Rational Analysis of Kentucky Flexible Pavement Design Criterion

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MEMO TO: A. O. Neiser
State Highway Engineer
Chairman, Research Committee

DATE: November 19, 1968

SUBJECT: Research Report (Interim); "Rational Analysis of Kentucky Flexible Pavement Design Criterion;"
HPR-1(4), Part II: KYHPR-64-20

The structural design system for asphaltic concrete pavements presently used by the Department was adopted in 1958. At that time, the original system, adopted in 1948, was revised and updated. Twenty years of experience and research are behind us. In 1966, it seemed compelling to re-examine the design system in overview and in relation to interim concepts emergent from the AASHO Road Test and mechanistic theory. In 1966, as before, deflection measurements were made on approximately 75 pavements. It became apparent, after an engrossing investigation by Mr. Southgate to resolve a deflection-temperature adjustment factor, that deflection data are pervaded with variances and uncertainties beyond those assignable to temperature. Actually we were remiss in not coring the pavements and evaluating the in-place condition of the soil, but we had chosen to rely on CBR data from subgrade samples taken prior to paving -- presupposing that the soil would not be any less competent than the minimum laboratory CBR. Deflections were generally higher than this supposition could admit. We then deferred further empirical analysis of the deflection data and sought guidance through elastic theory. This avenue proved so rewarding in other ways that we further deferred the deflection data analysis to more fully exploit an apparent discovery. The report submitted herewith is far more encompassing than the original purview. The deflection data remain to be reconciled; and hopefully, a confirming report on that phase will follow.

The current findings are in remarkably good agreement with other so-called rational criteria which have evolved recently. There is unique agreement between Kentucky's current design charts and mechanistic principles. We are inclined to believe that elastic theory coupled with fatigue equations suffices as a first-order approximation. The findings are timely -- if not belated.
Mr. Southgate was accorded principal authorship on the report in recognition of exceptional work. The assistance of D. C. Newberry, Jr. in interpolating and plotting is respectively acknowledged.

Respectfully submitted

Jas. H. Havens
Director of Research

Attachment

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Research Report

RATIONAL ANALYSIS OF KENTUCKY
FLEXIBLE PAVEMENT DESIGN CRITERION

INTERIM REPORT
KYHPR-64-20; HPR-1(4), Part II

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U. S. Department of Transportation
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The opinions, findings, and conclusions in this report are not necessarily those of the Department of Highways or the Bureau of Public Roads

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INTRODUCTION

Rational criteria for the structural design of pavements are emerging from classical theories equated to the observed behavior of real pavements. Pavement behavior is known to be affected by traffic, variations in soil support, and variations of component thicknesses. Considerable attention has been devoted to the mechanistic response of pavements to static and dynamic loads and to the development of theoretical design procedures, which rely, in part, on the computation of certain critical stresses, strains, and (or) deflections in the structure. A computer program (1) for the elastic analysis of multilayered pavement systems has enabled an extensive investigation of the effects of soil support properties, the materials used in the pavement structure, and component thicknesses. In this study, this computer program was used to determine the patterns of stresses, strains, and deflections of the pavement system. In the second portion of the study, attempts have been made to show the relationships between these stresses, strains, and deflections and current and proposed design curves using the fatigue concept (equivalent axleloads - EAL's, or equivalent wheel loads - EWL's).

From the mechanistic point of view, load deflection relationships 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 the bituminous concrete layers may (do) constitute overriding, fundamental limits. Therefore, thickness design criteria cannot be based directly upon deflection spectra. In other words, two different pavements having equal, 18-kip deflections are not necessarily equal designs unless all accompanying stresses and strains are also equal.
COMPUTATIONS BASED ON THE ELASTIC THEORY

The Chevron Research Company furnished a "privileged" duplicate of their computer program for the elastic analysis of a n-layered pavement system to the Kentucky Department of Highways, Division of Research. This program is capable of handling the analysis of a 15-layered system and computes for any specified depth and distance from the axis of loading the following:

1. Stresses and strains
   A. Vertical
   B. Tangential
   C. Radial
   D. Shear
   E. Bulk

2. Deflections.

The following assumptions were made by Michelow (1) in developing the Chevron program:

1. The asphaltic pavement is assumed to be a semi-infinite solid of n layers.
2. The elastic characteristics can be different from one layer to another.
3. The elastic properties are assumed to be homogeneous and isotropic and are characterized by Young's modulus and Poisson's ratio.
4. The elastic coefficients in each layer can assume any value.
5. A uniformly distributed load on a circular area is placed on and normal to the free surface of the pavement.
6. The interfaces between layers are considered to be rough.
7. The bottom layer is assumed to be a semi-infinite solid.

Input Data

Lettier and Metcalf (2) reported that the modulus of elasticity of a subgrade could be estimated by the product of 1500 psi and CBR. In the absence of more reliable information, this relationship was used in this study to determine the modulus for the subgrade. A review of the literature (3, 4) indicates that a reasonable minimum value for the modulus of elasticity for dense-graded aggregate of good quality is approximately 25,000 psi. This value was used throughout this analysis regardless of subgrade support values.
Dorman and Edwards (5) reported that Poisson's ratio varied from 0.35 to 0.45, and in this analysis, values of 0.40 for the asphaltic concrete and dense-graded aggregate layers and 0.45 for the soil subgrade were used. An 18,000-pound axleload and a tire pressure of 80 psi were taken to represent the loading.

Table 1 shows the pavement variables that were analyzed by the computer program. The different thicknesses of the component layers and subgrade support (CBR) were analyzed as a matrix in order to be able to compare one structure with another. Five average pavement temperatures, namely, 40°, 60°, 80°, 100° and 120°F, were used to determine the modulii of elasticity of the asphaltic concrete according to Southgate (6). The relationship between modulus and average pavement temperature is pictured graphically in Figure 1. The values for the modulus of elasticity for the five pavement temperatures investigated were 1,800,000 psi, 600,000 psi, 270,000 psi, 145,000 psi and 100,000 psi, respectively.

Table 1. Variables Investigated.

<table>
<thead>
<tr>
<th>Variables</th>
<th>Thickness in Inches</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dense-Graded Aggregate</td>
</tr>
<tr>
<td>Subgrade CBR</td>
<td>2</td>
</tr>
<tr>
<td>2.2</td>
<td>2</td>
</tr>
<tr>
<td>3.7</td>
<td>8</td>
</tr>
<tr>
<td>5.1</td>
<td>14</td>
</tr>
<tr>
<td>8.8</td>
<td>20</td>
</tr>
<tr>
<td>14.7</td>
<td>27</td>
</tr>
<tr>
<td>29.3</td>
<td>20</td>
</tr>
<tr>
<td>58.7</td>
<td>20</td>
</tr>
</tbody>
</table>

The Chevron n-layered computer program requires as input data the following:

1. Identification,
2. Number of layers,
3. Modulus of elasticity and Poisson's ratio for each layer,
4. Thickness (in inches) of each layer above the subgrade (subgrade is assumed to be infinitely thick),
5. The number of radii and their location (in inches) from the load, and
6. The number and location of the depths (in inches) at which the computed stresses, strains, and deflections are desired.
In Figure 2 are examples of the graphs contained in Appendix A. These plots show the effect of the variation of asphaltic concrete and dense-graded aggregate thicknesses upon (1) vertical compressive subgrade strains directly beneath the load, (2) pavement surface deflections directly beneath the load, and (3) tangential strains at the bottom of the asphaltic concrete layers and directly beneath the load. Each graph represents solutions for a specific CBR, temperature (or modulus of asphaltic concrete), and pavement structure. Data were read from this matrix of graphs and transposed to base plots -- on which the axes are total pavement thickness (ordinate) and log CBR (abscissa); this resulted in lines of equal deflections, equal subgrade strains and equal asphaltic concrete strains. A base graph was prepared for various temperatures (40°, 60°, 80°, 100°, and 120°F) and for given ratios of asphaltic concrete thickness to thickness of dense-graded aggregate (1:2, 1:1, and 1:0) (Figure 3).

![Figure 1. Mean Pavement Temperature vs Modulus of Elasticity for Asphaltic Concrete Pavement (6, p 36).](image)

Figure 1. Mean Pavement Temperature vs Modulus of Elasticity for Asphaltic Concrete Pavement (6, p 36).
Figure 2. Vertical Compressive Subgrade Strain, Pavement Surface Deflection, and Tensile Strain at the Bottom of the Asphalitic Concrete Layer in Relationship to Thickness of Asphalitic Concrete and Thicknesses of Granular Base.
Figure 3. Base Graph Showing Theoretical Deflections, Vertical Strains at the Top of the Subgrade ($\epsilon_s$) and Asphaltic Concrete Strains ($\epsilon_a$) for 60°F; One-third of Thickness is Asphaltic Concrete.
THEORETICAL RECONSTITUTION
OF
KENTUCKY DESIGN CURVES

The essential elements of a set of design criteria involving predictive theory are (1) equations of static (or dynamic) equilibrium and (2) equations of failure. Elastic theory (represented here by a computer program capable of solving multilayered systems) is presumed to suffice as a first-order approximation. Equations of failure are necessarily empirical or phenomenological; they bring into issue all manner of experience, performance histories, and discrete test data -- e.g. fatigue data. Failure equations are represented here by Kentucky's current design chart and other interpretative analyses of limiting strains or fatigue limits. Statements of equilibrium were equated to statements concerning failure and then graphically displayed. Hereinafter, all references to "base graphs" allude to elastic theory computations; all references to "EWL's and EWL curves" allude to failure. The two are equated by superpositioning.

Due weighting of the 1958-59 design curves (7) from the standpoint of deflection data would have positioned Traffic Curve X lower on the chart and thus required slightly greater thicknesses throughout. However, the final family of curves was tempered judiciously midway between the thicknesses required by earlier curves and the 1958-59 control points. Intuitively, it seemed that the curves should collapse toward the 100-CBR value and asymptotically approach infinite thickness toward the extreme low CBR's. It seemed also that doubling the EWL through each successive curve should require successively diminishing incremental increases in thickness. A deflection of 0.017 inch (9-kip wheel) was associated then with Curve X at a CBR of 7.1 and a 23-inch pavement thickness.

The present attempt to reconstitute the Kentucky curves theoretically began with the assumption that a mutual or coincident control point exists at a CBR of 7, total thickness of 21 inches, and a deflection value of 0.015 inch. This point was judiciously transposed to CBR 6 and 23 inches thickness by following the theoretical deflection curve to the new point. This shift is shown in Figure 4. The new point also has a deflection value of 0.015 inch and, there, has a value of $2.5 \times 10^{-4}$ for vertical strain ($\varepsilon_8$) at the top of the subgrade. This value of $\varepsilon_8$ was conservatively selected to be approximately 50 percent of Dorman and Edwards' critical value ($5.3 \times 10^{-4}$ for $10^6$ repetitions of the loading) (5).

The remaining traffic curves were interpolated from this control point on the basis of a resolved proportional relationship between various pavement structures and their respective 18-kip deflections and a reduced, equivalent, single axleload (or wheel load) corresponding to a given summation of EWL's.

Curve X is assigned a precise value of $256 \times 10^6$ EWL's. By custom, this has included two-directional traffic; thus, one-directional traffic for Curve X is $128 \times 10^6$ equivalent 5-kip wheel loads (or 10-kip axleloads) in one
Figure 4. Flexible Pavement Design Curves (revised 1959) Superimposed onto Theoretical Curves for 60°F; One-third of Thickness Composed of Asphaltic Concrete -- Shows Adjustment of "Control Point."
direction. The load equivalency (damage) factors used in the Kentucky method are given by

\[ f_K = a(2)^{P' - 5} = a(\sqrt{2})^{P - 10} \]  

(1)

where \( P' \) = axleload in tons or wheel load in kips,
\( P \) = axleload in kips, and
\( a = 1 \).

Then: \( P \) = 10 12 14 16 18 etc.
\( f_K \) = 1 2 4 8 16

Likewise, the AASHO load-damage factors can be described approximately by

\[ f_A = a(1.25)^{P_A - 18} \]  

(2)

where \( P_A \) = axleload in kips and
\( a = 1 \).

Thus: \( P_A \) = 10 12 14 16 18 20 etc.
\( f_A \) = .17 .26 .41 .64 1 1.56

Kentucky's computation of EWL's from mixed traffic data may be described as follows:

\[ \text{EWL}_i = N_i f_{Ki} = N_i (\sqrt{2})^{P_i - 10} \]

\[ \text{EWL(Total)} = \sum \text{EWL}_i = \sum N_i (\sqrt{2})^{P_i - 10} \]

All i's
All i's

where \( N_i \) = number of repetitions of load \( P_i \) and
\( P_i \) = axleload in kips.

Any EWL(Total) may, hypothetically, be transformed to an equivalent number of repetitions of other base loads by the equation

\[ \text{EWL(Total)} = N_i (\sqrt{2})^{P_i - 10}. \]  

(3)

Letting \( N_i = 1 \), \( P_i \) becomes an equivalent, single load which would, hypothetically speaking, be as damaging as the number of EWL's. Likewise, the AASHO 18-kip EAL's may be represented similarly as

\[ \text{EAL(Total)} = N_i' (1.25)^{P_{Al} - 18} \]  

(4)

where \( N_i' \) = number of repetitions of load \( P_{Al} \) and
\( P_{Al} \) = axleload in kips.

The principal difference between Equations 3 and 4 lie in the constant ratios. Furthermore, the assumption of a constant ratio throughout the full range of AASHO load-equivalency factors is an operative license -- which is real and valid only in the region of 18-kip loads -- but which permits com-
putations of a hypothetical, equivalent, single load, as before. It may be mentioned that the real values of AASHO load-equivalency factors become irrational in the range of extremely high axleloads. For instance, an incremental increase in load beyond 40 kips is proportionately less damaging than the same increment added to 30 kips. The above equation respects only the constant ratio of a geometric progression which is clearly evident within a limited range about the 18-kip load level.

A Kentucky EWL of $128 \times 10^6$ transforms to $8 \times 10^6$ 18-kip equivalent axleloadings; that is

$$N_1(\sqrt{2})^{P_1-10} = 128 \times 10^6\text{EWL's} = N_{18}\text{f}_{A18}$$

and

$$f_{A18} = (\sqrt{2})^{P_{18}-10} = 16.$$  

Thus $N = 8 \times 10^6$. Therefore $128 \times 10^6$ 10-kip axles is equivalent to $8 \times 10^6$ 18-kip axles, or

$$\text{Kentucky EWL's/16} = \text{AASHO EAL's.} \quad (5)$$

The equivalent, single axeloads corresponding to the above-equated EWL's and EAL's are shown comparatively below.

<table>
<thead>
<tr>
<th>Kentucky EWL</th>
<th>$P(N=1)$</th>
<th>AASHO EAL</th>
<th>$P_A(N'=1)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$128 \times 10^6$</td>
<td>63.9</td>
<td>$8 \times 10^6$</td>
<td>89.2</td>
</tr>
<tr>
<td>$64 \times 10^6$</td>
<td>61.9</td>
<td>$4 \times 10^6$</td>
<td>86.1</td>
</tr>
<tr>
<td>$32 \times 10^6$</td>
<td>59.9</td>
<td>$2 \times 10^6$</td>
<td>83.0</td>
</tr>
<tr>
<td>$16 \times 10^6$</td>
<td>57.9</td>
<td>$1 \times 10^6$</td>
<td>79.9</td>
</tr>
<tr>
<td>$8 \times 10^6$</td>
<td>55.9</td>
<td>$5 \times 10^5$</td>
<td>76.8</td>
</tr>
<tr>
<td>$1 \times 10^5$</td>
<td>43.2</td>
<td>$1 \times 10^5$</td>
<td>69.6</td>
</tr>
<tr>
<td>$1 \times 10^1$</td>
<td>16.6</td>
<td>$1 \times 10^1$</td>
<td>28.3</td>
</tr>
<tr>
<td>$1 \times 10^0$</td>
<td>10.0</td>
<td>$1 \times 10^0$</td>
<td>18.0</td>
</tr>
</tbody>
</table>

It is also apparent that for each value of EWL or EAL, $P$ and $P_A$ may be determined for other values of $N$ and $N'$. These inter-relationships are shown graphically in Figures 5 and 6.

**Limiting Subgrade Strains Versus EWL's**

Figure 6 was used in conjunction with Figure 7, which was derived independently from elastic theory computations and which relates the ratio of vertical strain in the subgrade ($\varepsilon_s$) at various impressed wheel loads to the strain at a basic wheel load of 9 kips ($\varepsilon_{s9}$) throughout an array of spectrum of pavement structures, to establish the plotting locus of each of the respective EWL-curves on the base graphs (Appendix B). From Figure 7,

$$\varepsilon_sN/\varepsilon_{s9} = M_4 \frac{P_1}{2} + K_4 \quad (6)$$
Figure 5. Relationships between Equivalent Single Axleloads and Accumulative EWL's and ASSHO EAL's.

Figure 6. Relationships between Equivalent, Hypothetical Axleloads and Number of Repetitions.
Figure 7. Ratio of Subgrade Strain to Strain under 9-Kip Wheel Load as a Function of Equivalent, Hypothetical Wheel Load.
where $M_i$ = tangent slope at point $i$,
$K_i$ = corresponding intercept,
$P_{1/2}$ = wheel load (in kips), and
$\varepsilon_{SN}/\varepsilon_{S9}$ = ratio of subgrade strain under load $P_{1/2}$
to corresponding subgrade strain under
a 9-kip wheel load.

From Figure 6, $P_1 = -6.62 \log N_1 + 63.7$ or

\[ P_{1/2} = -3.31 \log N_1 + 31.85 \quad (7) \]

Combining Equations 6 and 7,

\[ \varepsilon_{SN}/\varepsilon_{S9} = (-3.31 \log N_1 + 31.85)M_1 + K_1 \quad (8) \]

It was established from separate work (8, 9, 10) that an $\varepsilon_S$ of $2.5 \times 10^{-4}$
for Traffic Curve X ($8 \times 10^6$ 18-kip axles) would provide a high degree of as-
surance against rutting; this value was also assigned to $\varepsilon_{S9}$, so that at
$8 \times 10^6$ repetitions and $P/2 = 9$, $\varepsilon_{SN}/\varepsilon_{S9} = 1.00$. This provided the principal
control for positioning Curve X on all base graphs. Curve IX, for example,
was naturally based on $4 \times 10^6$ repetitions:

\[ P/2 = -3.31 \log (4 \times 10^6) + 31.85 \]

From Figure 7, a 10-kip wheel results in a ratio for $\varepsilon_{SN}/\varepsilon_{S9}$ of 1.105. Since
$\varepsilon_{S9} = 2.5 \times 10^{-4}$, $\varepsilon_{SN} = 2.725 \times 10^{-4}$. Respective solutions for all traffic
curves (IA, I, ... XII) are provided below.

<table>
<thead>
<tr>
<th>Traffic Curve</th>
<th>N(18-kips)</th>
<th>P/2</th>
<th>$\varepsilon_{SN}/\varepsilon_{S9}$</th>
<th>$\varepsilon_{SN}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>XII</td>
<td>$3.2 \times 10^7$</td>
<td>7</td>
<td>0.780</td>
<td>$1.950 \times 10^{-4}$</td>
</tr>
<tr>
<td>XI</td>
<td>$1.6 \times 10^7$</td>
<td>8</td>
<td>0.890</td>
<td>$2.225 \times 10^{-4}$</td>
</tr>
<tr>
<td>X</td>
<td>$8 \times 10^6$</td>
<td>9</td>
<td>1.000</td>
<td>$2.500 \times 10^{-4}$</td>
</tr>
<tr>
<td>IX</td>
<td>$4 \times 10^6$</td>
<td>10</td>
<td>1.105</td>
<td>$2.763 \times 10^{-4}$</td>
</tr>
<tr>
<td>VIII</td>
<td>$2 \times 10^6$</td>
<td>11</td>
<td>1.215</td>
<td>$3.038 \times 10^{-4}$</td>
</tr>
<tr>
<td>VII</td>
<td>$1 \times 10^6$</td>
<td>12</td>
<td>1.320</td>
<td>$3.300 \times 10^{-4}$</td>
</tr>
<tr>
<td>VI</td>
<td>$5 \times 10^5$</td>
<td>13</td>
<td>1.425</td>
<td>$3.563 \times 10^{-4}$</td>
</tr>
<tr>
<td>V</td>
<td>$2.5 \times 10^5$</td>
<td>14</td>
<td>1.530</td>
<td>$3.825 \times 10^{-4}$</td>
</tr>
<tr>
<td>IV</td>
<td>$1.25 \times 10^5$</td>
<td>15</td>
<td>1.626</td>
<td>$4.065 \times 10^{-4}$</td>
</tr>
<tr>
<td>III</td>
<td>$6.25 \times 10^4$</td>
<td>16</td>
<td>1.730</td>
<td>$4.325 \times 10^{-4}$</td>
</tr>
<tr>
<td>II</td>
<td>$3.12 \times 10^4$</td>
<td>17</td>
<td>1.825</td>
<td>$4.563 \times 10^{-4}$</td>
</tr>
<tr>
<td>I</td>
<td>$1.56 \times 10^4$</td>
<td>18</td>
<td>1.922</td>
<td>$4.805 \times 10^{-4}$</td>
</tr>
<tr>
<td>IA</td>
<td>$7.81 \times 10^3$</td>
<td>19</td>
<td>2.020</td>
<td>$5.050 \times 10^{-4}$</td>
</tr>
</tbody>
</table>

This type of stylized weighting of limiting strains in terms of equiva-
| lent repetitions implies equal assurance against rutting throughout the full
| spectrum of traffic. In one sense, it implies that a pavement designed by
| Curve IV, for example, would not rut to any greater extent than a pavement
Figure 8. Limiting Asphallic Concrete Strains vs Number of Repetitions of an 18-Kip Axleload.
designed by Curve X. Obviously, this is an extremely conservative approach. It was not possible, here, to assign rut-depths to the curves. Conceptually at least, rut-depths allowed for pavements in the class of Curve IV should be somewhat greater than those permitted in pavements in the Curve X class. No additional weighting scheme has been devised, but a judgement-type of weighting may be exercised by interpolating designs between limiting subgrade strains and limiting curves in the asphaltic concrete for the respective curves.

Curves III through IA have been omitted from some of the graphs because it is obviously impractical from an economic point of view to design pavements of those classes to minimize rutting -- in other words, rutting ceases to be a valid criterion there, and the sole consideration of criterion is prevention of fatigue and breakup. This is done by faithfully limiting asphalt strains only.

**Limiting Asphalt Strains Versus EWL's**

The criterion concerning limiting strains in the asphaltic concrete was based on interpretative analyses of other work (cf. 10). Van Der Poel (11, 12) indicated that a safe strain limit of asphalt was in the order of $1 \times 10^{-3}$ at $3^\circ F$. Since asphaltic concrete consists of approximately 10 percent asphalt by volume, this fixes the strain limit of asphaltic concrete at $30^\circ F$ in the order of magnitude of $1 \times 10^{-4}$. Others (2, 5, 9, 10) have established (by interpretative analyses of pavements and fatigue-test data) that the magnitude of asphalt strain ($\varepsilon_A$) assuring $1 \times 10^6$ repetitions at $50^\circ F$ is $1.45 \times 10^{-4}$. Extrapolated and interpolated values of strain corresponding to Kentucky EWL curves are shown in Figure 8 (all at $50^\circ F$). Since the base graphs (Appendix B) correspond to moduli of asphaltic concrete at $40^\circ$, $60^\circ$, $80^\circ$, $100^\circ$, and $120^\circ F$, it was necessary to relate the $50^\circ F$ strains to other temperatures. Although the computer program calculates asphalt strain -- as is shown on the base graphs -- the temperature-strain-repetitions relationship sought had to be pure and independent of CBR and thickness. Thus, asphalt strains were determined from a separate grid-array of twenty-five pavement structures, seven CBR values and three temperatures. Strains were plotted against temperature -- holding CBR constant and identifying each of the resulting curves by their structure. Limiting strains at $50^\circ F$ corresponding to certain repetitions taken from Figure 8 were superimposed on these graphs (one graph for each CBR). These respective repetitions were translated linearly to $40^\circ$, $60^\circ$, $80^\circ$, $100^\circ$, and $120^\circ F$ along the respective structure curves. The strains corresponding to ordered numbers of repetitions and temperatures were tabulated for each CBR and were then averaged. The resulting strains ($\varepsilon_A$) are given in Table 2.

Only one example of these graphical manipulations is shown here (Figure 9). The intermediate graphs (Figure 2) were found to be superfluous for this purpose inasmuch as the type of graph shown proved to be sufficient in itself when strains were taken directly from computed data. The value shown in the table determined the plotting locus of limiting strains ($\varepsilon_A$) curves associated with respective EWL curves (IA, I, ..., XII) superimposed on the basic graphs.

Presumably, this criterion of limiting asphalt strains provides equal assurance against cracking and breakup; it does not allow progressively increasing risk of failure toward the lesser classes of roads. In the event that weighting in that way is desired, those pavements could be designed for proportionately shorter life.
Figure 9. Asphaltic Concrete Strains as a Function of Temperature.
Table 2. Limiting Asphaltic Concrete Strains

<table>
<thead>
<tr>
<th>Traffic Curve</th>
<th>Temperature (°F)</th>
<th>50</th>
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Design Temperature

The judicious selection of a "control point," as described at the very outset, also weighted or biased the selection of a design temperature somewhat towards 60°F. If this is, in fact, the most appropriate temperature and is most expressive of year-round conditions from the standpoint of pavement design, then an independent argument must be provided. The coincidence between Kentucky's current design curves and the 60°F base curves would be annulled to some extent if an independent analysis of temperature data indicated that a temperature much higher than 60°F is more realistic. In that event, the present Kentucky curves would, by inference, be invalidated to some extent.

A rational notion suggests an analysis of temperature data from the standpoint of percentage of time during the year that the pavement is below a certain temperature. It would be impractical, of course, to design for the maximum. However, if designs are based on temperatures less than maximum, rutting to some degree may occur as the natural consequence of summer temperatures. A compensating effect emerges, since the foundation soil is thought to become more competent at that time due to drying. Also, the most critical season, from the standpoint of pavement breakup, has proven to be early spring.

Two sources of temperature data are helpful and persuasive in this respect. One is Kallas' (13) record taken at College Park, Maryland; the other is Straub's et al (14) record taken at Potsdam, New York. Each source provides approximately a year of record. A summary analysis of the College Park data shows the average pavement temperature to be approximately 64°F. The time that the temperature is below the average is slightly greater than 50 percent. Although significance is implied here by association, no direct relationship can be cited. Adoption of 60°F as a design temperature would be consistent with the implied correlation between the current design curves and the theoretical curves derived on the basis of 60°F. Resulting designs would assure performance at least equal to that expected from current designs (employing
usual proportions of dense-graded aggregate base and asphaltic concrete). Full-depth asphaltic concrete designs may rut to a greater extent than customary designs. A possible recourse in that event would be to adopt a design temperature closer to the summertime average -- say 85°F. According to the College Park data, five warm months (May, June, July, August, and September) have averages ranging between 80 and 90°F. Weighting these excess temperatures according to percentage of time of occurrence \[60^\circ F + \left(\frac{5}{12}\right) \times (85^\circ F - 60^\circ F) = 70^\circ F\] suggests that 70°F would be appropriate. To effect this type of weighting, designs would have to be interpolated between the 60°F charts and the 80°F charts -- no 70°F design chart is provided herein. The average temperatures for the five warm months mentioned for all depths are:

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<th>Month</th>
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<td>August</td>
<td>87°F</td>
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<tr>
<td>September</td>
<td>81°F</td>
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</table>

Straub's records taken at Potsdam, New York, indicate slightly lower temperatures in comparison to the College Park records. Inasmuch as College Park is at approximately the same latitude as Lexington, Kentucky, Straub's records were not considered herein.
DISCUSSION

Perhaps one of the significant findings of this study is that the 1958 design curves very nearly parallel iso-subgrade strain lines. Only in the region where extrapolation was made do the 1959 design curves not parallel the iso-subgrade strain lines. Also, the 1959 design curves do not parallel iso-deflections lines but cross them in a prominent manner. Historically it has been generally considered that limiting deflections would prevent structural failure. Recently, investigators (9, 10, 15, 16) have suggested "...the possibility of basing flexible pavement design on the limitation of critical stresses and strains within the pavement section. Excessive deformation of the pavement is prevented by limiting the vertical compressive strain in the subgrade, and fatigue cracking of the asphalt layer is prevented by limiting the tensile strain at the base of the asphalt layer" (10). The 1959 Kentucky design curves lend support to the idea of basing pavement designs upon limiting stresses and strains in the pavement structure. The above statement is particularly true for the range of field test data from which the 1959 curves were developed. Therefore, the elastic theory and mathematical approach used in the Chevron n-layered computer program appears to very nearly fit the empirical data and resultant curves that preceded this study. Thus elastic theory and performance experience are mutually enhanced and are hereby reconciled and correlated in this analysis.

Several interpretative observations are thought to be pertinent; they are offered here for guidance.

1. The principal criterion employed in reconstituting the design chart was "limiting strain" at the bottom of the asphaltic concrete layer; this criterion provides assurance against "fatigue cracking" and premature breakup of the pavement. All design curves drawn provide equal assurance against failure.

2. The attendant criterion concerns "rutting". Rutting in pavements is objectionable from the standpoint of steerage, riding quality, and hydroplaning (skid resistance). The magnitude of rutting that would be admissible or tolerable in designs remains conjectural. It would seem impractical and uneconomical to design pavements for low classes of roads so that they would not rut anymore under their design traffic than the highest class pavement would rut under its respective traffic. Indeed, the rutting criterion is violable in some proportion to the level of service expected from the road. It may be desirable to dismiss rutting altogether in designing pavements for classes IA, I, ..., V and to design solely on the basis of assurance against fatigue cracking and premature breakup.

3. The respective design curves are specific only insofar as numbers of equivalent loadings are concerned. Design life is implied by the number of years of traffic considered. For instance, designing for a pavement life of less than 20 years has the same effect as moderating the design criteria. Thus, lower category roads may be designed for
10 years in the lower extreme without violating the design charts. Here again, it seems impractical and unrealistic to design a Curve I class pavement for a 20-year life. Perhaps a more realistic approach would be to design those roads for less than a 10-year life and extend the life thereafter as needed by employing so-called "stage construction" concepts. "Staged" designs may be employed throughout all traffic classes by judicious selection of the number of years apportioned to each stage. However, it may be noted that the difference between a 10-year, first-stage design represented by Curve IX and a 20-year design represented by Curve X is approximately one inch of thickness. Here, too, the so-called "rutting" criterion could be violated in designing the first stage -- so that a greater portion of the asphaltic concrete could be reserved for the second stage. Of course, the rutting criterion should be respected fully in the design for the second stage.

4. The term "layer equivalence" is inappropriate and inconsistent with concepts of design; its meaning is restricted to a perfunctory expression of the ratio of the layer thickness required at specific values of design parameters (CBR and EWL's) on one design chart as compared to another.

5. Kentucky designs have customarily approximated 1:2 apportionments of asphaltic concrete and dense-graded aggregate, respectively, to the total structural thickness required for specific designs. Although this was the basis on which 1:2 base graphs were drawn, the apportionment of asphaltic concrete was customarily increased toward the lower order traffic curves; in some instances, the asphaltic concrete may have exceeded one-half the total thickness. For this reason, it was not possible to reconstitute all of the design curves faithfully. Even transposing curves from the 1:1 charts would not suffice.

6. A serious breach in logic may be recognized in the assumptions made at the beginning of the theoretical analyses. Whereas the modulus of asphaltic concrete was varied in a predisposed manner according to temperature and whereas the modulus of the foundation soil was varied implicitly by $E = 1500 \times CBR$, the modulus for crushed limestone base (DGA) was assumed to be 25,000 psi and was presumed to remain constant. Though the assumed value seemed reasonable, the soil modulus was allowed to progress continuously to 150,000 psi. Either the soil modulus progression should have been discontinued at CBR 16.6 or else the modulus for the granular base should have been magnified by the same progression afforded the soil modulus. Had the base course been credited with a modulus of 150,000 at the outset, this discrepancy would not have arisen -- even though dense-graded crushed limestone yields CBR's greater than 100 in actual tests. Nevertheless, and in spite of these incongruous notions, deflections and strains in the region of low CBR's aligned in orderly magnitude and direction about the control point. Thus, only the region of high CBR's remains obscure or suspect.

7. Edge loading was not considered in this analysis.

8. Stabilized soil foundations and base courses should be considered in future designs as an alternative means of minimizing rutting.
9. Deflections remain a useful measure of the structural integrity of existing pavements. The base graphs and design curves derived herein should provide insights into suspected anomalous behavior and general competency of pavements -- that is, through deflection measurements.

Appendix C contains base graphs on which results from an extensive series of Benkelman-beam deflection measurements taken in the spring and summer of 1966 are arrayed. Wide variances among the data are evident. Generally, the deflections are excessive in comparison to the theoretical values for the respective structures. All measurements were adjusted beforehand to a base temperature of 60°F. The CBR's, as plotted, are based on actual subgrade soil tests made prior to construction of the pavement (nearest 500-foot station). No "in place" CBR or other test was made to define the condition of the soil at the time the deflection was measured. Inasmuch as the Kentucky CBR test allows the soil to reach a fully-soaked-and-swelled condition, it is presumed to represent the soil at its worst possible state in service under the pavement. On this basis, the deflections should never become excessive unless the pavement has suffered some degree of damage -- under traffic or otherwise. Of course, fatigue cracking may have occurred, and this should be in some relationship to traffic history. Fatigue damage may be viewed as being equivalent to a reduction in effective thickness or merely as a reduction in modulus. There is some reason to suspect, at least in some instances, that the asphaltic concrete layer has been "unseated" due to hydrostatic pressures. Excessive deflections naturally magnify the strains and shorten the life of the pavement. Further analysis or rationalization of the deflection data will be undertaken in a future phase of this study.
REFERENCES


APPENDIX A

Relationship between Thicknesses of Asphaltic Concrete and Dense-Graded Aggregate and Asphaltic Tensile Strain, Vertical Compressive Subgrade Strain, and Pavement Surface Deflection for Various CBR's and Average Pavement Temperatures.
CBR 5.13
40°F

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

VERTICAL SUBGRADE STRAIN (10^-4)

DEFLECTION (10^-3 INCHES)

RADIAL STRAIN OF ASPHALTIC CONCRETE (10^-4)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)
CBR 14.67

40° F

Radial Strain of Asphalitic Concrete (10^-4)

Thickness of Asphalitic Concrete (Inches)

Vertical Subgrade Strain (10^-4)

Deflection (10^-3 Inches)

Thickness of Asphalitic Concrete (Inches)
CBR 3.67

60° F

VERTICAL SUBGRADE STRAIN (10^-4)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

DEFORMATION (10^-3 INCHES)

RADIAL STRAIN OF ASPHALTIC CONCRETE (10^-4)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

2" DGA
8" DGA
14" DGA
20" DGA
27" DGA
CBR 8.80

60° F

Thicknss of Asphaltic Concrete (Inches)

Deflection (10^-3 Inches)

Radial Strain (10^-4)

Vertical Subgrade Strain (10^-4)

2" DGA
8" DGA
14" DGA
20" DGA
27" DGA
CBR 10

80°F

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

VERTICAL SUBGRADE STRAIN (10⁻⁴)

DEFORMATION (10⁻³ INCHES)

RADIAL STRAIN OF ASPHALTIC CONCRETE (10⁻¹)

22 DGA
6" DGA
14" DGA
20" DGA
27" DGA
CBR M.67
(20°F)

VERTICAL SUBGRADE STRAIN (in.1)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

RADIAL STRAIN OF ASPHALTIC CONCRETE (in.1)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)
CBR 29.33

120° F

VERTICAL SUBGRADE STRAIN (10^-4)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

RADIAL STRAIN OF ASPHALTIC CONCRETE (10^-4)

THICKNESS OF ASPHALTIC CONCRETE (INCHES)

27" DGA
20" DGA
14" DGA
8" DGA
2" DGA

27" DGA
20" DGA
14" DGA
8" DGA
2" DGA
APPENDIX B

Relationship between Total Pavement Thickness, Asphaltic Tensile Strain, Subgrade Vertical Compressive Strain, Pavement Surface Deflection and CBR for Pavements Composed of 1/3 Asphaltic Concrete and 2/3 Dense-Graded Aggregate; 1/2 Asphaltic Concrete and 1/2 Dense-Graded Aggregate; and Full-Depth Asphaltic Concrete.
Traffic-CBR-Thickness Curves
Full Depth AC
40°F
TRAFFIC-CBR-THICKNESS CURVES
I-I AC TO DGA THICKNESS
60 °F
TRAFFIC-CBR-THICKNESS CURVES

1/2 AC TO DGA THICKNESS
80° F
Subgrade Strain

Deflection Inches

40.0 x 10^-3
35.0 x 10^-3
30.0 x 10^-3
25.0 x 10^-3
20.0 x 10^-3
15.0 x 10^-3

Subgrade Strain

10.0 x 10^-4
8.0 x 10^-4
6.0 x 10^-4
4.0 x 10^-4
3.0 x 10^-4
2.5 x 10^-4
1.5 x 10^-4

Asphalt Strain

12.5 x 10^-6
10.0 x 10^-6
8.0 x 10^-6
6.0 x 10^-6
4.0 x 10^-6
2.5 x 10^-6

Traffic - CBR - Thickness Curves

Full Depth AC

80 °F
TRAFFIC-CBR-THICKNESS CURVES
I-I AC TO DGA THICKNESS
100°F
CBR-THICKNESS CURVES
AC TO DGA THICKNESS
120° F
APPENDIX C

Comparison of 1966 Benkelman Beam Deflection Data with Revised Design Curves
# PROJECT DESCRIPTION

## 1966 FLEXIBLE PAVEMENT DEFORMATION STUDY

### TEST SITE DESCRIPTION

#### PROJECT TERMINI DESCRIPTION

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<tr>
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<td>2.0 MILES E OF JCT US 641, 1.1 MILES W OF CALDWELL COUNTY LINE</td>
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<td>6-23-66</td>
<td>CALDWELL</td>
<td>DF-546(6)</td>
<td>17-0182- D</td>
<td>66106</td>
<td>66101</td>
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<td>US 62 FROM LYON-CALDWELL COUNTY LINE 3.65 MILES EAST</td>
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<td>2.0 MILES E OF LYON-CALDWELL COUNTY LINE</td>
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<tr>
<td>7-05-66</td>
<td>JEFFERSON</td>
<td>F-18(3)</td>
<td>56-0388- A</td>
<td>66106</td>
<td>66101</td>
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<tr>
<td></td>
<td>US 31-E BUCHEL BY-PASS</td>
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<tr>
<td></td>
<td>0.3 MILES S OF NORTH END OF BY-PASS</td>
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7-12-66JEFFERSON  F-18(3)(4)  58-0116-A  061150
US 31E FROM FERNDALE ROAD TO BUECHEL BY-PASS 4-LANE DIVIDED
1.8 MILES N OF FERNDALE ROAD 2.7 MILES S OF BUECHEL BY-PASS
6-15-66BOYLE  DF-294(4)  11-0020- K  061160
US 150 AND US 127 FROM BOYLE-MERCO COUNTY LINE 2.02 MILES SOUTH
+0 MILES SOUTH OF MERCO COUNTY LINE South Bound Lanes Only
6-26-66FAYETTE  F-124(11)  34-0434- A  061170
KY 418 FROM US 29 JUNCTION TO I 75 INTERCHANGE
+2 MILES WEST OF I 75 INTERCHANGE
6-10-66BOYLE  F-264(5)  11-0120- U  061180
US 150 +1.72 MILES N OF LINCO-LIN Boyle Co. Line TO 4.33 MILES S OF Co. Line
+9 MILES S OF BOYLE-LINCO COUNTY LINE
7-13-66BELL  F-151(13)(14)  7-0964- P  061190
US 119 FROM 5.47 MILE N OF JCT. US 29E FOR DISTANCE OF 6.93 MILES
5.4 MILES FROM JCT. US 29E 6-119 MILES FROM JCT. US 29E
6-08-66ADAMIS  F-167(5)  76-0471- E  061200
US 227 FROM 175 TO KENTUCKY RIVER
+7 MILES NORTH OF I 75 INTERCHANGE
7-12-66MASON  F-234(14)  81-0175- P  061210
US 68 +2.38 MILES N OF FLEMING-MASON COUNTY LINE TO 9.71 MILES N OF LINE
+4.2 MILES N OF FLEMING COUNTY LINE
6-22-66GRAVES  F-144(10)  42-0398- E  061220
US 45 MAYFIELD BY-PASS
+4 MILES N OF KY 121 INTERCHANGE 0.5 MILES S OF JUNCTION VS 49
+8 MILES S OF KY 49 INTERCHANGE
6-16-66BOYD  U-537(5)  10-3085- Q  061230
US 235 CITY LIMITS OF CATLETTSBURG TO JUNCTION WITH 34TH STREET
+2 MILES N OF PROJECT MARKET TO 44 MILE S OF 34TH ST. NO LANES ONLY
7-08-66LAWRENCE  FAP-78(6)  14-0003- K  061240
US 231 FROM JOHNSON-LAWRENCE COUNTY LINE TO A POINT 10.97 MILES NORTH
9.6 MILES N OF JOHNSON COUNTY LINE
2.6 MILES N OF JOHNSON COUNTY LINE IN FRONT OF ULYSSES POST OFFICE
6-10-66BOYLE  3-04(V)  11-0064- I  061250
KY 34 FROM EAST CITY LIMITS OF DANVILLE TO CHENAULT BRIDGE
+1.5 MILES WEST OF EAST END OF PROJECT NEAR CHENAULT BRIDGE
7-21-66MORGAN  EKE 4-1 4-2 4-3  68-0629-EKA  061270
7-21-66WOLFE  EKE 4-1 4-2 4-3  119-0603-EKH  061270
MOUNTAIN PARKWAY FROM KY 1010 INTERCHANGE TO MORGAN-MAGOFFIN COUNTY LINE
MILE POST 54
MILE POST 56
6-30-66HARDIN  WK 28-2  47-0909=KG  061280
WESTERN KENTUCKY PARKWAY ROCK CUT APPROXIMATELY 10 MILES W OF ELIZABETH TO 661280
MILE POST 18 MILES IN ROCK CUT +0.1 MILE E OF QUARRY 661281
6-27-66GRAYSON
SP 43-265-2C1 43-0265-PB 661430
ROUGH RIVER DAM STATE PARK, MAIN ENTRANCE ROAD AND LODGE PARKING AREA
MAIN PARK ENTRANCE ROAD, 200 YD TOWARD LODGE FROM KY 79
6-28-66GRAYSON
S 75N12 43-0415-0 661440
LEITCHFIELD-LILAC KY 737: JCT: KY 259 IN LEITCHFIELD TO LILAC
1.1 MILES N OF JCT: KY 259
4.4 MILES N OF JCT: KY 259
6-27-66GRAYSON
S 163(4) 43-0315-0 661450
KY 79 FROM JCT: KY 1702 AND KY 110 APPROX. 1.9 MI S OF DAM TO KY 54
1.9 MILES S OF JCT: KY 110
4.6 MILES S OF JCT: KY 110: 1.4 MILES N OF JCT: KY 54
6-30-66HARDIN
RSC 47-269-2C1 47-0269-A 661460
RINEYVILLE SCHOOL: JCT: KY 220 IN RINEYVILLE EXTENDING 0.305 MILES SOUTH
300 FT S OF KY 220 ON APPROACH TO RINEYVILLE SCHOOL
6-28-66HARDIN
S 360(1) 47-0199-M 661470
8 MILES S E OF WHITE MILLS TO 1.2 MILES N W OF WHITE MILLS: KY 84
1.1 MILES E OF NOLIN RIVER BRIDGE: 50 FT E OF CEMETARY
6-30-66HARDIN
RSC 47-929-1C2 47-0929-D 661480
RED HILL ROAD FROM KY 144 AT NOLIN VINE GROVE TO HINTON ROAD AT MEADE CO: LINEN
1.0 MILES N OF JCT: KY 144 IN VINE GROVE (NOLIN)
7-01-66HENRY
RSC 56-493-1C1 56-0493-A 661490
GOOSE CREEK ROAD: FROM KY 22 TO WESTPORT ROAD (KY 1447)
8.3 MILES S OF BROWNSBORO RD (KY 22) 8.1 MILES N OF WESTPORT ROAD
7-06-66JEFFERSON
RSC 56-533-1L 56-0533-L 661500
LONG RUN ROAD: FROM US 60 NEAR BRIDGE OVER LONG RUN TO SHELBY COUNTY LINE
1.5 MILES S OF US 60
6-21-66SIMPSON
SP 107-845-361 107-0845-D 661520
INDUSTRIAL ACCESS ROAD TO PUTTER-BRUMFIELD MACHINE FOUNDRY CO:
3.2 MILES FROM KY 171 TOWARD KY 100
7-15-66BREATHITT
F 102(8) 13-0367-D 661530
KY 15 FROM CAMPBELL TO JACKSON
3.4 MILES N OF BRIDGE AND 1.65 MILES S OF JCT: KY 1812
6-29-66LARUE
I 85-3(10)(70) 62-0661-C 661540
65 UPTON INTERCHANGE, STARTING AT EXIT RAMP AND ENDING AT ENTRANCE RAMP
150 FT N OF N EDGE OF BRIDGE OVER I-65: 50 FT N OF N END OF GUARD RAIL
8-01-66MADISON
I 75-3(4)87 76-0288-A 661550
75 FROM BARNES MILL RD 1.0 MI SW OF SWCL RICHMOND TO US 23 INTERCHANGE
MILE POST 88.5: 1.0 MILE N OF BARNES MILL ROAD INTERCHANGE
6-27-66GRAYSON
SP 43-260-2C2 43-0260-PB 661560
ROUGH RIVER DAM STATE PARK, FROM JCT: MAIN ENTRANCE RD TO BOAT DOCK HAMPS
PARK ROAD BETWEEN LODGE AND BOAT DOCK AT WATER TREATMENT PLANT
661561
KEY TO SAMPLE IDENTIFICATION

Project Code Number
Site Number
T = Test CBR Value
D = Design CBR Value
Average Deflection (Inches)
Adjusted to 60°F

Note: Refer to PROJECT DESCRIPTION for descriptive information.

TEST CONDITIONS

Measurements made by Benkelman Beam

18-kip Axleload
80 psi Tire Pressure
Dual Tires

Creep Speed Deflections - Point of beam positioned three feet ahead of rear truck axle. Truck moved forward at 1 to 1-1/2 miles per hour. Rebound measurements were obtained after truck was removed from test area.

Static Deflections - Point of beam positioned between dual wheels of rear truck axle. Truck moved forward to points at specified distances. Rebound measurements were made after truck had been in position long enough for all rebound to have occurred.
DEFLECTION INCHES

15.0 x 10^-3
2.0 x 10^-3
2.5 x 10^-3
3.0 x 10^-3
7.0 x 10^-4
6.0 x 10^-4
5.0 x 10^-4
4.0 x 10^-4
3.0 x 10^-4
2.0 x 10^-4

SUBGRADE STRAIN

7.0 x 10^-4
6.0 x 10^-4
5.0 x 10^-4
4.0 x 10^-4
3.0 x 10^-4
2.0 x 10^-4

TOTAL DEPTH, INCHES

0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

DEFLECTION

D = DESIGN CBR VALUE
T = TEST CBR VALUE

STATIC DEFLECTION

I = AC TO DGA THICKNESS

60 °F
DEFLECTION INCHES

10.0 x 10^{-3}
15.0 x 10^{-3}
20.0 x 10^{-3}
25.0 x 10^{-3}
30.0 x 10^{-3}

ASPHALT STRAIN
4.0 x 10^{-4}

DESIGN CBR VALUE

TEST CBR VALUE
2.5 x 10^{-4}

SUBGRADE STRAIN

TOTAL DEPTH, INCHES

CREEP DEFLECTION
1:2 AC TO DGA THICKNESS
60 °F
DEFLECTION INCHES

SUBGRADE STRAIN

TOTAL DEPTH, INCHES

CREEP DEFLECTION

1 IN AC TO DGA THICKNESS

60 °F
APPENDIX D

Discussion

of

Full-Depth Asphalitic Concrete
Both the WASHO and AASHO test roads, and other case studies, disclosed the merits of thick asphalt layers. The Asphalt Institute translated these findings into the "Deep Strength" concept. Others persisted in the notion that the structural integrity of a pavement system was inherently embodied in the unbound granular bases — thereby, relegating the function of the bituminous surfacing to that of preserving or protecting the granular base. Kentucky's design criterion and practices for interstate and primary roads allocate nominally one-third of the thickness to bituminous concrete. On lower echelon roads, higher proportions of asphaltic concrete are employed. This has been practiced somewhat intuitively and judiciously. Departures therefrom have employed significantly greater proportions and even full-depth designs — in no instance has bituminous concrete been placed directly upon subgrade soil. Nominally 1-1/2 inches to 2 inches of crushed stone have been spread beforehand.

Full-depth asphalt paving, as it is now termed, is really only a revival of a former concept that has been practiced to some extent since the beginning. An interesting account of 'black base' paving — including full-depth sections — is recorded in the Proceedings of the Fifth Asphalt Paving Conference, published by the Asphalt Association (now the Asphalt Institute) in 1926. The specific reference is to a paper entitled: "Black Base (Asphaltic or Bituminous Concrete Base) and Its Place in Standard Specifications," by Hugh W. Skidmore, President of Chicago Paving Laboratory. A companion article entitled "Black Base as a Time Saver in Mercer County, New Jersey," by Harry F. Harris, is found in the same issue. The 1927 Proceedings contains two reports: one by Warren H. Booker entitled "Black Base in the Southern States" and one by W. P. Cottingham entitled "Black Base at Gary, Indiana". These articles attest the favorable performance of designs ranging between four and six inches in thickness.

Apparently this type of paving was abandoned in the early- or mid-1930's — probably because of evident needs for greater structural thicknesses. The apparent recourse was water-bound macadam bases rather than increased thicknesses of asphaltic concrete.

Full-Depth Designs in Relation to Subsurface Drainage

Current notions concerning full-depth bituminous paving are somewhat in conflict with concepts and practices which have prevailed for some time in the past. For instance, so-called trench-type construction was customary during an earlier epoch, and this was followed by prolific use of French drains for relief of captive water in pavement substructures. Granular base courses were then extended full width across the embankment and even daylighted on the slopes to provide continuous lateral drainage. Even so, the concept that granular base courses comprise a "condensation chamber" is not disputed. Whereas granular bases extending to great depths may act as a "dry well" and otherwise limit the free water rise beneath the pavement, other arrangements which minimize capacity for water would therefore minimize condensation and infiltration. Moreover, if
capacity were limited to such an extent that the capillary suction of surrounding soil would imbibe any free water occurring in the system, a more favorable moisture balance would be assured. In times of deluge, the entire system would likely be quite saturated; but, since capacity is minimal, the recovery time may be lessened. Hence, the mechanism of "wicking" in soils is an adjunctive concept to that of full-depth paving. So-called impervious soils cannot possibly dispose of significant quantities of water by mere percolation mechanisms but may do so through strong capillary attraction.

Feasibility of Full-Depth Construction

Whereas the feasibility of constructing "thick lifts" of bituminous concrete has been demonstrated and is currently practiced in some areas of the country, two divergent points of view have emerged: one is the employment of "thick lifts" in situations where a firm foundation is not realized -- such as "quicking" soils; and the other is solely an economic or convenience consideration. Undoubtedly, the first-mentioned case demands a greater thickness of bituminous concrete to compensate for the deficient foundation and is, therefore, an expedient measure only. It is still quite logical to build up from a firm foundation. However, there remains an abiding reluctance to forsake granular base courses and to rely upon subgrade soils to provide the "working platform" and compaction reactance for the construction of high-type bituminous concrete base courses. Whereas soil subgrades are susceptible to dusting and "muddying" and would have to be maintained at or prepared to the proper template ahead of paving, granular bases are much less sensitive to disturbing influences and would be expected to withstand construction equipment and material deliveries more competently than soil. Hence, latent problems attendant to construction are matters for concern.