18-kip) Single Axleloads for a Terminal Serviceability of 2.5.
Figure 22. Main Factorial Experiment, Relationship between Design and Axle Applications (Reference 7, Figure 22, page 27).
Figure 23. Total Pavement Thickness for 1958-59 Kentucky Design Curves, 1973 Kentucky Design Curves, and 1972 AASHO Interim Guide for $P_I = 2.5$, $SN = 5.0$, as a Function of Repetitions of 80-Kilonewton (18-kip) Single Axleload.
Figure 24. Relationship between Soil Support Value and Static CBR Value (Reference 3, page 71).
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TABLE 3

**PAVEMENT DESIGN THICKNESSES**
*(ONE-THIRD OF THICKNESS BEING ASPHALTIC CONCRETE)*

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<th>STATION NUMBER</th>
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