Evaluation of 1985 AASHTO Flexible Pavement Design Equations and Nomographs

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January 1987
The 1985 AASHTO nomograph and equations for flexible pavements were reviewed and analyzed to determine requirements for input data, sensitivity of designs based on the additional variables for percent reliability, standard deviation, and resilient modulus of subgrade material. Design thicknesses using the 1985 AASHTO Guide were compared to thicknesses using the 1981 Kentucky thickness design method for flexible pavements.
INTRODUCTION

The flexible pavement thickness design procedure in the 1985 AASHTO Guide (1) involves such factors as

1. loss in serviceability,
2. standard deviation,
3. reliability, and
4. resilient modulus of subgrade.

Questions to be answered to implement the 1985 Guide include, but may not be limited to, the following:

1. What is the effect of any individual variable, or combination of variables, upon the design Structural Number?

2. What values should be used by the Kentucky Transportation Cabinet, or others, for the above variables to produce designs compatible with Kentucky conditions and experience?

3. What is the relationship between resilient modulus of the subgrade and the elastic modulus of the subgrade?

4. What are the relationships between resilient modulus of the subgrade, Soil Support Value, R value, CBR, and other expressions of subgrade support?

5. What is the sensitivity of the Structural Number to each of these variables?

6. What values should be used for the "a" coefficients in the equation defining the Structural Number as a function of layer thicknesses?

7. How do Kentucky engineers, or others, obtain values for these variables?

8. What data need to be collected to determine appropriate relationships for the above variables?

AASHTO DESIGN GUIDES

1972 INTERIM NOMOGRAPHS

The 1972 Guide (2) contains nomographs that may be used to obtain pavement thickness designs based on the desired level of serviceability. The nomographs supposedly are solutions of the following equation:

\[ \log W = 9.36 \log (SN + 1) - 0.20 \frac{G_t}{B} + 0.372 (81 - 3.0), \]  

where \( W \) = number of 18-kip EALs,

\[ G_t = \frac{4.2 - P}{4.2 - 1.5}, \]
The basic form of Equation (1) was developed at the AASHO Road Test (3) and the constants were given at the St. Louis Conference (4). During the course of research in 1984 (5), it was determined that solutions of Equation 1 superimposed over the AASHO Road Test data (3) did not coincide with the mean fit of the Road Test data. However, that analysis incorporated the use of load equivalency factors based on Kentucky research and not on factors reported in the 1972 Guide (2).

Figures 1 and 2 illustrate the data reported in Tables C.2-3 and C.2-4 of Reference 2. Appendix A of Reference 3 presents observed fatigue data for each pavement thickness design section for five levels of serviceability. Layer thicknesses were converted to SN using

\[ SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \]  

where \( a_1 = 0.44 \), \( a_2 = 0.14 \), \( a_3 = 0.11 \), and \( D_1, D_2, D_3 \) = thicknesses of layers 1, 2, and 3, respectively.

At the AASHO Road Test, each lane of each loop was assigned a specific axle arrangement and axleloading (6). Thus, for a specific loop, the load equivalency for each section's SN was determined from the appropriate curves of Figures 1 or 2 (single or tandem axles, respectively). The load equivalency factor then was multiplied by the observed number of load repetitions to obtain the resulting fatigue data according to the respective level of serviceability and the results are shown in Figures 3-7. Design EALs for respective SNs were obtained from Equation 1 and have been superimposed over the fatigue data in Figures 3-7. Inspection of Figures 3-7 shows that solutions from Equation 1 do not describe a mean fit of the fatigue data as has been reported (1-4). Close inspection indicates that for high EAL values, the corresponding SN from the equation may be as much as 1.5 units greater than would be given by the line corresponding to a mean fit of the data. Yet for low EALs, the reverse is true to a much lesser differential value. Thus, the nomographs in the 1972 Guide (2) may be subject to question since their validity is based on an equation that does not truly fit the data from which it was apparently derived.
1985 DESIGN GUIDE -- Flexible Pavements

The flexible pavement design nomograph (1) is based on the solution of

\[
\log W = 2R \times SD + 9.36 \times \log (SN + 1) - 0.20 + -\frac{G'}{B} + 2.32 \times \log (MR) - 8.07, \tag{3}
\]

where

\[
G' = \frac{A \text{ PSI}}{4.2 - 1.5}
\]

2R = a statistical constant corresponding to a selected percentage reliability,
SD = overall standard deviation, and
MR = resilient modulus of the subgrade.

COMPARISON OF 1972 AND 1985 GUIDES

A comparison of Equations 1 and 3 shows that they are equivalent in format. A statistical term has been added in Equation 3 to allow the designer to select a confidence limit. The Soil Support Value has been replaced with a Resilient Modulus.

Equation 1 apparently was intended to be a mean fit to the AASHO Road Test data. A value of 1.0 for R (typical regional factor for Kentucky) causes that term to have a value of 0.0. A Soil Support Value of 3.0 also will cause that term to have a value of 0.0. In Equation 3, the term 2R \times SD is zero for a 50-percent confidence limit (a mean fit). In the term G', a change in PSI of 1.7 (4.2 - 2.5) is assumed to make a comparison between the two equations. The assumed relationship between Soil Support Value and Resilient Modulus is taken from Appendix FF of Reference 7:

\[
SI = 6.24 \times \log (MR) - 18.72. \tag{4}
\]

Substituting a value of 3.0 for SI into Equation 4 yields a value of 3,025 for MR. A comparison of the results from Equations 1 and 3 is given in Table 1 and indicates that the two equations essentially are the same for a mean fit and Equation 4.

COMPARISON OF 1985 GUIDE TO KENTUCKY SYSTEM

Implementation of the 1985 AASHTO Design Guide requires the use of

- resilient modulus,
- standard deviation,
- reliability, and
- change in serviceability.
Values, and/or relationships, must be chosen before the system may be used. The definitions, relationships, and values of these inputs to represent Kentucky conditions must be determined.

RESILIENT MODULUS

Heukelom and Klomp (9) reported that "dynamic modulus" (MD) could be approximated by

\[ \text{MD} = 1500 \times \text{CBR} \]

where CBR = California Bearing Ratio.

The 1981 Kentucky flexible pavement design system (9) makes use of Equation 5 and adequately represented empirical experience and test data used to model the 1959 Kentucky flexible pavement thickness design curves (10) with elastic theory. Equation 5 was used by Van Til et al. (11) to determine the moduli of elasticity to represent a wide range of subgrades. Deflection test data obtained by the Road Rater have been duplicated using Equation 5 to assign elastic moduli for the subgrade layer for analyses using elastic theory. Appendix FF of Reference 7 of the 1985 Guide virtually quoted Van Til, except the word "resilient" was used to modify "modulus of elasticity".

The 1985 Guide (1) specifies that the AASHTO T274 test procedure be followed for determining a modulus of the soil. The test method requires that a specified number of cycles of axial loading be applied at a specified confining pressure before using the measured deflection and recoverable deformation. Then the specimen is subjected to another confining pressure and the same number of cycles applied, after which measurements are recorded. This process is continued until a maximum confining pressure is reached and the process is reversed until the minimum confining pressure is reached. This test method uses only the recoverable strain and ignores any permanent strain to determine the modulus. In reality, the observed pavement behavior is a function of both permanent strain and recoverable strain. An extensive testing program is needed to determine any correlation between resilient modulus, modulus of elasticity, and CBR; or a method of test that more consistently matches observed pavement behavior is required.

CBR VERSUS SOIL SUPPORT

Hopkins and Deen (12) reported results of laboratory tests on subgrade samples taken from the AASHO Road Test. CBR tests were performed by both the Kentucky method and the ASTM method. As an average, a Kentucky CBR of 5.3 corresponded to a soil support value of 3.0. Figure C.3-4 of Reference 2 indicates that a CBR of 100 corresponded to a soil support value of approximately 8.25. Hopkins and Deen (12) estimated the accumulated EAL on a number of Kentucky pavements for which CBR test data were available. A plot of soil support versus log CBR of that data indicated a straight line regression would nearly be a mean fit through soil support values of 3.0 and 8.25, corresponding respectively with CBR values of 5.3 and 100:
RESILIENT MODULUS VERSUS SOIL SUPPORT VALUE

Appendix FF of Reference 7 gives the relationship between resilient modulus and soil support value as shown in Equation 4. Solving Equation 4 for resilient modulus yields

\[ MR = 10 \times (0.160256 \times SI + 3.0) \]

and corresponds to a resilient modulus of 21,000 psi at a Soil Support Value of 8.25, which is different from the 150,000 psi based upon Equation 5 and the correlations given in the 1972 Guide (2).

STANDARD DEVIATION

Appendix EE of the 1985 Guide (7) recommends a value of 0.49 for the standard deviation. This value incorporates variances of a number of variables such as layer thicknesses, their structural coefficients, estimates of future traffic, load equivalencies, etc. The Guide states that the value of 0.49 may be reduced if the highway agency has detailed data for the traffic using existing pavements. However, the value of the standard deviation must be determined for local conditions. Appendix EE of Reference 7 gives the details of the derivation. The rationale for suggesting the collection of traffic data with weigh-in-motion systems is that a larger volume of axleload and vehicle classification data would be provided.

The value of 0.49 has been used for the standard deviation in the analyses reported herein.

RELIABILITY

Appendix CC of Reference 7 states, "Increasing the reliability of a pavement design (or conversely, decreasing the probability for failure) normally results in increased construction costs. This increase in initial construction cost may be more than offset by a decrease in maintenance and repair costs over the design life, as well as a decrease in 'extra user costs' associated with early pavement failure. The impact of increased roughness or slipperiness and lane closure for rehabilitation can be extremely large for highway facilities which carry very high levels of traffic. If no reasonable detours exist, increased user costs associated with closing down traffic lanes can be so large that they overwhelm the entire life-cycle cost analysis. Thus, it is likely to be cost-effective to design pavements subject to large volumes of traffic at high reliability levels." Kentucky research (5) indicated that the 1981 Kentucky thickness design curves incorporated approximately 90 to 95 percent of the AASHO Road Test data. Using thickness designs from the 1981 Kentucky method and converting them to equivalent SN values using Equation 2 matched

\[ SI = 4.277 \times \log(CBR) - 0.344 \]

This relationship was used with Equation 5 to correlate the 1959 Kentucky thickness design curves that were based on field tests and empirical experience with elastic theory.
the SN obtained using the 1985 AASHTO method when values of 1.7, 0.49, and 92 percent are assumed for loss of serviceability, standard deviation, and reliability, respectively. Thus, a reliability factor of 92 percent appears to be reasonable for Kentucky and has been used in these analyses.

INTERRELATIONSHIPS OF INPUT VARIABLES

MATERIAL PROPERTIES

Coefficients a1, a2, and a3 used in Equation 2 were used in the sensitivity analyses. Equations 4-6 were used to relate CBR or Soil Support Value to resilient modulus. A value of 0.49 was chosen for the standard deviation. A terminal serviceability, PT, of 2.5 was chosen to relate results using the 1985 Guide to the 1972 Guide. To make a direct comparison to the 1972 and 1985 Guides, the value for reliability used in the 1985 Guide must be set to 50 percent, which sets the ZR term to 0.00 in Equation 3. With this combination of input values, the two nomographs produce essentially identical results as shown in Table 1. However, for the same SN and EAL, Equation 3 contains three variables, any one of which has a wide range of values dependent upon the other two variables. Table 2 contains calculated EALs using Equation 1 for given combinations of soil support value and structural number. Figures 8-10 show the interrelationships between standard deviation, percent reliability, and resilient modulus for soil support values of 2.24, 3.54, and 4.29, respectively. Using Figure 8 as an example, the interrelationships between standard deviation, percent reliability, and resilient modulus are not altered by the combinations of structural number and EAL given in Table 2 for the respective soil support value. Close inspection will show that the "fan" of curves is the same in Figures 8-10 except for the minimum resilient moduli values that are correlated to the soil support values from Equation 7. Thus, the user must be aware of the choice of values for the standard deviation and percent reliability and their effect upon subgrade resilient moduli. Conversely, for a given EAL and a given change in serviceability, the use of Equation 5 to obtain the subgrade resilient modulus results in a wide range of SN values, depending upon the choice of values for reliability and standard deviation.

PAVEMENT PERFORMANCE

The use of Equation 5 to obtain a subgrade resilient modulus, a value of 0.49 for standard deviation, and 92 percent reliability will yield an unrealistically low value of SN based upon past experience. Table 2 compares results using the above values in the 1985 Guide with results for the same "subgrade" condition and EAL used in the 1972 Guide.

DESIGN CRITERIA

Terminal serviceability has been replaced with "a change in serviceability". This is appropriate provided the original value of serviceability is known. However, in many cases, the initial value is not known and the pavement may be too old to estimate
the original serviceability. Thus, the "change in serviceability" is subject to becoming a "floating" value that cannot be specified. Likewise, a fixed value for "change in serviceability" may result in the pavement being allowed to deteriorate beyond the designer's original intention because construction practices did not meet expected quality standards. Conversely, if construction practices exceeded expected quality standards, the pavement might not be allowed to deteriorate to the expected condition and rehabilitation efforts might be initiated too soon.

COMPARISONS WITH 1981 KENTUCKY DESIGN METHOD

Pavement thickness designs using the 1981 Kentucky method (9) have been converted to an equivalent Structural Number using the same values for coefficients a1 and a2 used in Equation 2. While the SN values are very close to those shown in part of the 1985 Analysis No. 2 in Table 2, empirical experience has been correlated with theoretical solutions from elastic theory and subgrade moduli from Equation 5 (see 1985 AASHO Analysis No. 3 in Table 2). Dynamic deflection testing of pavements using the Road Rater also has been correlated with great success using Equation 5 to obtain the subgrade moduli (see 1985 AASHTO Analyses No. 3 in Table 2). However, values for Structural Number in Analyses No. 3, Table 2, differ greatly from the values using the 1981 Kentucky method.

SUMMARY

PROBLEMS OF IMPLEMENTATION

Figures 3-7 illustrate that the form of Equations 1 and 3 do not fit the fatigue data (3) observed at the AASHO Road Test as apparently has been reported (1-4). Even if the equation is assumed to be appropriate, the relationship between the various methods for evaluating soil support capabilities against "resilient modulus" is a gross approximation at this time. An intense research testing program is required to provide the required data to develop correlations between Resilient Modulus, Soil Support Value, CBR, R-value, and other soil support parameters.

At this time, an innocent designer may misuse the 1985 AASHTO Guide and obtain designs grossly too thin or too thick simply by choosing inappropriate values for any one or combination of parameters such as subgrade modulus, percent reliability, and standard deviation.

The 1985 AASHTO Guide does not provide the designer with an adequate relationship of load equivalency factors for steering axles, excessive tire pressures, and uneven load distributions known to exist on approximately 90 percent of the trucks having tandem or tridem axle groups. Relationships for these have been published (13). Research (13) has shown that the front axle may be accounting for as much as 40 percent of the total pavement fatigue because of the single tire and higher inflation pressures. Not accounting for these factors initially may result in the premature pavement failure that often is being observed.
currently.

LOSS OF SERVICEABILITY
The 1972 AASHO Guide (2) uses the terminal serviceability concept; the 1985 AASHTO Guide (1) employs the concept of loss in serviceability. The concept of loss in serviceability requires the designer to know the initial serviceability value upon completion of construction, or to assume an initial value for serviceability. The concept of loss in serviceability also raises some questions concerning the rates of change in serviceability for a pavement structure as a function of specific material and subgrade characteristics, quality control during construction, variations according to the accumulation of fatigue, and variations of the function of the specific facility (interstate, parkway, primary, secondary, etc.).

The concept of loss of serviceability is a major consideration for the application of the 1985 AASHTO Guide (1) for the design of overlays and/or other rehabilitation strategies. Research is needed to determine relative variations for the change in serviceability for new pavements relative to overlaid or rehabilitated pavements and/or for the various classes of facilities (interstate, parkway, primary, secondary, etc.).

RELIABILITY
The desired level of reliability acceptable for design may be a function of level of terminal serviceability, or loss of serviceability. The current format of the design monograph in the 1985 Guide (1) will require a mathematical shift to the resilient modulus to make a corresponding change in reliability from the value of 92 percent as used in the above analyses. Additional research is required to correlate the required changes in other parameters for changes in desired reliability. Additional sensitivity analyses are needed to provide the user with information relating the consequences associated with various assumptions.

STANDARD DEVIATION
The 1985 Guide (1) suggests using a value of 0.49 as the initial value. To determine the appropriate value for Kentucky conditions, or for other areas, will require an extensive research effort using Kentucky data, or data appropriate to the respective agency. One such effort may require the use of automated equipment for collecting better vehicle classification and weigh data. A second effort would require evaluating quality control procedures for the various stages of construction. Additional sensitivity analyses are needed to provide the user with information relating the consequences associated with the range of different assumptions.

CRUSHED STONE BASE MODULI
A part of the problem with Equation 6 arises from attempts by Van Til et al. (11) to correlate behavior with elastic theory in which the same pitfall Kentucky experienced in 1968 was encountered. One elastic modulus value for crushed stone
regardless of the moduli of the subgrade and asphaltic concrete. Kentucky research determined that the use of one value of modulus could not be correlated with observed behavior and deflection test results of in-place pavements. Instead, the modulus of the crushed stone base was found to vary as a function of the subgrade modulus as defined by Equation 5. Varying the modulus in this manner permitted the coupling of subgrade, base course, and asphaltic concrete moduli with deflection test results of in-place pavements and correlation of empirical behavior with the 1959 Kentucky thickness design curves (10). In summary, the use of one modulus value for the crushed stone base will produce irrational results for both weak and strong subgrades yet appear to be reasonable for a CBR range of approximately 4 to 7 (soil support values of 2.3 to 3.5). Additional research is required to determine the appropriate relationship of varying the moduli of the base as a function of the layer moduli above and below the base material.

SUBGRADE MODULI

Subgrade moduli obtained using Equation 5 correlate well with Kentucky experience and field test data. Prior research and testing of subgrade samples showed that Equation 7 correlates well with CBR test results of subgrade samples from the AASHO Road Test (12) and with Utah test results at a CBR 100 and Soil Support Value of 8.25 (Figure C.3-4 in Reference 2). Using Equations 4 or 6 requires a mathematical shift that will not agree with results using Equation 5 as input for resilient modulus in Equation 3. A testing program is needed to determine the proper relationship between elastic modulus, resilient modulus, and CBR.

In summary, it is prudent to state that suggesting the use of "Resilient Modulus" at this time may be premature. Resilient modulus testing may be the better approach in the future after a testing program provides data to develop correlations with soil evaluation methods currently in use. Table 2 provides evidence that almost ANY value for subgrade modulus may be obtained using various known relationships. It may be that the position of the Resilient Modulus scale in the 1985 AASHTO nomograph needs to be shifted, depending upon the ultimate correlations that may be developed.

DESIGN TERMINAL SERVICEABILITY

The 1981 Kentucky design method (9) has incorporated a variable level of terminal serviceability that is a function of design EAL. The 1985 Guide (1) does permit the designer to alter the level of terminal serviceability. A few questions needing answers prior to adoption of the 1985 Guide follow:

- How should the terminal serviceability vary as a function of design EAL?
- Should a lesser terminal serviceability be used to coincide with a shorter design time period?
- If so, what value should be used?
- Should the terminal serviceability vary as a function of highway classification?
"a" FACTORS IN EQUATION 2

For those agencies where the 1972 AASHO Guide (2) has been used for pavement thickness design, information relating the structural layer coefficients are documented. However, for those agencies where other procedures have been used, structural layer coefficients must be determined or assumed to apply the 1985 Guide. For most efficient implementation, research is needed to define those relationships between material properties (compressive strengths, modulus of elasticity, stability, resilient moduli, etc.) and structural layer coefficients.

Layer coefficients different from the values used in Equation 2 may be selected to produce the same structural number, depending upon the ratio of layer thicknesses. For example, substituting 0.36 for a1 and 0.18 for a2 and assuming D2 is twice D1, Equation 2 produces the same structural number as obtained when using the more typical values for the coefficients.

Unreported Kentucky research using fatigue data from the AASHO Road Test (3) has shown that the values for the "a2 and a3" coefficients in Equation 2 vary as a function of the thickness of the layer above and below the particular layer in question, even when the modulus for each layer is held constant. The coefficient also changes as the subgrade support varies. Analyses using elastic theory indicate the coefficient value also changes as a function of the modulus of the material used in that layer.

While the normal base course material has been a dense-graded limestone aggregate, there is increased interest and use of other materials. The value(s) to be assigned to these alternate materials and how those coefficients vary with layered conditions must be assessed. Thus, additional research is required to adequately define these variations and to recommend a procedure to incorporate those changing values into the design procedures.

LIST OF REFERENCES


5. H. F. Southgate and R. C. Deen, "Variable Serviceability
Concept for Pavement Design Confirmed by AASHO Road Test Fatigue Data", Research Report UKTRP-84-12, Kentucky Transportation Research Program, University of Kentucky, Lexington, KY, April 1984.


FIGURE 1. LOAD EQUIVALENCY FACTOR VERSUS SINGLE AXLELOAD (2).
FIGURE 2. LOAD EQUIVALENCY FACTOR VERSUS TANDEM AXLELOAD (2).
FIGURE 3. STRUCTURAL NUMBER VERSUS EQUIVALENT SINGLE AXLELOAD (EAL) FOR AASHO ROAD TEST DATA AT TERMINAL SERVICEABILITY LEVEL OF 3.5.

FIGURE 4. STRUCTURAL NUMBER VERSUS EQUIVALENT SINGLE AXLELOAD (EAL) FOR AASHO ROAD TEST DATA AT TERMINAL SERVICEABILITY LEVEL OF 3.0.
FIGURE 5. STRUCTURAL NUMBER VERSUS EQUIVALENT SINGLE AXLELOAD (EAL) FOR AASHO ROAD TEST DATA AT TERMINAL SERVICEABILITY LEVEL OF 2.5.

FIGURE 6. STRUCTURAL NUMBER VERSUS EQUIVALENT SINGLE AXLELOAD (EAL) FOR AASHO ROAD TEST DATA AT TERMINAL SERVICEABILITY LEVEL OF 2.0.
FIGURE 7. STRUCTURAL NUMBER VERSUS EQUIVALENT SINGLE AXLELOAD (EAL) FOR AASHO ROAD TEST DATA AT TERMINAL SERVICEABILITY LEVEL OF 1.5.
FIGURE 8. SUBGRADE RESILIENT MODULUS VERSUS STANDARD DEVIATION AS A FUNCTION OF PERCENT RELIABILITY FOR CBR 4 SUBGRADE.
FIGURE 7. SUBGRADE RESILIENT MODULUS VERSUS STANDARD DEVIATION AS A FUNCTION OF PERCENT RELIABILITY FOR CBR 8 SUBGRADE.
FIGURE 10. SUBGRADE RESILIENT MODULUS VERSUS STANDARD DEVIATION AS A FUNCTION OF PERCENT RELIABILITY FOR CBR 12 SUBGRADE.


### TABLE 1. COMPARISON OF STRUCTURAL NUMBERS OBTAINED FROM 1972 AASHTO INTERIM GUIDE (2) AND 1985 AASHTO GUIDE (1) AT 50 PERCENT RELIABILITY

<table>
<thead>
<tr>
<th>SUBGRADE</th>
<th>SOIL SUPPORT</th>
<th>RESILIENT MODULUS (psi)</th>
<th>STRUCTURAL NUMBER</th>
</tr>
</thead>
<tbody>
<tr>
<td>KENTUCKY</td>
<td>CBR=1500</td>
<td>4,507</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7,266</td>
<td>4.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9,607</td>
<td>6.00</td>
</tr>
</tbody>
</table>

### TABLE 2. COMPARISON OF STRUCTURAL NUMBERS OBTAINED FROM 1972 AASHTO GUIDE (2) AND 1985 AASHTO GUIDE (1) FOR STANDARD DEVIATION OF 0.49 AND 92 PERCENT RELIABILITY

<table>
<thead>
<tr>
<th>ANALYSIS NO. 1</th>
<th>ANALYSIS NO. 2</th>
<th>ANALYSIS NO. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1972 AASHTO DESIGN PROCEDURE</td>
<td>1985 AASHTO DESIGN PROCEDURE</td>
<td>1985 AASHTO DESIGN PROCEDURE</td>
</tr>
<tr>
<td>SOIL SUPPORT VALUE</td>
<td>CBR**</td>
<td>2.00</td>
</tr>
<tr>
<td>1972 AASHTO DESIGN METHOD</td>
<td>KENTUCKY SUPPORT STRUCTURAL NUMBER</td>
<td>9.172</td>
</tr>
<tr>
<td>EAL***</td>
<td>4.00</td>
<td>4.00</td>
</tr>
<tr>
<td>STRUCTURAL NUMBER</td>
<td>6.00</td>
<td>6.00</td>
</tr>
<tr>
<td>REQUIRED RESILIENT MODULUS BY EQUATION 7</td>
<td>2.288</td>
<td>2.288</td>
</tr>
<tr>
<td>STRUCTURAL NUMBER</td>
<td>10,000</td>
<td>10,000</td>
</tr>
<tr>
<td>KENTUCKY SUPPORT STRUCTURAL NUMBER</td>
<td>6,000</td>
<td>6,000</td>
</tr>
</tbody>
</table>

* ASSUMED VALUES:
  RELIABILITY = 92 PERCENT
  STANDARD DEVIATION = 0.49

** KENTUCKY SOAKED CBR

*** EAL = VALUE CALCULATED USING 1972 AASHTO PROCEDURE

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TABLE 3. COMPARISON OF RESULTS PRESENTED IN TABLE 2 USING SN = 4.00, CBR = 8 SOLUTIONS

<table>
<thead>
<tr>
<th>RESILIENT MODULUS:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1985 ANALYSIS NO. 1 / 1985 ANALYSIS NO. 2</td>
<td>$7266/3688 = 1.97$</td>
</tr>
</tbody>
</table>

DIFERRENCE IN STRUCTURAL NUMBERS:

| 1985 ANALYSIS NO. 2 - 1972 ANALYSIS | $= 4.98 - 4.00 = 0.98$ |
| 1985 ANALYSIS NO. 3 - 1972 ANALYSIS | $= 3.38 - 4.00 = -0.62$ |
| 1985 ANALYSIS NO. 2 - 1985 ANALYSIS NO. 3 | $= 4.98 - 3.23 = 1.75$ |

DIFFERENCE IN EQUIVALENT THICKNESSES OF ASPHALTIC CONCRETE:

(ASSUME COEFFICIENT FOR $a_1 = 0.44$ AND FULL-DEPTH ASPHALTIC CONCRETE)

| $0.98 / 0.44$ | $2.23$ inches |
| $-0.62 / 0.44$ | $-1.41$ inches |
| $1.75 / 0.44$ | $3.98$ inches |