Research Report
UKTRP-88-14

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGN SYSTEMS

by

Herbert F. Southgate
Chief Research Engineer

Kentucky Transportation Center
College of Engineering
University of Kentucky
Lexington, Kentucky 40506-0043

in cooperation with the
Transportation Cabinet
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and

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GOVERNOR

December 2, 1991

Mr. Paul E. Toussaint
Division Administrator
Federal Highway Administration
P.O. Box 536
Frankfort, KY 40602

Dear Mr. Toussaint:

SUBJECT: Research Report UKTRP-88-14
Comparison of Rigid Pavement Thickness Design Systems

As the title indicates, the focus of this research involved a comparison of rigid pavement design procedures. Rigid pavement design procedures investigated during this study include the 1986 AASHTO, the American Concrete Pavement Association (ACPA), the Portland Cement Association (PCA), and Kentucky methods. This report demonstrates the difficulty and complexity in comparing pavement design procedures.

Many of the figures and tabulations of data presented in this report already have been used by pavement design staff. Of special note is the information presented in Appendix C describing the variations in relationships for AASHTO soil support, resilient modulus, and other parameters for characterizing subgrade strength. The tabulations of Kentucky thickness designs and 1986 AASHTO thickness designs contained in Appendix F currently are being implemented by pavement design staff for development of a tabulation of pavement designs to be submitted to the FHWA for their review and concurrence. These designs will be used for rigid pavement designs on federally-funded projects in Kentucky.

The report has not demonstrated conclusively the superiority of Kentucky methods relative to 1986 AASHTO procedures. The report does demonstrate the relative strengths and weaknesses of the various procedures. This information is an obvious benefit to our pavement design staff in their efforts to determine the most effective and efficient pavement designs from both an engineering and economic perspective. At this time, current practice of using both the Kentucky procedures and the 1986 AASHTO procedures for determination of thickness requirements will continue.

Finally, the report demonstrates the need for continued study in this area. Kentucky has been a pioneer in the mechanistic design arena. Research will continue in this area. The current Kentucky procedures for rigid pavement design are based on an elastic layer analysis which assumes each layer has infinite horizontal dimensions. Current analyses are only valid for analysis of conditions at the center of the slab. We recognize the need for investigations using finite element analyses which will permit analysis of corner and edge conditions. Research to refine and improve rigid pavement design procedures will continue as time and funding permit.

Sincerely,

O. G. Newman, P.E.
State Highway Engineer

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16. Abstract

Rigid pavement thickness design systems investigated during this study were the 1986 AASHTO, American Concrete Pavement Association (ACPA), Portland Cement Association (PCA), and Kentucky methods. The ACPA system is a computer program based upon the 1986 AASHTO design equation. It was difficult to evaluate and compare the Kentucky method to the PCA system because the input and analysis procedures differ greatly.

The Kentucky method is based upon the fatigue relationship involving the value of work at the bottom of the concrete pavement caused by the applied load and repetitions of an 18-kip single axleload. The AASHTO method was derived from data obtained at the AASHO Road Test where the rigid pavements failed primarily due to pumping of the subgrade from under the slab. In Kentucky, pumping is a minor problem compared to failures caused by compressive forces at joint openings. Compression occurs due to annual temperature fluctuations resulting in slab movement and subsequent intrusion of debris into the joint openings. Eventually, the slab cannot move and compressive forces increase until failure occurs. Failure criterion used in the Kentucky thickness design system is quite different from the mode of failure observed at the AASHO Road Test and makes direct comparisons between design methods somewhat questionable.

The expression of soil stiffness values is a major contributor to the confusion arising between design methods.

Using elastic theory to develop load equivalency relationships, the ratio of rigid pavement EALs to flexible pavement EALs is approximately 1.1. According to W-4 Tables, the ratio of AASHTO rigid pavement EALs to AASHTO flexible pavement EALs is approximately 1.6. Thus, the AASHTO combination of pavement structures used in W-4 tables are not equivalent for fatigue calculations. Another combination should be chosen.

Thickness designs using the 1986 AASHTO, ACPA, and Kentucky methods can be made to match provided the terminal serviceability varies with Kentucky CBR. To help understand the behavior at the AASHO Road Test, published data for the cracking index, pumping index, and serviceability index were investigated. All three data sets influenced one another and could be correlated fairly well for serviceability values greater than 1.5. A method was devised to normalize the data to account for tire load and pavement thickness variations.

Serviceability data proved to be revealing. Of the 76 rigid pavement sections at the AASHO Road Test, 43 were given a serviceability rating greater than 1.5 at the end of testing operations. Of the 43, 10 had ratings between 2.5 and 4.0. The remaining 33 sections had ratings of 4.0, or greater. The 1986 AASHTO Guide recommends a terminal serviceability of 2.5 for major highway pavements. A 2.5 rating is appropriate for flexible pavements. From this investigation, a rating of 2.5 appears to be low for rigid pavements and suggests that a comparable set of serviceability values should be derived.

It is recommended that Kentucky Department of Highways use the 1984 Kentucky Concrete Thickness Design Curves for design of rigid pavements.

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EXECUTIVE SUMMARY

Rigid pavement thickness design systems investigated during this study were the 1986 AASHTO, American Concrete Pavement Association (ACPA), Portland Cement Association (PCA), and Kentucky methods. The ACPA system is a computer program based upon the 1986 AASHTO design equation. It was difficult to evaluate and compare the Kentucky method to the PCA system because the input and analysis procedures differ greatly.

The Kentucky method is based upon a fatigue relationship involving the value of work at the bottom of the concrete pavement caused by the applied load and repetitions of an 18-kip single axleload. The AASHTO method was derived from data obtained at the AASHO Road Test where the rigid pavements failed primarily due to pumping of the subgrade from under the slab. In Kentucky, pumping is a minor problem compared to failures caused by compressive forces at joint openings. Failure criterion used in the Kentucky thickness design system is quite different from the mode of failure observed at the AASHO Road Test and makes direct comparisons between design methods somewhat questionable.

Kentucky load equivalencies are based on work at the bottom of the concrete pavement as calculated by the Chevron N-layer computer program that is based on elastic theory. AASHTO load equivalencies were developed empirically from data collected at the AASHO Road Test. These load equivalencies include the effects of fatigue, pavement roughness, cracking, and pumping. Thus, the two sets of load equivalencies are based on different criteria and are not exactly equal. Analyses indicate that the average value for the ratio of AASHTO rigid EALs to Kentucky flexible EALs is 1.1. The ratio of AASHTO rigid EALs to AASHTO flexible EALs shown in Kentucky W-4 Tables is approximately 1.6. Thus, the chosen AASHTO pavement structures are not equivalent in fatigue.

A major portion of the discrepancy in thickness designs arises as a result of attempting to assign some numerical number to represent the stiffness of the subgrade. Literature review revealed 15 different scalar systems. Of the 15, one was Soil Support Value, one was AASHO 3 Pt., four scales were variations of R-value, four were variations of CBR, two were Resilient Modulus, and three were Modulus of Subgrade Support, k. To illustrate the confusion, according to one R-value scale, half of the soils in Kentucky would have a negative number indicating they were pure liquids on which a vehicle could not be supported.

Thickness designs using the 1986 AASHTO, ACPA, and Kentucky methods can be made to match provided the AASHTO EAL is adjusted to an equivalent Kentucky EAL, the percent reliability varies with thickness for a given CBR, and by Kentucky CBR.

To help understand the behavior at the AASHO Road Test, published data for the cracking, pumping, and serviceability indicies were investigated. All three data sets influenced one another and could be correlated fairly well for serviceability values greater than 1.5 and correlated to work as defined by classical physics. A method was devised to normalize the data to account for tire load and pavement thickness variations.

The 1986 AASHTO Guide recommends a terminal serviceability of 2.5 for major highway pavements. Of the 76 rigid pavement sections at the AASHO Road Test, 43 were given a serviceability rating greater than 1.5 at the end of testing operations. Of the 43, 10 had ratings between 2.5 and 4.0. The remaining 33 sections had ratings of 4.0, or greater. While no numerical method to account for a variable serviceability level or percent reliability was used directly in the development of the 1984 Kentucky curves, analyses indicate that the correlation between the AASHTO and Kentucky design methods requires the level of serviceability to increase as the design EAL increases for a given percent reliability. This confirms the Kentucky concept that as the design EAL is increased, the level of serviceability must be increased.

Appendix G contains tables of calculated design thicknesses for rigid pavements using the 1984 Kentucky thickness design curves and the 1986 AASHTO Guide for Design of Pavement Structures.

It is recommended that the Kentucky Department of Highways use the 1984 Kentucky Concrete Thickness Design Curves for design of rigid pavements.

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INTRODUCTION

The purpose of this study was to compare thickness designs for new rigid pavements using the Kentucky, Portland Cement Association, the American Concrete Pavement Association, the 1972 AASHTO, and 1986 AASHTO design methods. All input variables were to be assigned the same values for all methods so that differences between thicknesses would be a function of the design methodology. The 1986 AASHTO method has the largest number of variables and the Kentucky method has the fewest number of variables.

Pavement designers typically calculate a design thickness to one tenth of an inch and round to the next higher whole inch (if the calculated thickness is 9.4 inches, round to 10.0 inches). Conditions imposed upon in-service pavements vary, even within the same slab. Other designers feel that design systems should be refined for those variables reflecting the best available technology so that other variables may be "fine tuned" as research findings become available. The latter was assumed during this investigation.

The Kentucky thickness design curves for portland cement concrete (1) are based upon the principle of work and are shown in Figure 1. The criteria and development of these curves are contained in Appendix A.

COMPARISON OF THICKNESS DESIGN METHODS

Kentucky Rigid Pavement Design Curves

The 1984 Kentucky Thickness Design Curves for portland cement concrete pavements are shown in Figure 1 and were used to obtain a reference set of design thicknesses for designated values of 18-kip EAL and subgrade moduli. Results are listed in Table 1.

Assumed Values Used To Compare Design Methods

Table 2 lists assumed variables and corresponding values used to determine design thicknesses using the 1986 AASHTO (2), American Concrete Pavement Association (ACPA)(3, 4), and 1984 Kentucky design methods (1). ACPA developed a computer program based upon the 1986 AASHTO equation for rigid pavements.

Portland Cement Association

The only input factor included in Table 2 used in the PCA method is subgrade stiffness, k. Traffic and the associated EAL calculations are computed by procedures uncommon to any of the other methods included in this study. Appendix B contains the basis for the 1985 PCA design method.

Comprehensive analyses of pavement thickness designs using the 1985 PCA Thickness Design (5) method were not pursued because of the following observations:

1. In Figure 2 (Fig. 4. Design 1A., Reference No. 5), the trial pavement thickness of 9.5 inches on a subbase-subgrade k of 130 pci (CBR 3) for a pavement having doweled joints and without tied concrete shoulders shows that a 22.0-kip single axle is considered to be safe for an unlimited number of repetitions in terms of fatigue and 11 million repetitions when considering an erosion analysis. For a 21.6-kip single axle, the fatigue analysis allows an unlimited number of repetitions and 64 million repetitions considering an erosion analysis. For tandem axles, axleloads up to 57.6 kips are permitted at an unlimited number of repetitions for a fatigue analysis, and a 43.2-kip tandem is limited to 9.5 million repetitions when considering an erosion analysis. A 33.6-kip tandem axle group is allowed 92 million repetitions under an erosion analysis. These numbers do not appear to agree with observed performances of interstate pavements which are failing in less than 20 years when the legal load limits are 20 kips for single axles and 34 kips for tandem axles. Such a discrepancy might suggest that limiting "fatigue" relationships for the PCA, AASHTO, and Kentucky procedures are not identical.

2. If the 1966 PCA design method (5) produced reasonable design thicknesses that generally agree with results using the 1972 AASHTO method (6), then the change in criterion permitting higher stress ratios for the same EAL may have been in error. Perhaps better agreement would be obtained if the criterion line had remained as shown in Figure 3 or shifted toward lesser stress ratios for the same 18-kip EALs.

3. Development of the PCA method was based upon use of a finite element program. Program results depend upon selection of input values to describe material characteristics and behavior. For some parameters, chosen values may have significant impact upon calculated results. It is possible that some input values should be adjusted to produce more reasonable results.

American Concrete Pavement Association

The ACPA (3, 4) furnished computer programs to design both flexible and rigid pavements based upon design equations presented in the 1986 AASHTO Design of Pavement Structures (2). The 1986 and 1988 ACPA computer programs (3, 4) were used to evaluate the 1986 AASHTO design equation (2). The input variables and associated numerical values were not identical for the two computer programs. The 1986 ACPA (3) computer program utilized resilient modulus, M_r, as the input parameter to describe soil stiffness. The 1988 (4) computer program used the modulus of subgrade reaction, k, to describe soil stiffness. Input values shown in Table 2 of the main text were based on the 1986 ACPA (3) version. Keeping all input values the same for the 1988 ACPA (4) version requires using a k value based on M_r = 1,500 x CBR for the 1986 version to obtain the same rigid pavement design thickness. The ACPA (4) version avoids the problem of identifying the resilient modulus relationships by using the modulus of subgrade reaction, k, and letting the designer determine which k-M_r relationship should be used.

1986 AASHTO Design Method

Figure 4 displays the relationships between Kentucky resilient modulus which utilizes the relationship 1,500 x CBR and resilient modulus relationships for the ACPA and AASHTO design methods. Dorman and Metcalf (7) used 1,500 x CBR to represent resilient modulus for clay subgrades. Rada and Witczak (8) state that the 1,500 value is too high for sands.

Table 2 contains input data for variables included in the 1986 AASHTO computer program for computing pavement thickness. The 1986 ACPA computer program (3) uses resilient modulus (not the modulus of subgrade reaction, k) as the soil stiffness input parameter. Thicknesses resulting from use of the computer program are listed in Table 3. The Kentucky method does not consider a thickness design less than 6 inches. Figure 5 illustrates the relationship between Kentucky's thickness designs and the 1986 AASHTO's designs for a serviceability level of 2.50, 80 percent reliability, for six design EALs, and four CBRs. Figure 5 illustrates that Kentucky design thicknesses are more sensitive to subgrade stiffness than are the AASHTO design thicknesses. Figures 6-9 show the relationship between design thicknesses for six levels of design EALs and five serviceability levels. Note the increasing change in level of serviceability for equal design thicknesses and the slight increase as a function of CBR. Figure 10 shows the relationship between an increasing percent reliability as the design EAL increases for equal design thicknesses and for constant CBR and level of serviceability.

1986 AASHTO Design Method -- Design Nomographs

Figures 11 and 12 are the 1986 AASHTO Design Charts for rigid pavements and must be used together. Examination of the nomograph shown in Figure 11 (2) reveals that the beginning relationship is k (effective modulus of subgrade reaction, pci) and not resilient modulus. Appendix FF (2) provides the relationship for resilient modulus as a function of soil support as shown in the top right corner of Table 4. Regression analyses were made to fit a second degree polynomial equation and a straight line equation to the input data. The correlation coefficient, R^2, is at least 0.9997 for both equations. The lower part of Table 4 evaluates both regression equations using soil support number as X and resilient modulus as Y. Using the relationship for k and resilient modulus shown in Figure 13 (Appendix HH, Reference 2) provides corresponding values for k which are included in Table 4. Values for k corresponding to values of resilient moduli used as input (Table 1) are:

Resilient Modulus, psi (1,500xCBR)	k, pci (1,500XCBR)	k, pci (M_R/19.4)
4,500	232	106
7,500	387	150
10,500	541	186
15,000	773	238

Table 3 contains thickness designs for combinations of 80 and 90 percent reliability, serviceability levels of 2.5, 2.75, 3.00, 3.25 and 3.5, for subgrade resilient moduli of 4,500, 7,500, 10,500, and 15,000 psi, and the six fatigue design values included in Table 1.

INFLUENTIAL FACTORS IN THE 1986 AASHTO DESIGN METHOD

Subgrade Resilient Moduli Relationships With Soil Support

The 1986 AASHTO Guide (2) provides two mathematical relationships for Resilient Modulus of Soil, M_r. The first relationship is (2, Page FF-5) which is:

$$S_i = 6.24 \times \log_{10} M_r - 18.72 \qquad (1)$$

Appendix C contains the development of Equation 1 and has been rearranged as:

$$\mathrm{Log}_{10}(M_r) = 2.997414 + 0.160345 S_i \qquad (2)$$

The calculated EAL using Equation 2 is the same as that calculated using the 1972 AASHTO design equation.

The second relationship for Resilient Modulus of Soil, M_r, is (2, Page I-14):

$$M_r\ (\text{psi}) = 1{,}500 \times \text{CBR} \qquad (3)$$

Equation 3 is the basis for the development of the Kentucky rigid and flexible pavement thickness design curves.

The Kentucky relationship between CBR and soil support value was the result of round robin tests conducted in the late 1960's on subgrade samples obtained after the AASHO Road Test was completed. A Kentucky CBR of 5.3 was determined to correspond with an AASHO soil support value of 3.0 (9). Kentucky designs relating fatigue to in-place soil test values were analyzed and converted to corresponding values of soil support. A best fit line for those data passed through the point of a CBR of 100 and a soil support number of 8.25 which is also the same value shown in Figure C.3-4 of the 1972 AASHTO Guide (Figure 14). Figure 15 contains the two relationships for resilient modulus and corresponding values of soil support. A literature review revealed multiple methods for defining subgrade stiffness of which Figure 15 is one of many.

Appendix D contains a lengthy discussion of the problems related to soil stiffness values revealed during the literature review. One example is that approximately the weakest half of Kentucky soils would have a negative R-value. Literature review revealed that for the given Soil Support Value scale, there were four scales labeled CBR, four labeled R-value, two labeled Resilient Modulus, and three as "k", modulus of subgrade reaction. Figure 16 is a compilation of these scales.

Level Of Serviceability With EAL

The Kentucky fatigue relationship is based upon work at the bottom of the concrete slab and not upon level of serviceability. Assuming that the design EAL is the same for both the 1986 AASHTO and 1984 Kentucky design methods, Figures 17-21 contain thickness design curves by the 1986 AASHTO Guide for 80 percent and 90 percent reliability for a k = 106 pci and Kentucky thickness design curve for a subgrade modulus of 4,500 psi (CBR 3) for levels of serviceability 2.50, 2.75, 3.00, 3.25, and 3.50, respectively. Figures 22 and 23 (k = 106 and 238 pci, respectively) show that the Kentucky thickness designs for increasing 18-kip EALs correspond generally with increasing percent reliability according to design thicknesses using the 1986 AASHTO Guide (2). Figure 22 shows that relationship of percent reliability as a function of design EAL will exceed 85 percent for 18-kip design EALs of 10 million, or more, when k = 106 pci (AASHTO resilient modulus relationship, Figure 15). Likewise, Figure 23 shows that the same 18-kip design EALs will exceed 90 percent reliability when k = 238 pci (M_r = 1500 x CBR).

Load Equivalency Factors

Kentucky load equivalency factors were developed using all 100 possible combinations of layer thicknesses for flexible pavement sections constructed at the AASHO Road Test (10). These factors were based upon a matrix of loads placed on these sections and subjected to analyses using the Chevron N-layer computer program as modified by Kentucky (11). Factors were developed for a two-tired steering axle, four-tired single axle, eight-tired tandem, 12-tired tridem, and other combinations not in common use. These factors are used to calculate 18-kip EALs for a given traffic stream and projected for a design based upon the designer's choice of a specific number of years. The logic used was that only one set of EAL calculations would be made. The pavement thickness should be adjusted to be valid for the fatigue relationship without regard to the specific pavement material.

Experience with rigid pavements in Kentucky indicated that a 10-inch concrete pavement on a CBR 5 subgrade was the normal design for interstate traffic and corresponded to 8 million 18-kip EALs for both asphalt and concrete. The legal load limit of 80,000 pounds can be carried by a 5-axle semi-trailer truck having 18 tires. An 8 million EAL design thickness of 10 inches would correspond to:

80,000 pounds / 18 tires / 10 inches = 445 pounds (0.445 kip) per tire per inch of concrete thickness.

As will be shown later, this also corresponds to a serviceability level of 4.0 as determined from data collected at the AASHO Road Test.

The AASHTO fatigue equation included the effects of cracking in the concrete and pumping of soil as well as fatigue effects of axleloads. The Kentucky load equivalency factors are based on the fatigue of concrete only and in terms of work as defined by classical physics. Thus, a discrepancy of some magnitude should be expected. Appendix E contains the development of the Kentucky load equivalency relationships.

Appendix F contains figures for a fixed level of serviceability of 2.50. Figure F1 displays the relationship of AASHTO EALs and thickness designs for CBR 3 as a function of percent serviceability. Figures F2 and F3 correspond to CBRs 7.5 and 15 respectively. Figures F4-F6 contain the Kentucky thickness designs superimposed on the AASHTO family of curves in Figures F1-F3, respectively.

OTHER INFLUENTIAL FACTORS CONSIDERED DURING THE DEVELOPMENT OF THE 1986 AASHTO DESIGN METHOD FOR RIGID PAVEMENTS

General Comment

Comparing thickness designs from previous analyses indicates there is reasonably good agreement for CBR 3-7 designs (subgrade modulus of 4,500 psi, to 10500,psi, Figures 6-8). However, there are other factors influencing the design values that are not direct inputs to the design equation. Factors that are not directly included but which were used in the development were pavement cracking and pumping of the subgrade. Pavement serviceability was included in the design equation; but, the relationship of cracking and pumping with serviceability was not used. Data were obtained at the AASHO Road Test (12) describing these factors that should aid in understanding their effects upon each other and resulting calculations using the design equation.

Cracking Index

For the same axleload, the amount of cracking in a pavement should increase if the pavement thickness is decreased and vice versa. Increases in axleloads should cause an increase in the amount of cracking for the same pavement thickness.

A cracking index was used at the AASHO Road Test to describe the deterioration of the concrete as the test progressed. Table 5 is a copy of Table 50 (12) which provides the cracking index values for each rigid section at the end of the Road Test. The index was defined as the number of feet of cracking per 1,000 square feet of pavement (an area approximately 12 feet by 83 feet). A cracking index of 48 corresponds to two transverse cracks in a 12-foot wide by 42-foot long slab. Many slabs in Kentucky have two transverse cracks. Data contained in Table 5 were difficult to interpret and a more meaningful interpretation of the cracking index values was sought. The calculated theoretical work at the bottom of the rigid pavement caused by the actual axleload applied to that pavement section was obtained from the Chevron N-layer computer analyses for the pavement sections. Correlations are shown in Figure 24.

Figure 25 is another way of analyzing the cracking index data shown in Table 5. In Figure 25, the total vehicle load on that respective pavement was divided by the total number of tires on that vehicle and that quotient was divided by that pavement thickness. There is a strong resemblance between the data patterns in Figures 24 and 25. While the values of cracking index are the same in Figures 24 and 25, the average tire load per inch of concrete thickness in Figure 25 is based upon known loads (13) and thicknesses at the AASHO Road Test. The calculated work shown in Figure 24 is a theoretical number. This suggests that the observed data (Table 5) may be supported and explained through elastic theory.

Pumping Index

A pumping index also was developed at the AASHO Road Test to quantify the volume of unbound material that was pumped from beneath the pavement by traffic. Table 6 is a copy of Table 54 (12) which provides the values for the pumping index for each rigid section at the end of the Road Test. The index was defined as the volume of soil expressed in cubic inches per linear inch of pavement. An index value of 144 corresponds to a 1-inch deep void per linear inch of a 12-foot wide slab.

Photographs included in Report 61E (12) show small mounds of materials as a result of the subgrade being ejected from under the slab when the axleload passes over that spot. Similar mounds have been noted in Kansas and Oklahoma. This type of pumping action is not common in Kentucky. A typical Kentucky condition is that fines are pumped from within the dense-graded aggregate base below the rigid pavement. Rain and wind from passing traffic remove fines from the shoulder surface. After some time, a void is created beneath the slab and may result in faulted joints or cracks. The height of the fault corresponds to the depth of the void.

Table 6 includes data for single-axle semi-trailer trucks (Lane 1) and tandem-axle semi-trailer trucks (Lane 2). Pavement thicknesses in both lanes of the same loop were identical and permit comparison of the pumping index values for the same thickness versus lanes--synonymously with number of loaded axles. Figure 26 shows that the pumping index for Lane 2 is approximately twice that of Lane 1. This suggests that the pumping index is a function of the actual number of axles passing over that spot rather than the number of "load applications" (such as a tandem group).

To better interpret the values shown in Table 6, analyses similar to that described above under "CRACKING INDEX" were performed except that the work was the value calculated at the top of the subgrade. Results are shown in Figure 27. Figure 28 shows the same pumping index values for the average tire load per inch of concrete. Figures 27 and 28 correspond to Figures 24 and 25 for cracking index. Except for four data points, data separate into two distinct groups--pavements that had reached failure (serviceability index = 1.5) before the end of the AASHO Road Test and those that had not reached failure (serviceability index > 1.5). Data corresponding to a P_t >1.5 cluster in a small area representing a low index value and a relatively low value of average tire load per inch of concrete and/or "work". Data representing failed pavements (P_t = 1.5) appear to "explode" and the scatter is comparatively large. One explanation might be that once the pavement has cracked, the smaller pieces are more easily moved in a rocking motion by passing axleloads. The result is a cyclic action of more subgrade being pumped out until there is a sufficient void so that the stresses in the "cantilevered" slab are relieved by an additional crack in the pavement. The similarity between Figures 27 and 28 suggests that the observed behavior of the subgrade at the AASHO Road Test may be explained by elastic theory.

Cracking Index Versus Pumping Index

If the supporting layer beneath the pavement is removed and contact is lost, the slab should crack. The cracking index should increase if the pumping index increases and vice versa.

Matching data in Tables 5 and 6 permitted the creation of Figure 29. Note the strong correlation for pavements having a serviceability index greater than 1.5 and the relatively large scatter of data for pavements having reached a serviceability index of 1.5. Figures 25 and 28 indicate this pattern should be true.

Serviceability Index

Patterns shown in Figures 25 and 28 suggest the possibility of correlating values of serviceability index and average tire load per inch of concrete. Appendix A, Report 61E (12) contains values for serviceability index at the end of the AASHO

Road Test for those rigid pavements still in service and were correlated to the pavements by respective loop number and load as shown in Tables 5 and 6. Figures 30 and 31 show the relationship between serviceability index versus cracking index and pumping index, respectively. Regression analyses were made for serviceability index versus cracking index and pumping index and a serviceability index scale superimposed on Figures 25 and 28, respectively. A matrix of values calculated by evaluating each regression equation was submitted to regression analyses also and the results of the regression permitted superimposing a mean fit serviceability index scale on Figure 29.

Figure 32 illustrates the relationship between serviceability index and average tire load per inch of concrete thickness. All data points are for pavement sections that survived to the end of the test at the AASHO Road Test. The following observations are made for Figure 32.

1. Note the distinct lower boundary that might correspond to those pavements for which the ratio of actual stress due to axleload to rupture stress has reached some minimum. PCA suggests (Figure B1) that failure will not occur when the stress ratio is less than 0.40 to 0.50.

2. The data pattern suggests a strong correlation of loss of serviceability with time or fatigue. In Figure 32, time, or fatigue, is implied within the value for serviceability index in combination with load per tire per inch of concrete.

The following summary table shows the number of AASHO Road Test sections that failed ($P_t = 1.5$) prior to the end of testing. The data points are shown in Figures 25 and 28, but not in Figure 32. Figure 33 is a combination of Figure 32 and the data summarized in the following table.

Kips per Tire per Inch of Concrete	No. of Sections
0.57	1
0.60 - 0.65	11
0.66 - 0.86	17

DISCUSSION

Four rigid pavement thickness design systems were investigated and only the PCA and Kentucky methods were independent of the 1986 AASHTO method. Of the two involving the 1986 AASHTO method, one uses the nomograph and the other used a computer program based upon the 1986 AASHTO equation to compute rigid pavement thickness. Direct comparison of the systems was difficult because of the following major differences in basic relationships and criteria.

- Fatigue-load equivalency relationships:
  1. PCA is based upon an allowable number of repetitions as a function of specific axleloads.
  2. AASHTO uses the rigid pavement thickness design equation to develop load equivalencies for four-tired single axles, eight-tired tandems, and twelve-tired tridems.
  3. 1984 Kentucky method is based upon a theoretical relationship between work calculated at the bottom of the rigid pavement and repetitions as developed from a fatigue relationship merging compatible tensile strain versus allowable number of repetitions from PCA and 1972 AASHTO design methods.

- Subgrade strength relationships:
  1. PCA uses the relationship of 800 x CBR.
  2. 1986 AASHTO method is based upon subgrade reaction, k, developed by Westegaard and correlates a "k" with a resilient modulus, M_r. M_r is related to soil support value through an equation provided in the 1986 AASHTO Guide.
  3. 1984 Kentucky method is based upon the subgrade modulus equal to 1,500 x Kentucky CBR. Correlation with soil support value, SSV, was developed through laboratory testing of AASHO Road Test soils in the late 1960s. Another correlation with M_r was obtained through resolving the mathematical terms in the AASHTO design equations involving SSV in the 1972 AASHTO Guide and M_r in the 1986 AASHTO Guide.
  4. With so many confusing relationships of subgrade stiffness with Soil Support Value, it seems appropriate to revert to the original relationship between CBR and "k" developed and reported by the Corps of Engineers in 1942 (14) as shown in Figure 16 (also same as Figure D9 in Appendix D).

- Rigid Pavement Thickness Relationships:
  1. PCA method is based upon results of a finite-element computer program.
  2. 1986 AASHTO is based upon the pumping of the subgrade, cracking of the concrete, and repetitions of loads applied to various pavement

thicknesses at the AASHO Road Test. The system is based primarily upon empirical data. It is generally recognized that one important limitation is that there was only one soil type used at the AASHO Road Test. 1984 Kentucky method is based upon the fatigue relationship between the number of 18-kip equivalent repetitions and work at the bottom of the slab. The design thickness is the thickness required to match the allowable value of work correlated to the calculated work at the bottom of the rigid pavement.

3. While the Kentucky rigid pavement thickness design curves were developed based on work at the bottom of the concrete, Appendix E contains the methodology to adjust the criterion to equivalent values at the top of the subgrade. It is shown in Appendix E that there is relatively little difference between the sets of load equivalencies appropriate to the bottom of the concrete or the top of the subgrade.
4. Elastic theory was used to determine thickness relationships as a function of CBR. Empirical data were correlated to theoretical results. Many factors considered in the AASHTO method are either implied or not considered. Such factors include coefficient of load transfer, subgrade drainage coefficient, and variable levels of serviceability. The most severe limitation at this time may be not being able to directly vary level of serviceability. However, empirical observations suggest that the level of serviceability of rigid pavements remains almost constant for most of the pavement's fatigue life and failure occurs over a relatively short time or relatively few additional repetitions.

The important observations from these analyses were:

1. The Kentucky thickness design curves correspond to a variable level of serviceability as a function of CBR as shown in Appendix F.

2. The AASHTO curves are not parallel to the Kentucky designs corresponding to a line of equality as shown in Figures 6-9.

3. The difference in design thicknesses between the Kentucky (1) and AASHTO (2) methods are more fundamental in nature. The AASHTO method (2) is based upon empirical observations of cracking of the slabs and loss of subgrade by pumping. The Kentucky design criterion is based upon the amount of work at the bottom of the rigid pavement slab. Thus, true comparisons between the two thickness design systems should result in differences between thickness designs.

4. Normalizing the various loads and pavement thicknesses employed at the AASHO Road Test into the parameter "average tire load per inch of concrete thickness" aided in reducing the scatter of empirical data for

cracking index and pumping index (Figures 25 and 28, respectively). The similarity of patterns in Figures 24 and 27 indicates that the empirical data may be supported by elastic theory.

5. Serviceability index ratings for the concrete pavements at the end of testing at the AASHO Road Test versus the average tire load per inch of concrete (Figure 32) may be a useful tool to estimate the relative serviceability level for combinations of average tire load and concrete pavement thickness. Caution should be exercised because Figure 32 implies a fatigue relationship, but the maximum fatigue is limited to the maximum value recorded at the end of the test and not the end of the pavement section's fatigue life for over half of the rigid sections. An example of how Figure 32 might be used follows. A value of 0.445 kip per inch of concrete appears to be the point where lesser values would correspond to a minimum value of 4.0 serviceability. For an 80-kip 5-axle semi-trailer truck having 18 tires, the average load per tire is 4.44 kips. Dividing 4.44 kips per tire by 0.445 kip per tire per inch of concrete results in a minimum pavement thickness of 9.98 inches, or 10 inches. For a gross load of 73,280 lb., the quotient would be 4.071 kips per tire. Dividing by 0.445 results in a pavement thickness of 9.15 inches--very nearly the thickness used to select the load equivalency values for use in calculating the FHWA W-4 Tables. The 5-axle semi-trailer truck assigned to Lane 2 of Loop 4 had a gross load of 73,500 pounds. For a 10-inch pavement, a 100-kip load on the same truck would yield an average load per tire of 5.56 kips and 0.556 kip per tire per inch of concrete. Using the line marked "conservative limit" in Figure 32, the expected level of serviceability would drop from 4.0 to approximately 3.0 for the same number of vehicle loadings. To maintain the 0.445 kip per tire per inch of concrete would require approximately 12.4 inches of concrete pavement.

6. A 10-inch concrete pavement has been a typical Kentucky design. This design corresponds to a combination of 0.445 kip per tire per inch of concrete and a minimum serviceability level of 4.0 based upon AASHO Road Test data. The 1986 AASHTO Guide (2) suggests (2215, page II-12), "...An index of 2.5 or higher is suggested for design of major highways and 2.0 for highways with lesser traffic volumes...Following are general guidelines for minimum levels of P_t obtained from studies in connection with the AASHO Road Test (12):"

Terminal Serviceability Level	Percent of People Stating Unacceptable
3.0	12
2.5	55
2.0	85

The above values are appropriate for the flexible pavements tested at the AASHO Road Test. Figure 32 contains empirical data for non-failed concrete pavements at the AASHO Road Test at the end of testing. Because the same number of vehicle trips were applied to both flexible and rigid pavements at the AASHO Road Test, the trends in Serviceability Index suggest that the values in the discussion and table above are too low for rigid pavements compared to Figures 32 and 33. A comparable set of adjusted values is needed for rigid pavements.

7. Comparable thickness designs can be obtained using the 1986 AASHTO Guide, the ACPA computer program, and the Kentucky rigid pavement design curves provided the terminal serviceability varies as a function of percent reliability and Kentucky design CBR as shown in Appendix F.

CONCLUSIONS

o Figure D9 (Appendix D) shows 15 different scales to assign a value of stiffness to subgrade materials. Soils having a Kentucky CBR of 4, or less, would be classified as "liquids" if scales 7 or 8 were chosen for use in Kentucky. Thus, a universal method needs to be developed to assign stiffness values to soil.

o True comparison between the Kentucky and 1986 AASHTO rigid design methods are somewhat questionable because the design criteria are totally different. The 1986 AASHTO design procedure is an empirical method based upon pumping of the subgrade from under the pavement while the Kentucky method is based upon the work at the bottom of the concrete slab (calculated by elastic theory) coupled with empirical experience.

o Comparable thickness designs may be obtained using the 1986 AASHTO Guide, the ACPA computer program, and the Kentucky rigid pavement design curves if the terminal level of serviceability varies with percent reliability and design CBR as shown in Appendix F. Differences may become comparatively large depending upon the choice of value for reliability, serviceability, load transfer coefficient, resilient modulus (coefficient of subgrade reaction, k), subgrade drainage coefficient, and choice of load equivalency relationships (based upon the designer's choice of pavement thickness). Most of the above factors are not specific parameters in the Kentucky design method. The effects of some of them are included implicitly. Analyses of the cracking and pumping indices indicate that elastic theory can be used to explain and support the observed behavior as shown in Figures 24 and 27 respectively. Analyses indicate that a serviceability scale may be fitted to the observed data using a regression equation fitted to the data.

o Load equivalency relationships used in the 1986 AASHTO Guide vary as a function of pavement thickness, but not as a function of subgrade support.

o Figure E17 (Appendix E) shows that the ratio of AASHTO rigid EAL to Kentucky flexible EAL has an average value of 1.1 across all CBRs. Figure E15 (Appendix E) shows the ratio AASHTO rigid EALs to AASHTO flexible EALs as recorded in Kentucky W-4 Tables from 1965 through 1985. The 1965 average is approximately 1.4 and has increased to 1.65 by 1985. One conclusion is that the rigid and flexible structures chosen for comparison are not equivalent structures. Therefore, the sets of load equivalencies are not compatible as "equivalent for fatigue calculations".

o Load equivalency relationships for rigid pavements developed during this study were based upon elastic theory analyses used to develop the Kentucky thickness design curves for rigid pavements. These load equivalencies vary as a function of pavement thickness and subgrade stiffness. Load equivalency relationships appropriate to Kentucky's flexible pavement thickness design curves are based on analyses of CBR 4 subgrade only, but do include the effects of all possible combinations of layer thicknesses for the flexible pavements constructed at the AASHO Road Test. Thus, there is no one set of load equivalencies appropriate to all conditions regardless of design method as shown by analyzing 34,025 trucks weighed in Kentucky during 1989 using WIM scales.

o Analyses of the serviceability index values recorded at the AASHO Road Test indicate that only 42 of the 76 rigid pavement sections had a terminal serviceability of 2.5 or greater. The effects of increasing load and increasing pavement thickness were minimized by dividing the gross vehicle load by the number of tires and then dividing that quotient by the pavement thickness to produce a scaler of "average tire load per inch of concrete thickness". The data relating terminal serviceability and the cited scaler indicate that 33 of the 46 surviving pavement sections had a terminal serviceability of 4.0 or greater. This suggests that the recommended terminal serviceability of 2.50 for major highways is appropriate for flexible pavements but too low for rigid pavements. Since both types of pavements at the AASHO Road Test were subjected to the same number of load applications, it is suggested that an appropriate set of terminal serviceability values should be developed for rigid pavements. Analyses indicate these values should be higher than the set now included in the Guide.

o Analyses indicate it may be possible to determine internal stress distributions through the concrete pavement thickness using elastic theory. If so, using the cracking index and serviceability index values might yield a relationship to determine the stress ratios induced in those pavements. Such a relationship might provide a method to limit cracking and/or pumping. Extrapolation of thickness designs for other subgrade stiffnesses (by whatever system) might be made with greater confidence in the final design curves.

SUMMARY

o Figure 16 shows that for the one scale of Soil Support Value, there are many methods to evaluate subgrade stiffness--four scales involving R-value, three scales for CBRs, two scales for resilient modulus, M_r, one scale for AASHTO 3 pt. values, and three scales for subgrade modulus of reaction, "k". For at least one relationship of R-value, the weakest half of Kentucky soils would have a negative value, while on other scales, there might be unrealistically high values. Agreement is not evident on what is the resilient modulus scale corresponding to CBR, R-value, Soil Support Value, and particularly "k". The original Corps of Engineers CBR vs k relationship was found in the literature (14) and is labeled as Scale Number 15 on Figure 16 (also Figure D9).

o The strengths and weaknesses for the design systems investigated during this study have been identified. Severe limitations exist when attempting to compare thickness designs using the different design methods.

o Elastic theory has been used to analyze some of the empirical data for rigid pavements obtained at the AASHO Road Test. It is possible that better failure criteria may result from these analyses by matching elastic theory analyses with observed data obtained at the AASHO Road Test.

o Analyses of serviceability data for surviving rigid pavement sections at the AASHO Road Test show that 33 of the 76 sections had a terminal serviceability of 4.0 or greater. The 1986 AASHTO Guide recommends a terminal serviceability of 2.5 be used for major highways. On that basis, 33 sections would not have survived. Of the remaining 43 sections, 10 had been assigned a serviceability rating between 2.5 and 4.0. The remaining 33 sections had been given a rating of 4.0 or greater.

o Analyses of load equivalencies indicate that the combination of AASHTO pavement structures chosen for calculating ESALs are not equivalent in fatigue behavior.

RECOMMENDATIONS

1. The Kentucky rigid pavement thickness design method should continue to be used by the Department of Highways. The method is based upon elastic theory and all thicknesses were developed using the same fatigue criterion. The 1986 AASHTO Guide has many desirable features and is based upon the cracking index and pumping index used at the AASHO Road Test. The pumping criterion is not appropriate for Kentucky subgrades.

2. Appendix D provides the methodology to correlate CBR, R-value, resilient modulus, and modulus of subgrade reaction, k.

3. Direct correlation of AASHTO rigid ESALs to Kentucky flexible ESALs cannot be made. However, ranges have been established indicating that AASHTO rigid ESALs are approximately 1.1 times the Kentucky flexible ESALs versus approximately 1.6 as the ratio of AASHTO rigid EALs to AASHTO flexible EALs as shown in the W-4 Tables. It should be emphasized that the ratios of 1.1 and 1.6 were obtained from limited data and should be accepted as relative indicators rather than absolute values.

4. Sensitive and non-sensitive input variables have been identified.

5. Tables of calculated design thicknesses for rigid pavements have been made and are included in Appendix G. These tables may be used to develop a standard set of thickness designs corresponding to subgrade stiffness and construction equipment thickness limitations.

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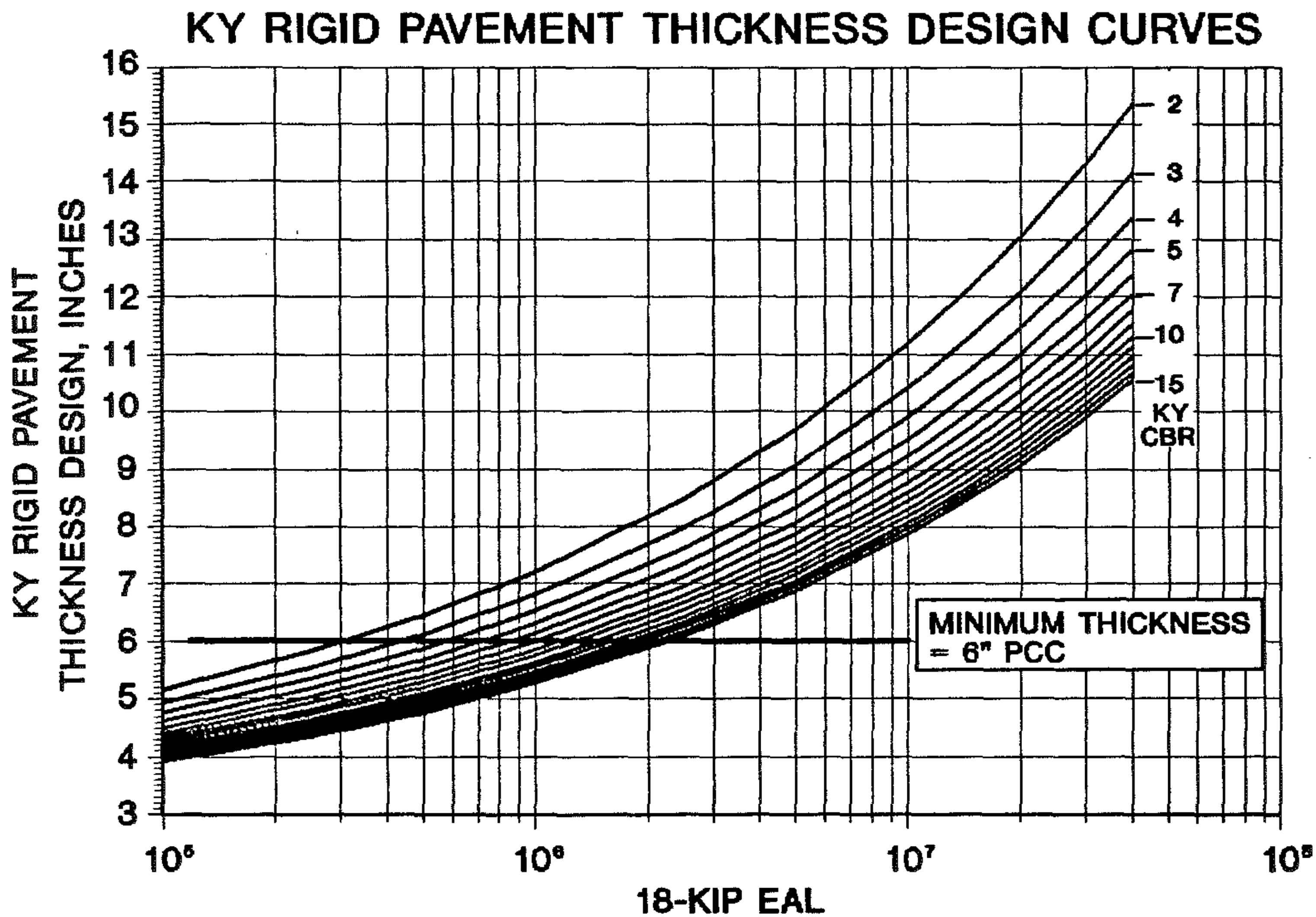


FIGURE 1. KENTUCKY THICKNESS DESIGN CURVES FOR RIGID PAVEMENT.

Calculation of Pavement Thickness

Project Design 1A, four-lane Interstate, rural

Trial thickness 9.5 in.

Subbase-subgrade *k* 130 pci

Modulus of rupture, MR 650 psi

Load safety factor, LSF 1.2

Doweled joints: yes ✓ no ____

Concrete shoulder: yes ____ no ✓

Design period 20 years

4-in. untreated subbase

Axle load, kips	Multiplied by LSF 1.2	Expected repetitions	Fatigue analysis: Allowable repetitions	Fatigue analysis: Fatigue, percent	Erosion analysis: Allowable repetitions	Erosion analysis: Damage, percent
1	2	3	4	5	6	7

8. Equivalent stress 206

9. Stress ratio factor 0.317

10. Erosion factor 2.59

Single Axles

30	36.0	6,310	27,000	23.3	1,500,000	0.4
28	33.6	14,690	77,000	19.1	2,200,000	0.7
26	31.2	30,140	230,000	13.1	3,500,000	0.9
24	28.8	64,410	1,200,000	5.4	5,900,000	1.1
22	26.4	106,900	Unlimited	0	11,000,000	1.0
20	24.0	235,800	"	0	23,000,000	1.0
18	21.6	307,200	"	0	64,000,000	0.5
16	19.2	422,500			Unlimited	0
14	16.8	586,900			"	0
12	14.4	1,837,000			"	0

11. Equivalent stress 192

12. Stress ratio factor 0.295

13. Erosion factor 2.79

Tandem Axles

52	62.4	21,320	1,100,000	1.9	920,000	2.3
48	57.6	42,870	Unlimited	0	1,500,000	2.9
44	52.8	124,900	"	0	2,500,000	5.0
40	48.0	372,900	"	0	4,600,000	8.1
36	43.2	885,800			9,500,000	9.3
32	38.4	930,700			24,000,000	3.9
28	33.6	1,656,000			92,000,000	1.8
24	28.8	984,900			Unlimited	0
20	24.0	1,227,000			"	0
16	19.2	1,356,000				
				Total 62.8		Total 38.9

Fig. 4. Design 1A.

FIGURE 2. EXAMPLE OF CALCULATION OF PAVEMENT THICKNESS DESIGN SHEET FROM 1984 PORTLAND CEMENT ASSOCIATION'S MANUAL.

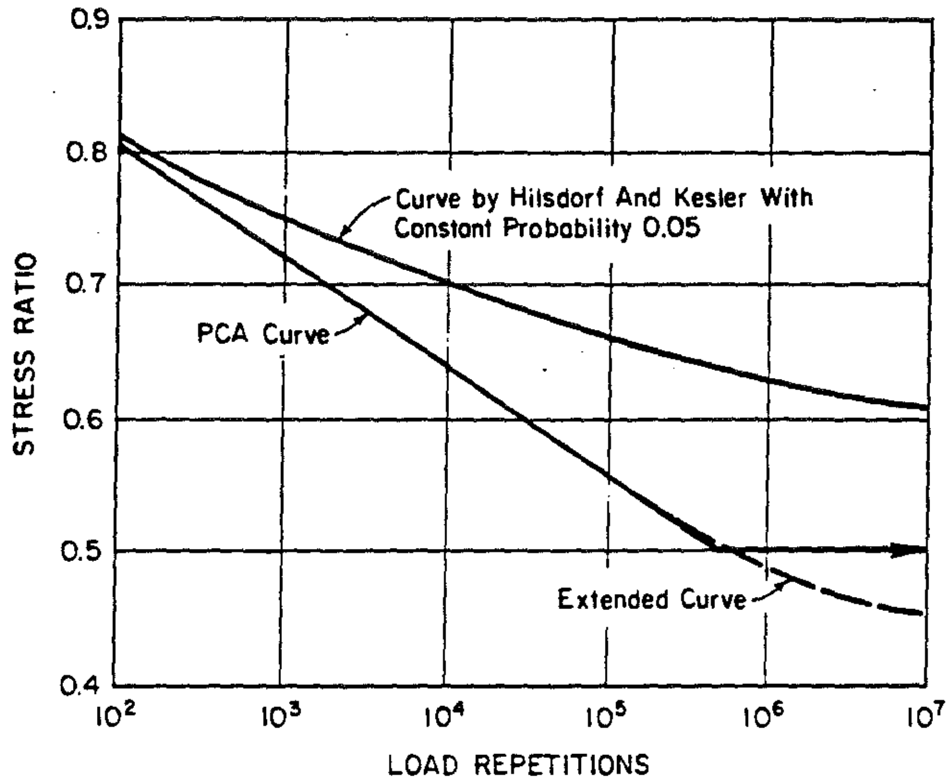


Fig. A3. Fatigue relationships.

FIGURE 3. PORTLAND CEMENT ASSOCIATIONS'S FATIGUE RELATIONSHIPS.

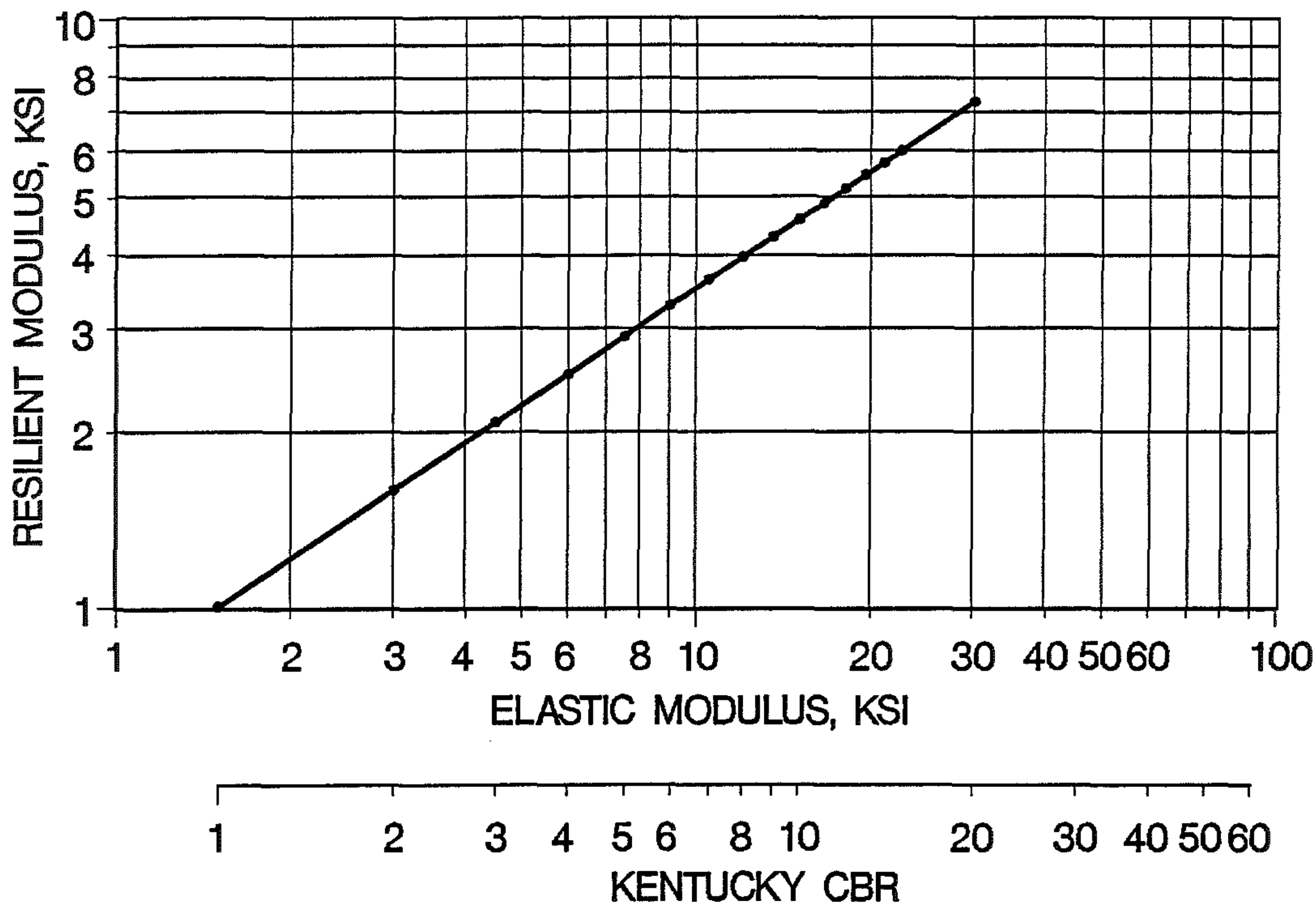


FIGURE 4. RELATIONSHIP BETWEEN KENTUCKY CBR, KENTUCKY SUBGRADE MODULUS AND SUBGRADE RESILIENT MODULUS USED IN 1986 AASHTO AND ACPA DESIGN METHODS.

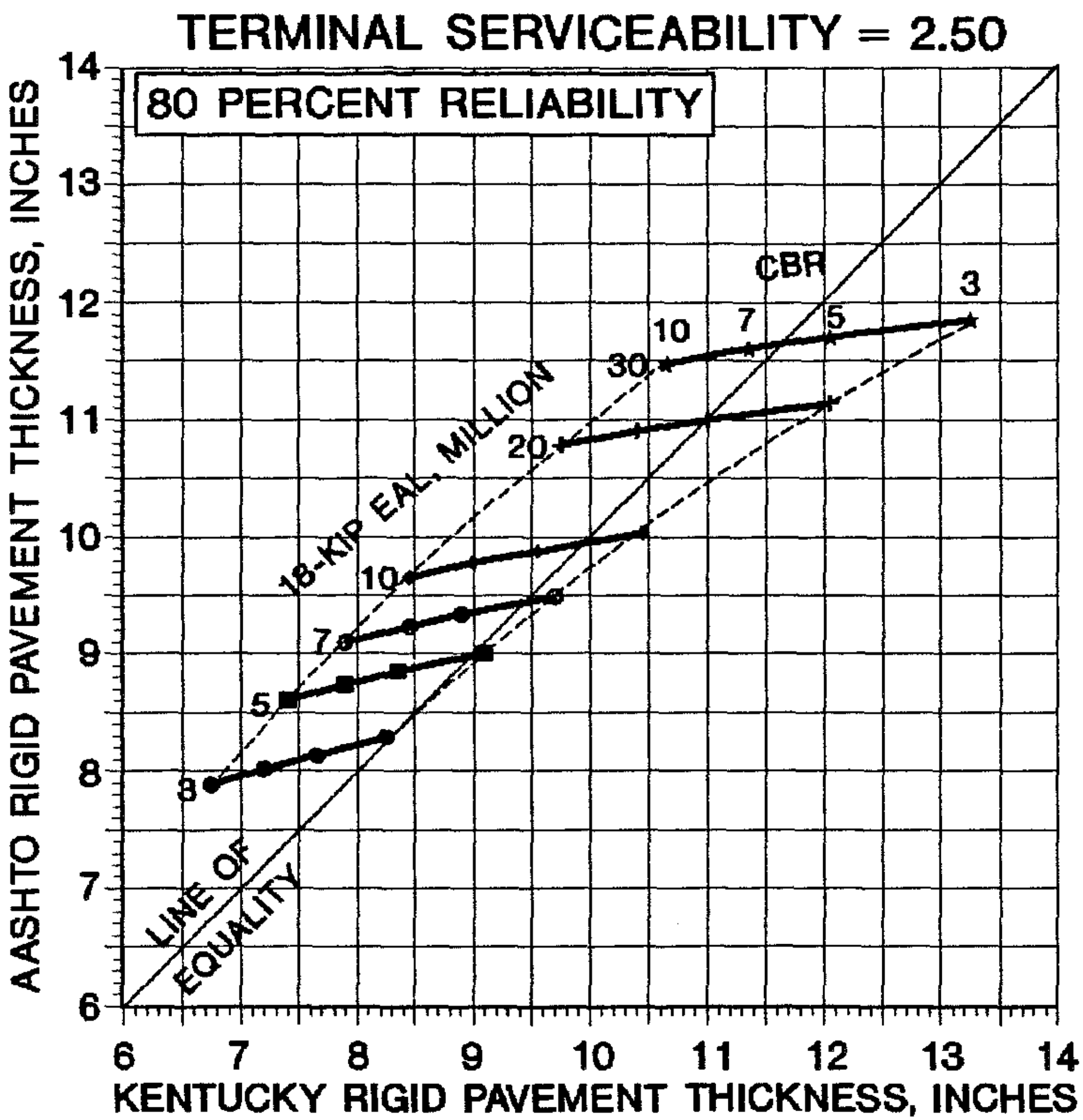


FIGURE 5. COMPARISON OF 1986 AASHTO AND KENTUCKY RIGID PAVEMENT DESIGN THICKNESSES FOR VARIABLE SUBGRADE MODULI AND 18-KIP EAL AT A 2.50 TERMINAL SERVICEABILITY LEVEL AND 80 PERCENT RELIABILITY.

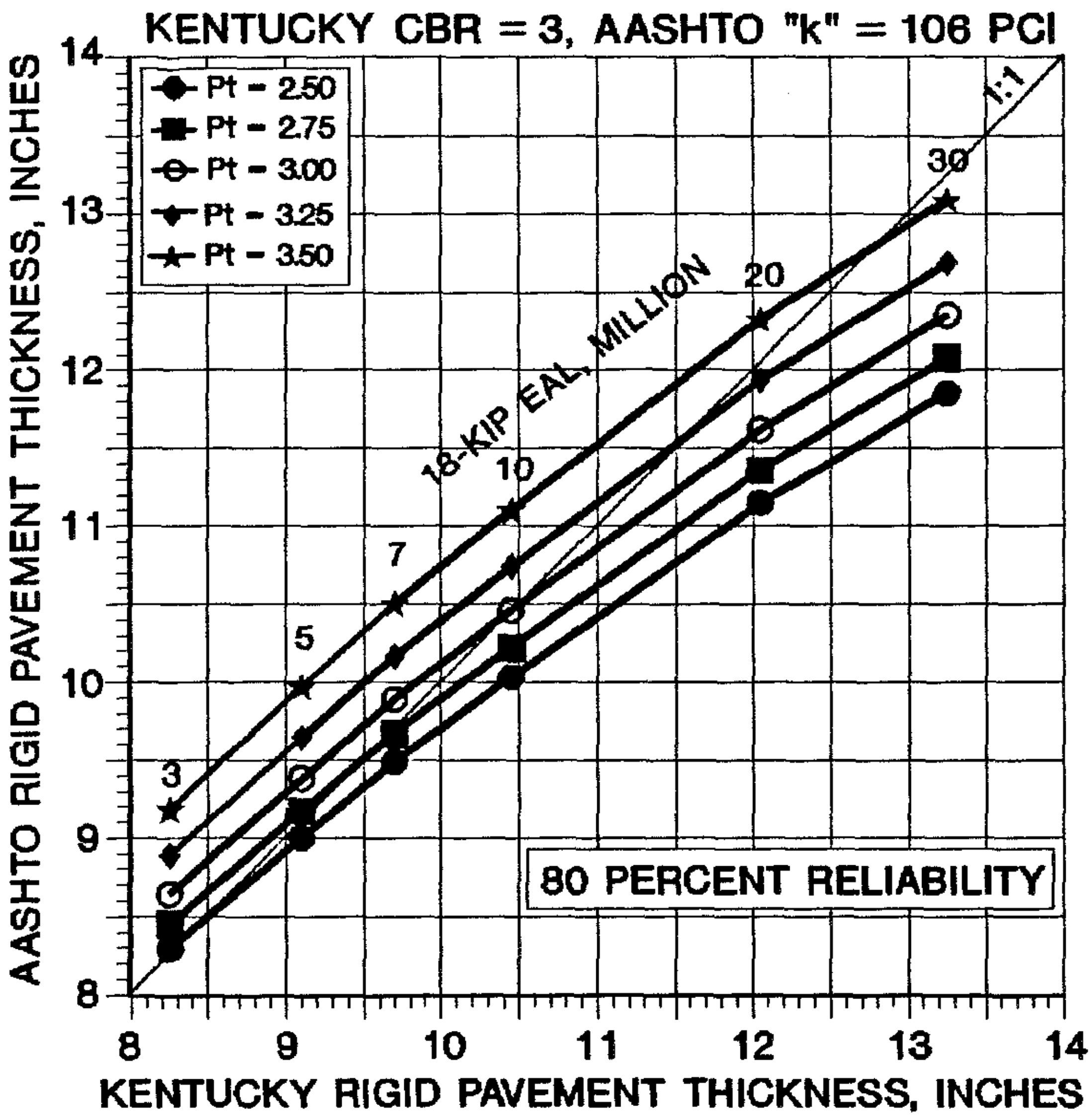


FIGURE 6. COMPARISON OF 1986 AASHTO AND KENTUCKY RIGID PAVEMENT DESIGN THICKNESSES AND VARIABLE 18-KIP EAL LEVELS FOR CBR 3 (AASHTO k = 106 PCI), 80 PERCENT RELIABILITY, AND 5 LEVELS OF TERMINAL SERVICEABILITY.

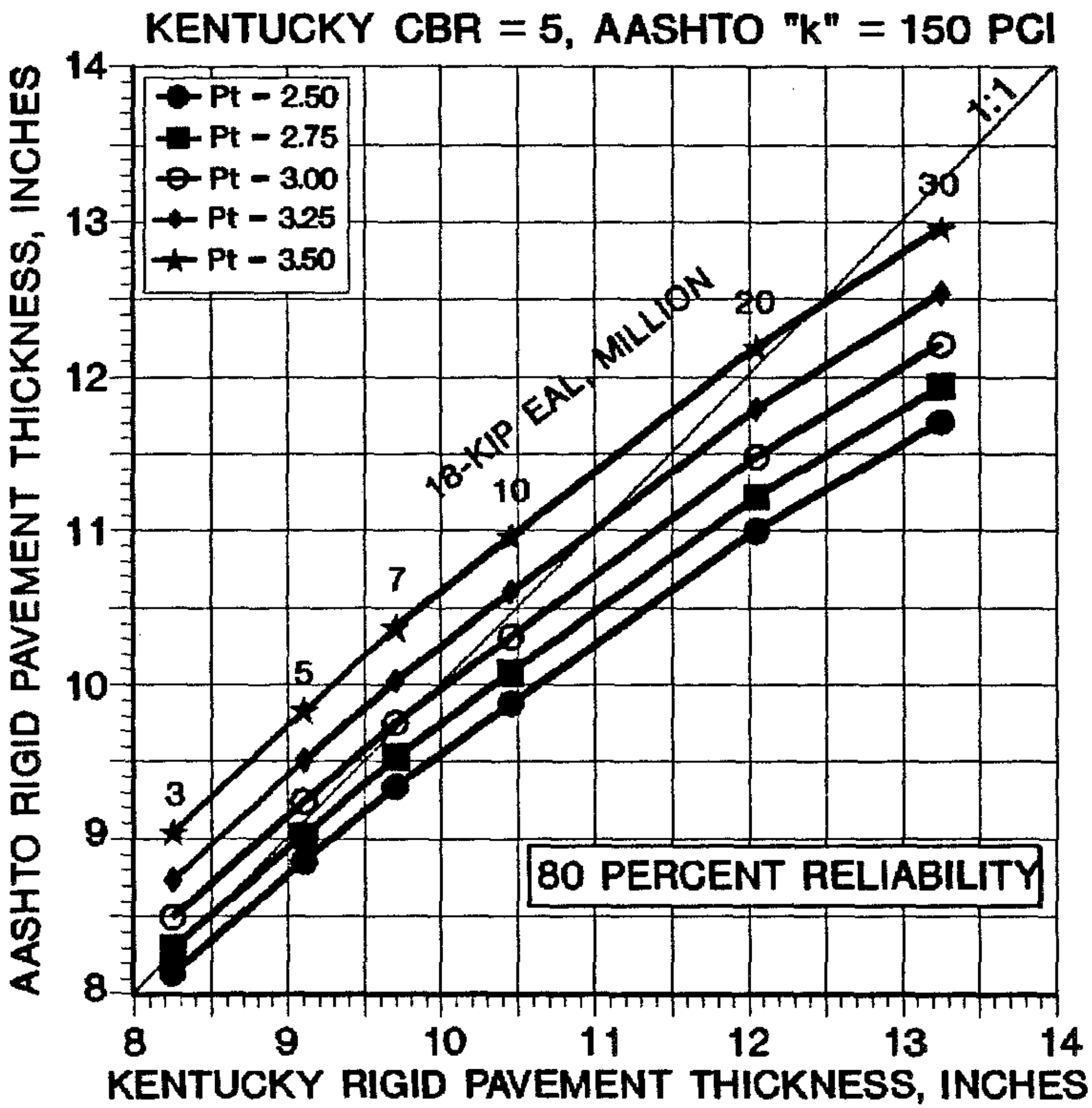


FIGURE 7. COMPARISON OF 1986 AASHTO AND KENTUCKY RIGID PAVEMENT DESIGN THICKNESSES AND VARIABLE 18-KIP EAL LEVELS FOR CBR 5 (AASHTO k = 150 PCI), 80 PERCENT RELIABILITY, AND 5 LEVELS OF TERMINAL SERVICEABILITY.

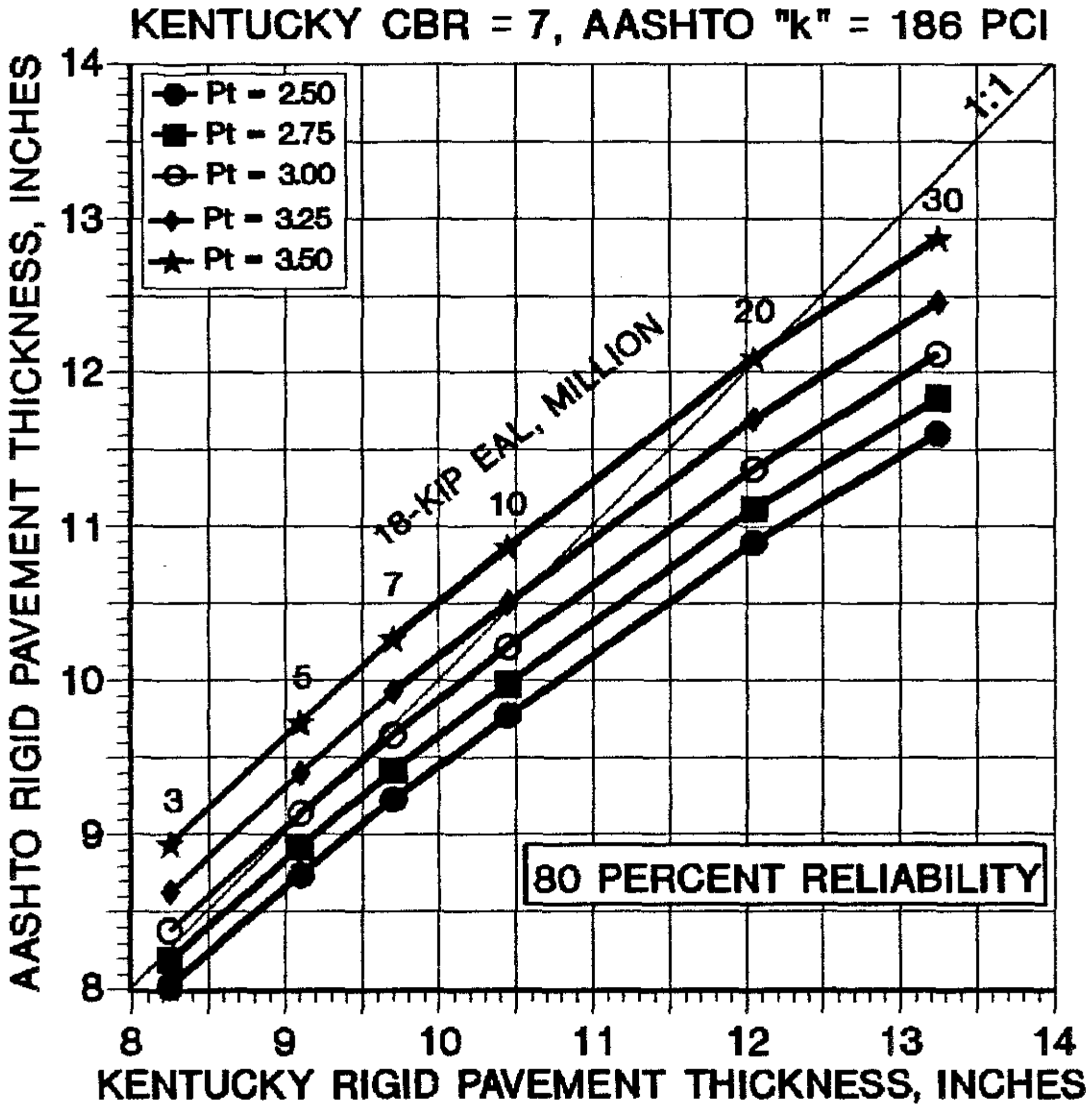


FIGURE 8. COMPARISON OF 1986 AASHTO AND KENTUCKY RIGID PAVEMENT DESIGN THICKNESSES AND VARIABLE 18-KIP EAL LEVELS FOR CBR 7 (AASHTO k = 186 PCI), 80 PERCENT RELIABILITY, AND 5 LEVELS OF TERMINAL SERVICEABILITY.

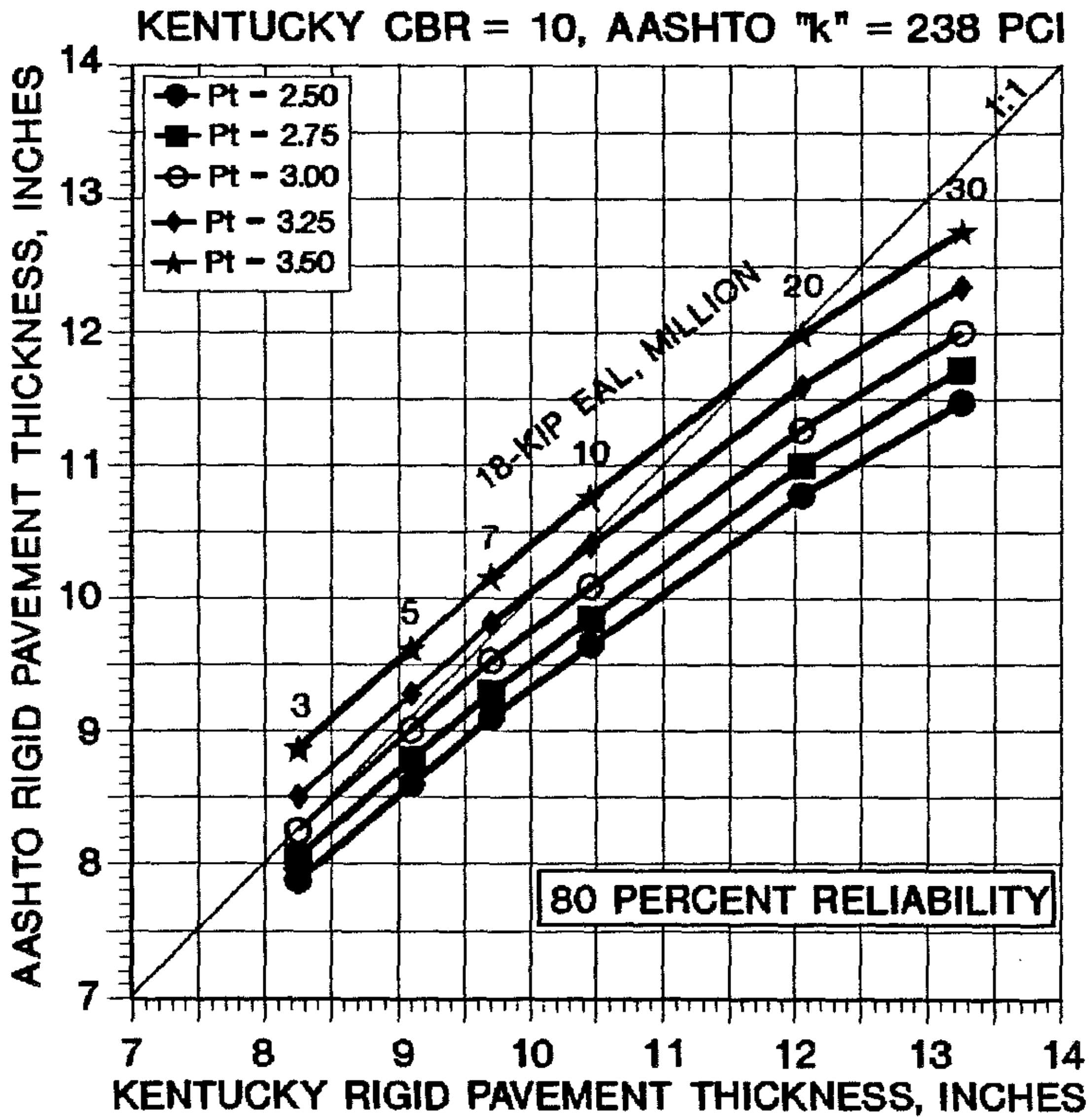


FIGURE 9. COMPARISON OF 1986 AASHTO AND KENTUCKY RIGID PAVEMENT DESIGN THICKNESSES AND VARIABLE 18-KIP EAL LEVELS FOR CBR 10 (AASHTO k = 238 PCI), 80 PERCENT RELIABILITY, AND 5 LEVELS OF TERMINAL SERVICEABILITY.

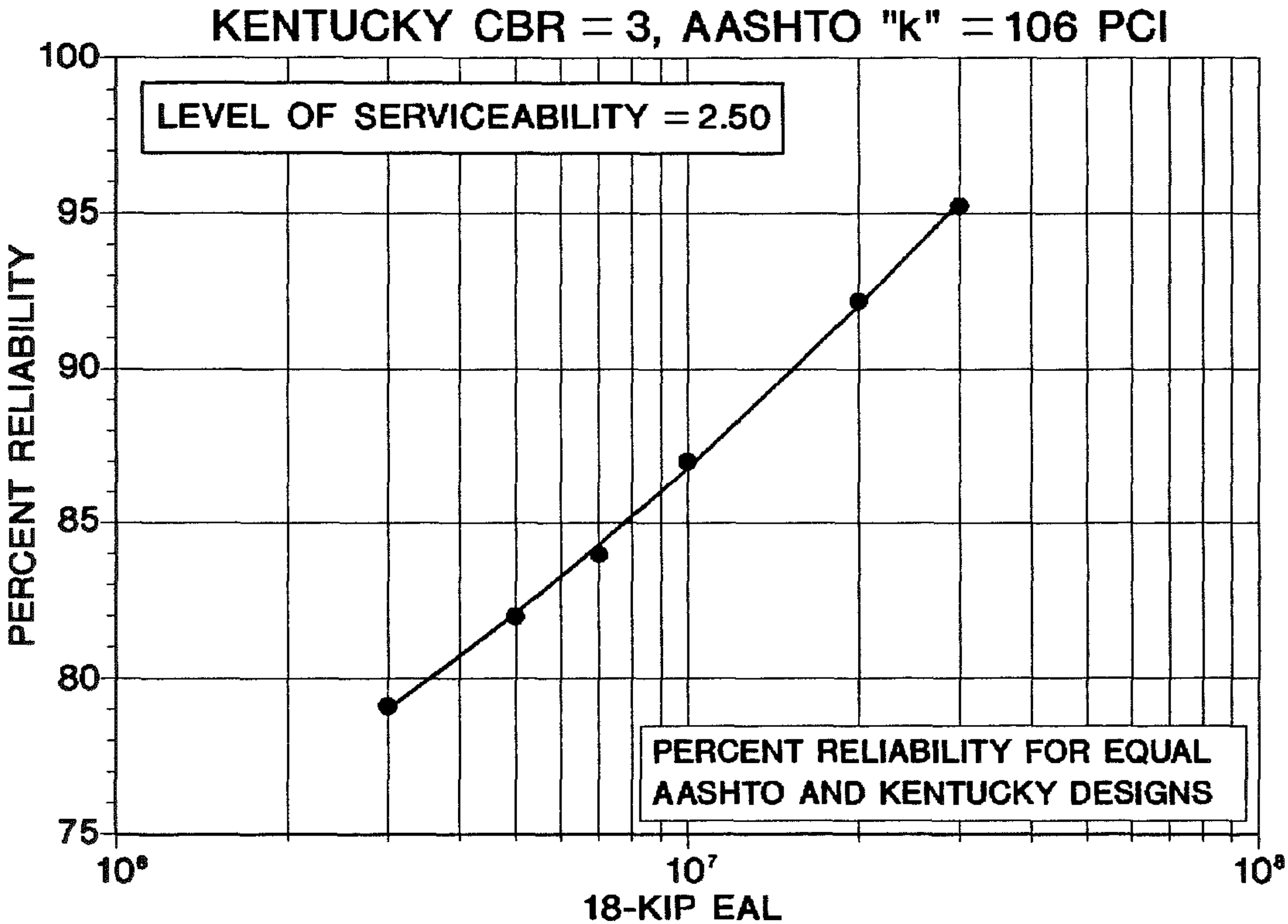
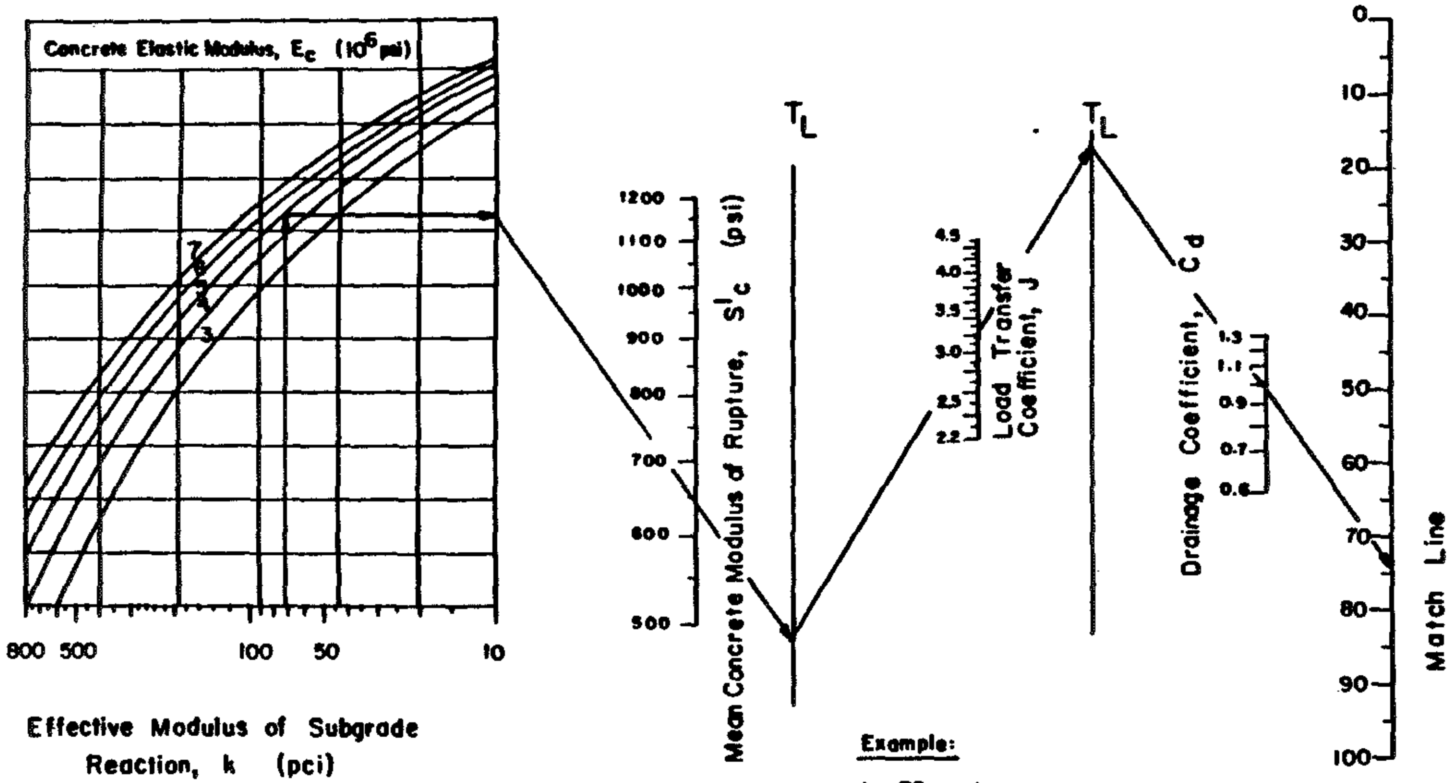


FIGURE 10. RELATIONSHIP OF 18-KIP EAL AND PERCENT RELIABILITY FOR 2.50 SERVICEABILITY LEVEL AND KENTUCKY CBR 3 FOR EQUAL AASHTO AND KENTUCKY THICKNESS DESIGNS.

NOMOGRAPH SOLVES:

$$\log_{10}W_{18} = Z_R{*}S_o + 7.35{*}\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.624{*}10^7}{(D+1)^{8.46}}} + (4.22-0.32p_t){*}\log_{10}\left[\frac{S_c'{*}C_d\left[D^{0.75}-1.132\right]}{215.63{*}J\left[D^{0.75}-\dfrac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$



Example:

k = 72 pci	S_o = 0.29
E_c = 5 x 10^6 psi	R = 95 % (Z_R = -1.645)
S'_c = 650 psi	ΔPSI = 4.2 - 2.5 = 1.7
J = 3.2	W_{18} = 5.1 x 10^6 (18 kip ESAL)
C_d = 1.0	Solution: D=10.0 inches (nearest half-inch, from segment 2)

Figure 3.7. Design chart for rigid pavement based on using mean values for each input variable (Segment 1).

FIGURE 11. SEGMENT 1 OF 1986 AASHTO RIGID PAVEMENT THICKNESS DESIGN NOMOGRAPH.

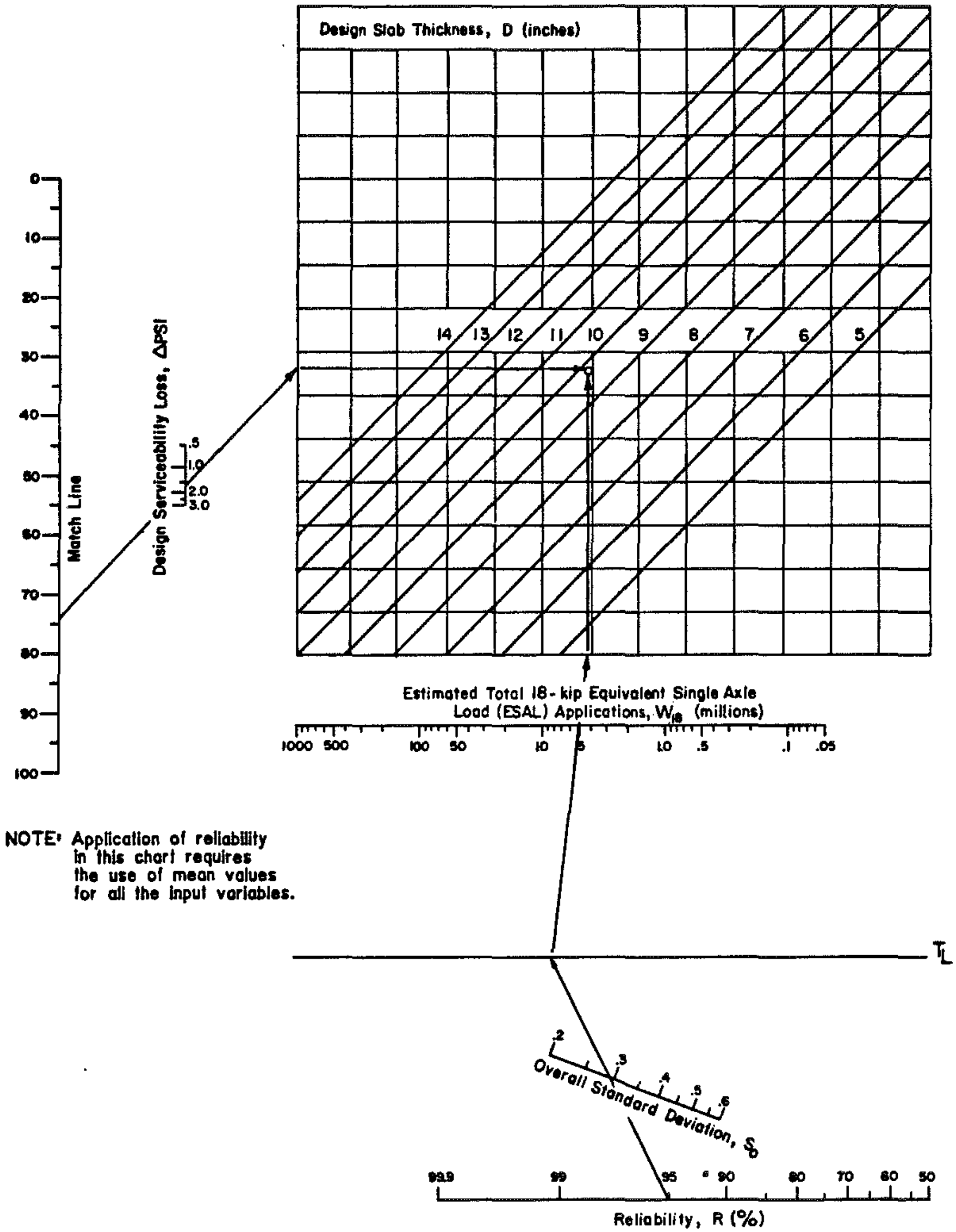


Figure 3.7. Design chart for rigid pavements based on using mean values for each input variable (Segment 2).

FIGURE 12. SEGMENT 2 OF 1986 AASHTO RIGID PAVEMENT THICKNESS DESIGN NOMOGRAPH.

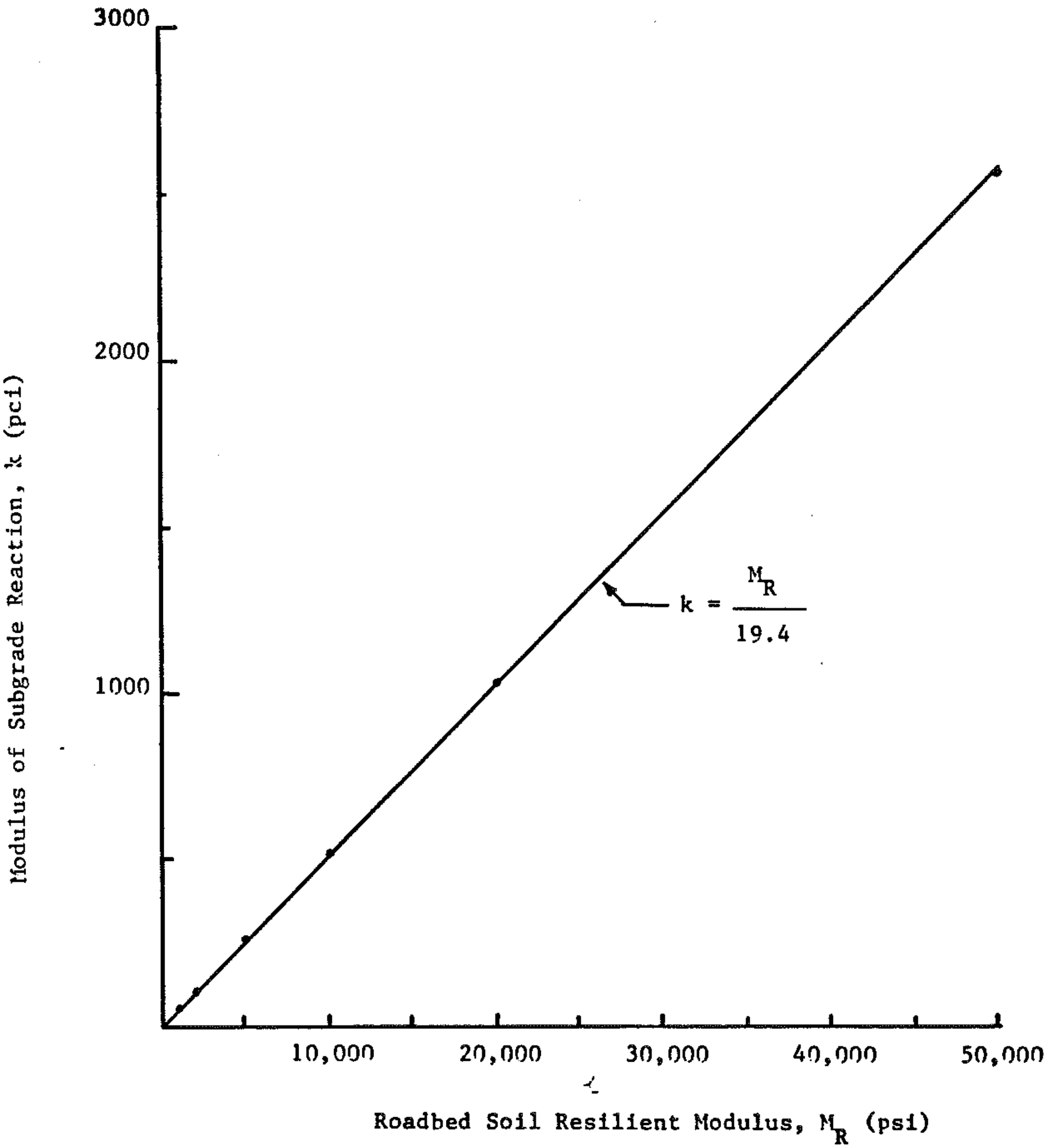


FIGURE 13. THEORETICAL RELATIONSHIP BETWEEN MODULUS OF SUBGRADE REACTION AND ROADBED SOIL RESILIENT MODULUS.

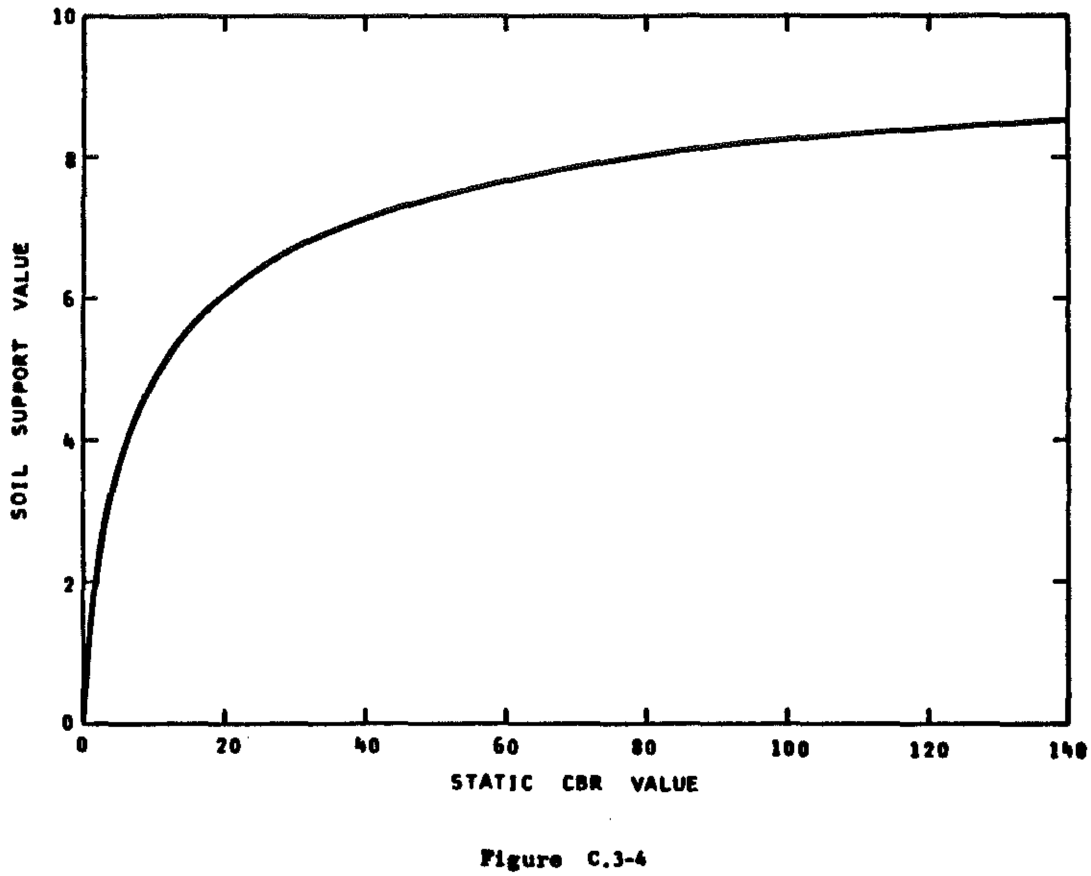


FIGURE 14. RELATIONSHIP BETWEEN SOIL SUPPORT VALUE AND STATIC CBR VALUE, FIGURE C.3-4, 1972 AASHTO INTERIM GUIDE FOR DESIGN OF PAVEMENT STRUCTURES.

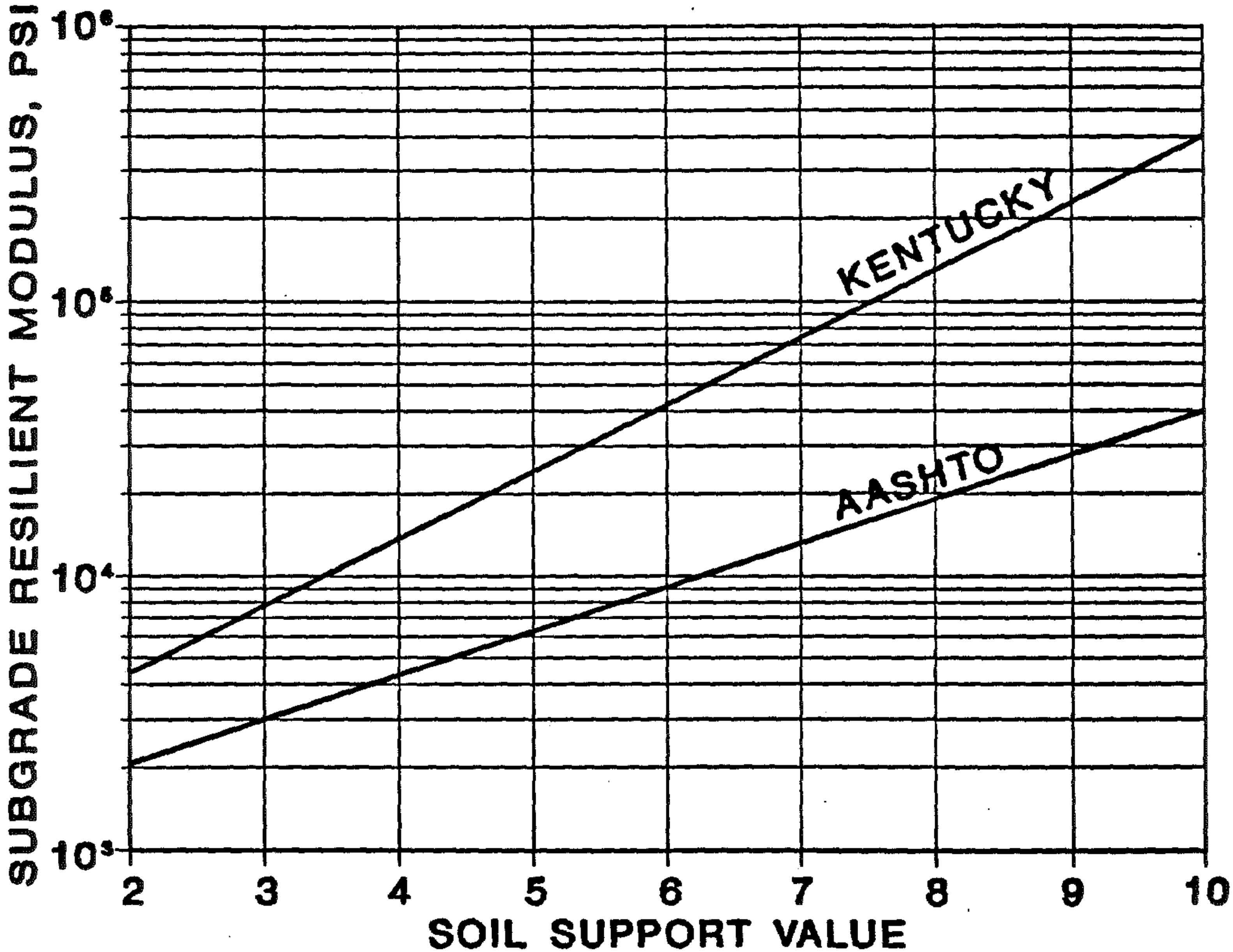


FIGURE 15. SUBGRADE RESILIENT MODULI VERSUS SOIL SUPPORT VALUE FOR KENTUCKY AND AASHTO RELATIONSHIPS.

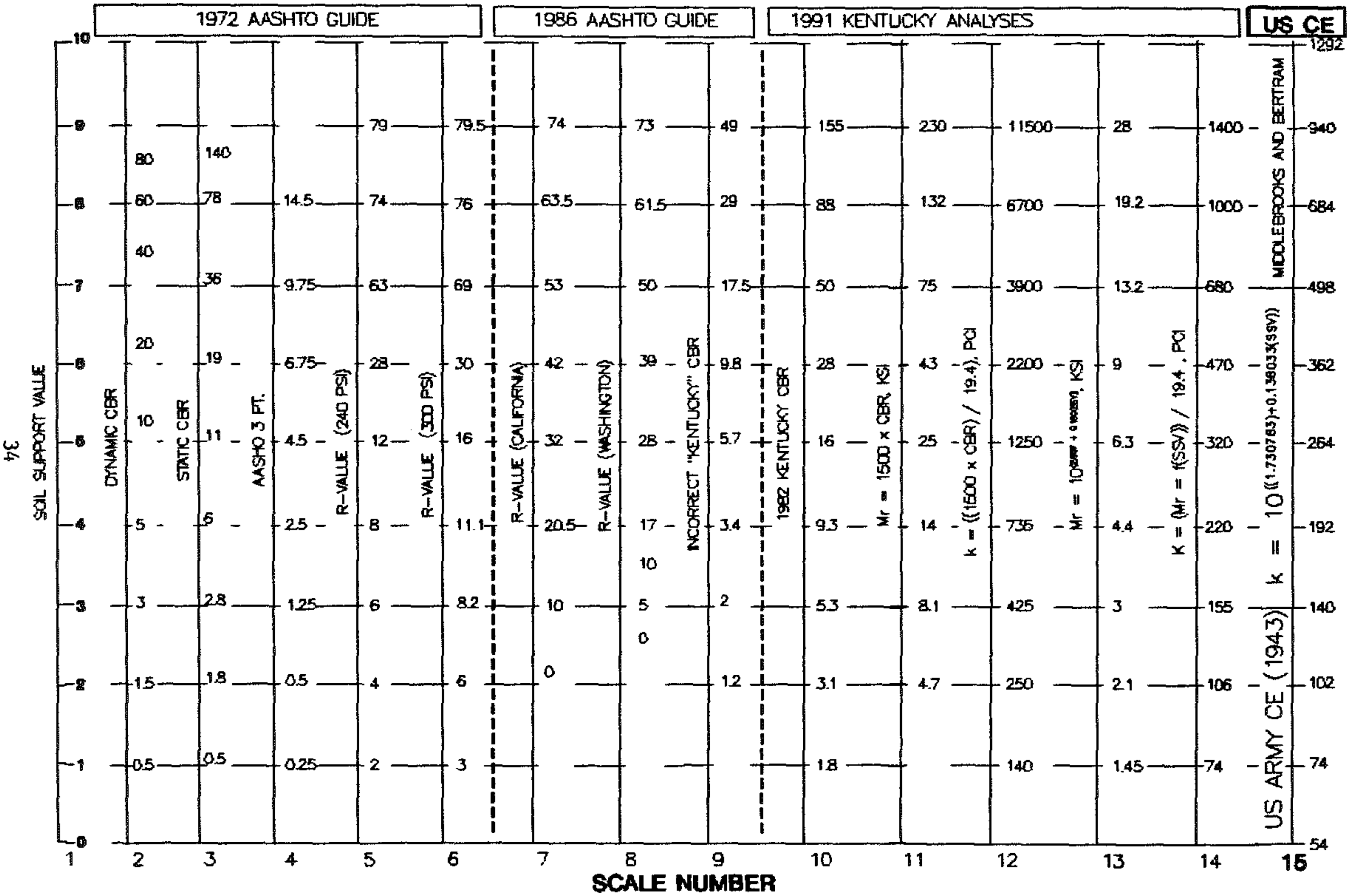


FIGURE 16. NOMOGRAPH RELATING AASHTO SOIL SUPPORT VALUE, CBR, R-VALUE, AASHTO 3 PT., RESILIENT MODULUS, AND MODULUS OF SUBGRADE REACTION.

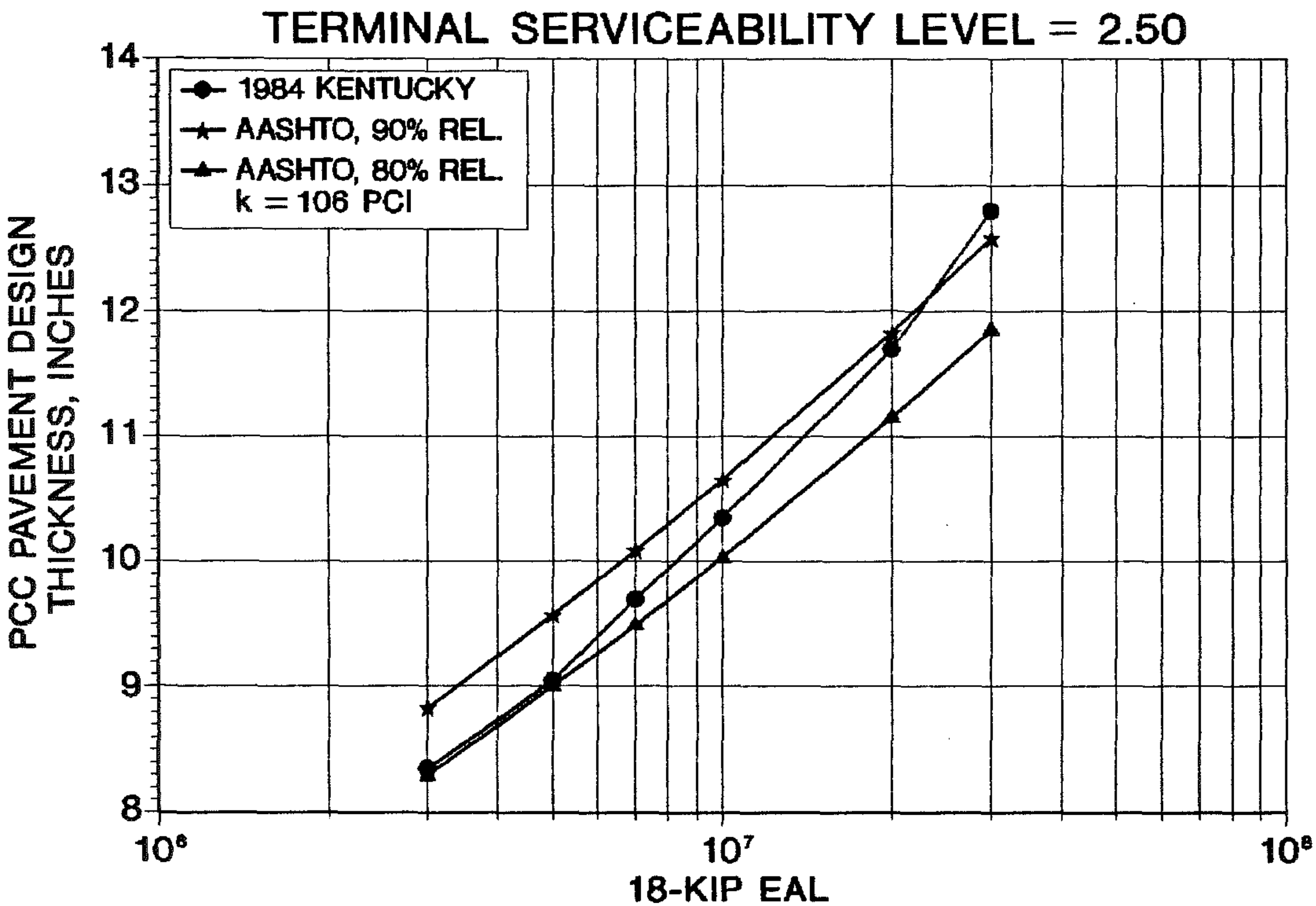


FIGURE 17. COMPARISON OF KENTUCKY AND 1986 AASHTO PCC PAVEMENT THICKNESS DESIGNS FOR A SUBGRADE MODULUS OF 3 KSI AND VARYING 18-KIP EALS AT A TERMINAL SERVICEABILITY OF 2.50.

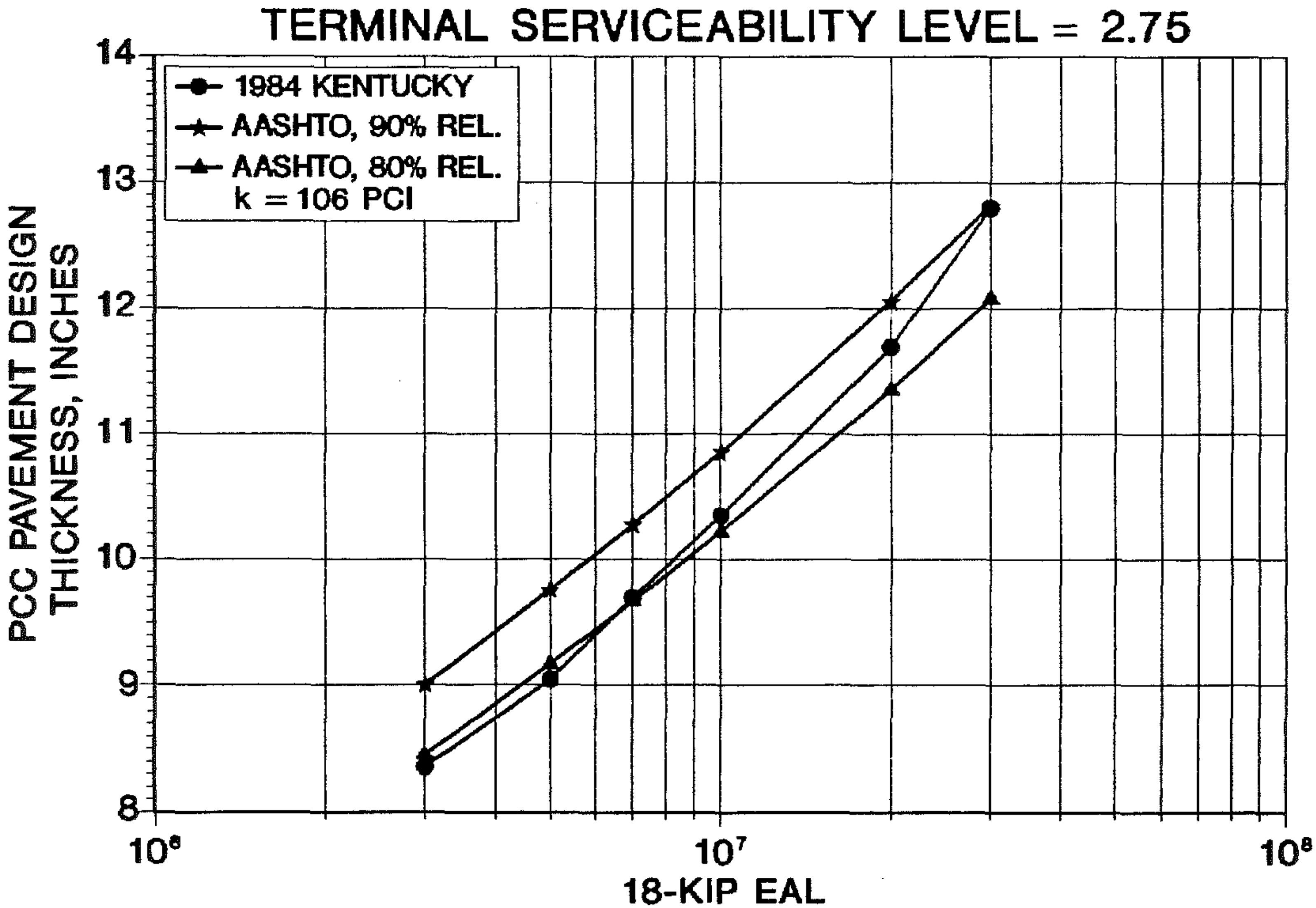


FIGURE 18. COMPARISON OF KENTUCKY AND 1986 AASHTO PCC PAVEMENT THICKNESS DESIGNS FOR A SUBGRADE MODULUS OF 3 KSI AND VARYING 18-KIP EALS AT A TERMINAL SERVICEABILITY OF 2.75.

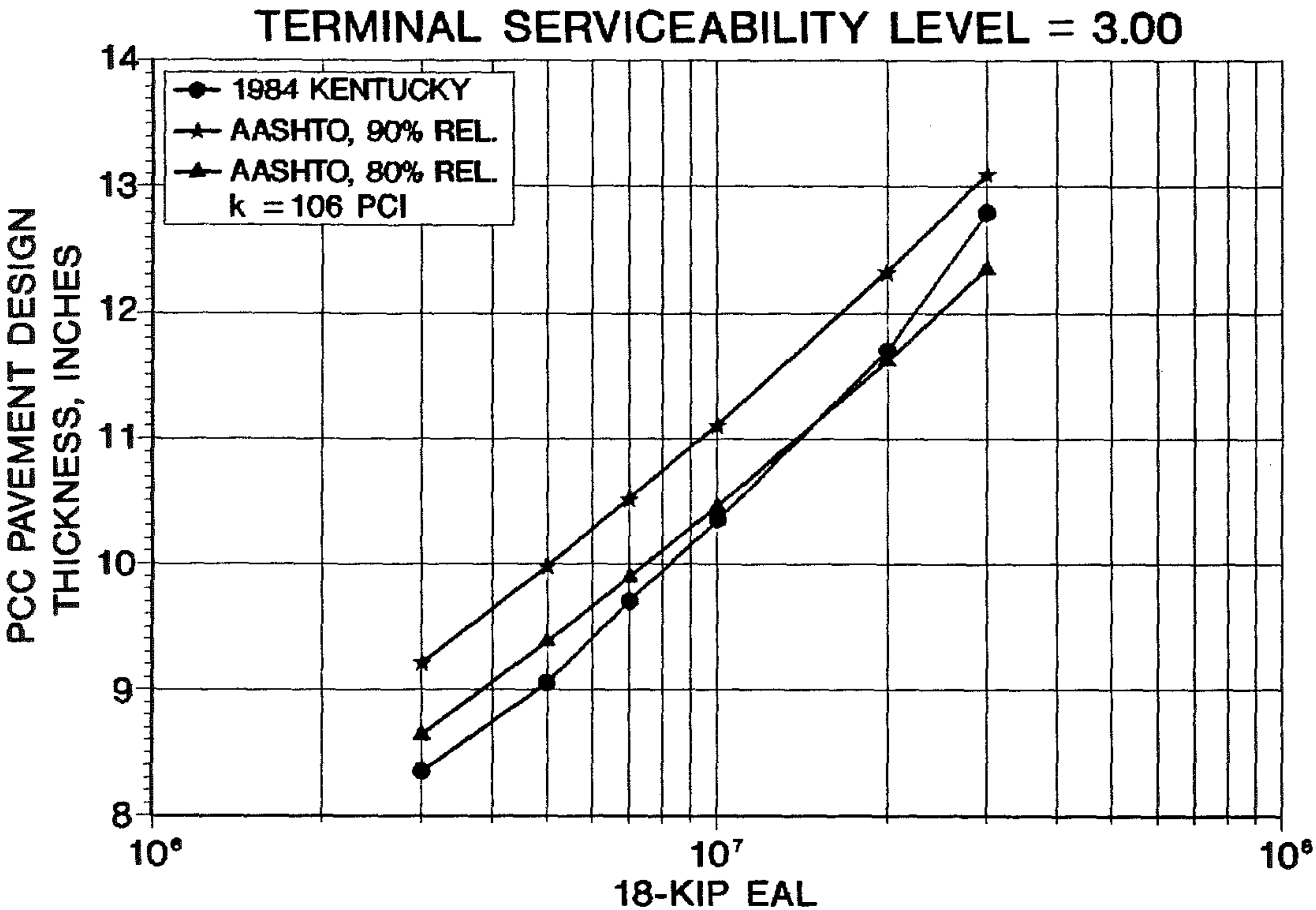


FIGURE 19. COMPARISON OF KENTUCKY AND 1986 AASHTO PCC PAVEMENT THICKNESS DESIGNS FOR A SUBGRADE MODULUS OF 3 KSI AND VARYING 18-KIP EALS AT A TERMINAL SERVICEABILITY OF 3.00.

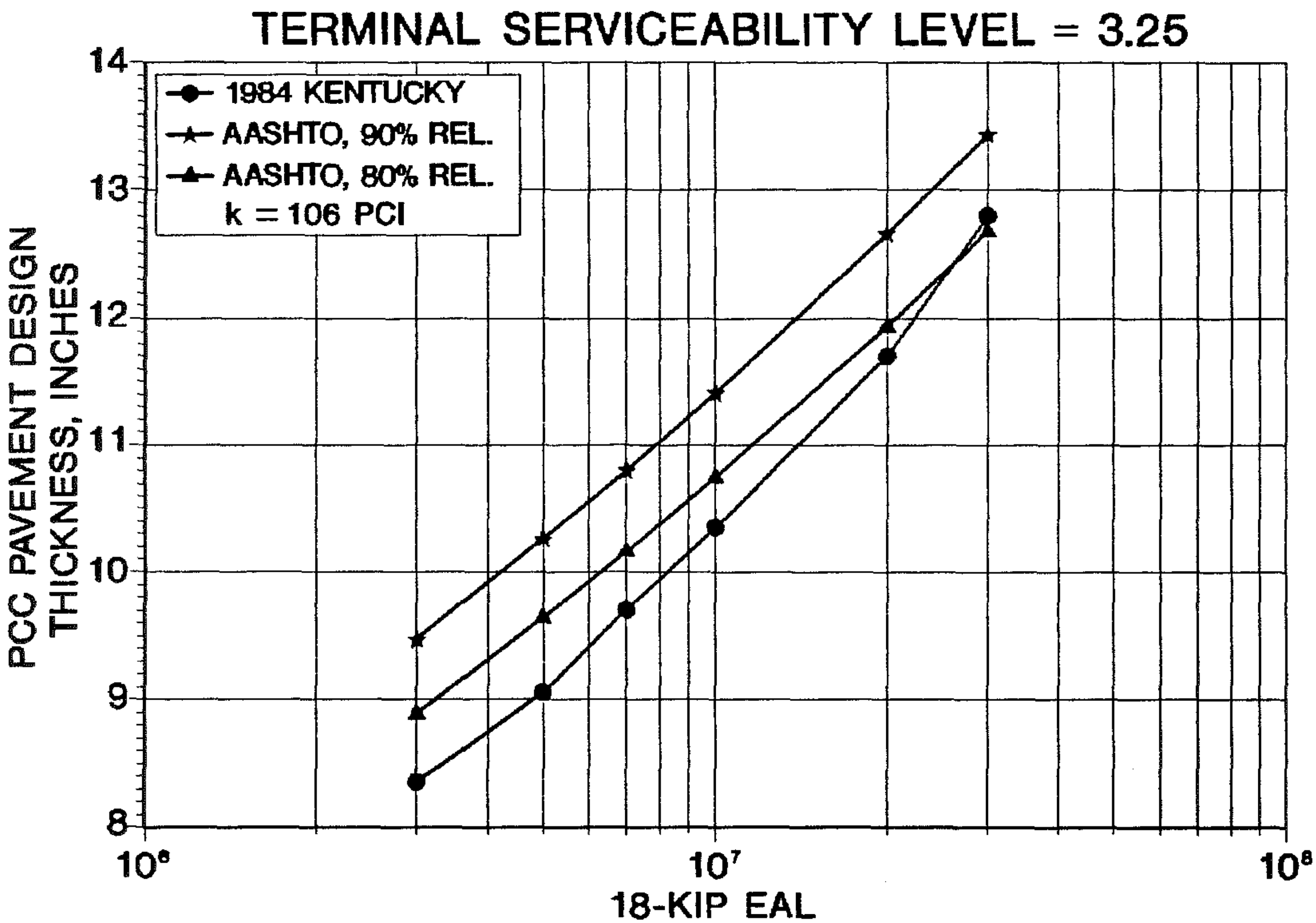


FIGURE 20. COMPARISON OF KENTUCKY AND 1986 AASHTO PCC PAVEMENT THICKNESS DESIGNS FOR A SUBGRADE MODULUS OF 3 KSI AND VARYING 18-KIP EALS AT A TERMINAL SERVICEABILITY OF 3.25.

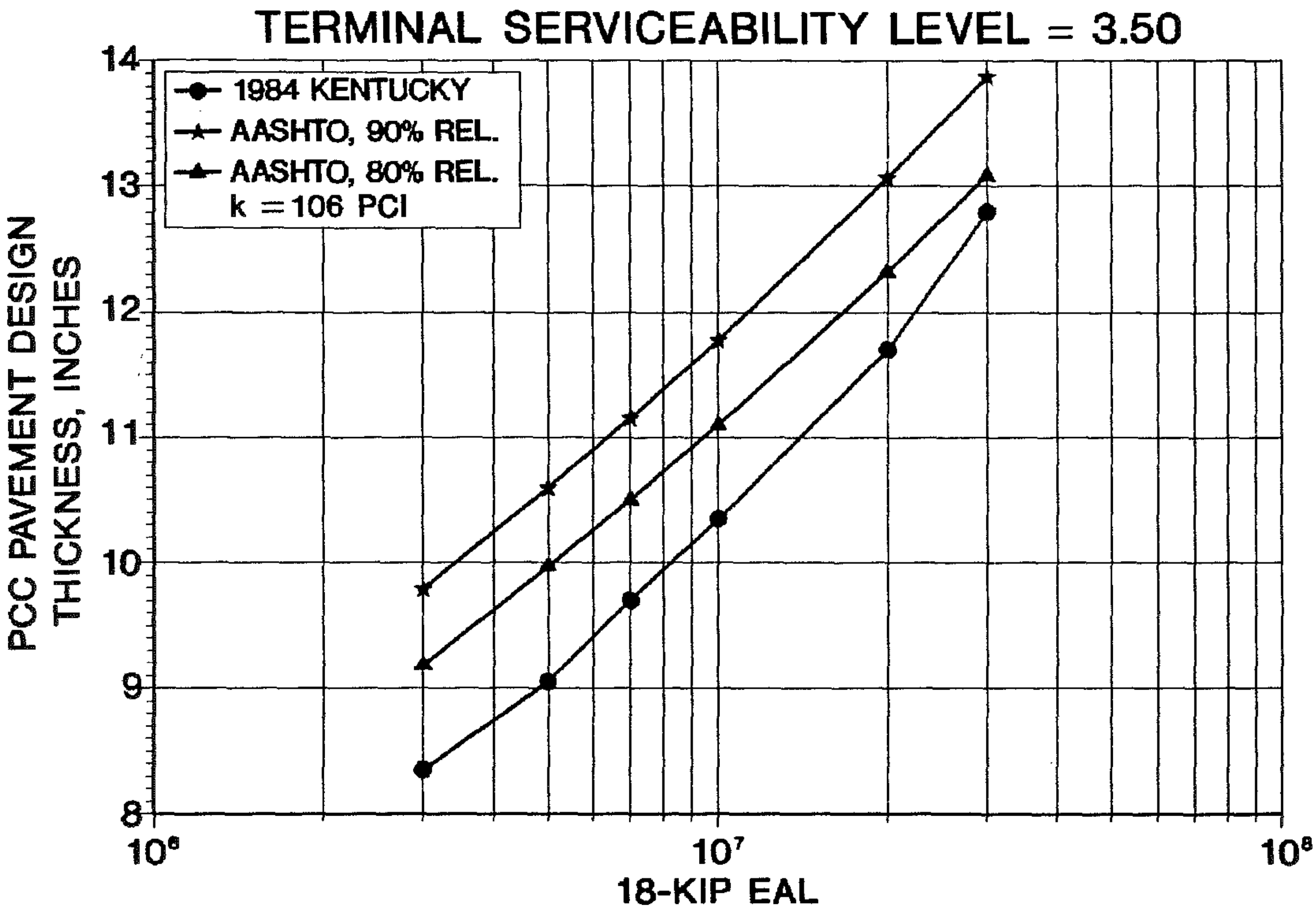


FIGURE 21. COMPARISON OF KENTUCKY AND 1986 AASHTO PCC PAVEMENT THICKNESS DESIGNS FOR A SUBGRADE MODULUS OF 3 KSI AND VARYING 18-KIP EALS AT A TERMINAL SERVICEABILITY OF 3.50.

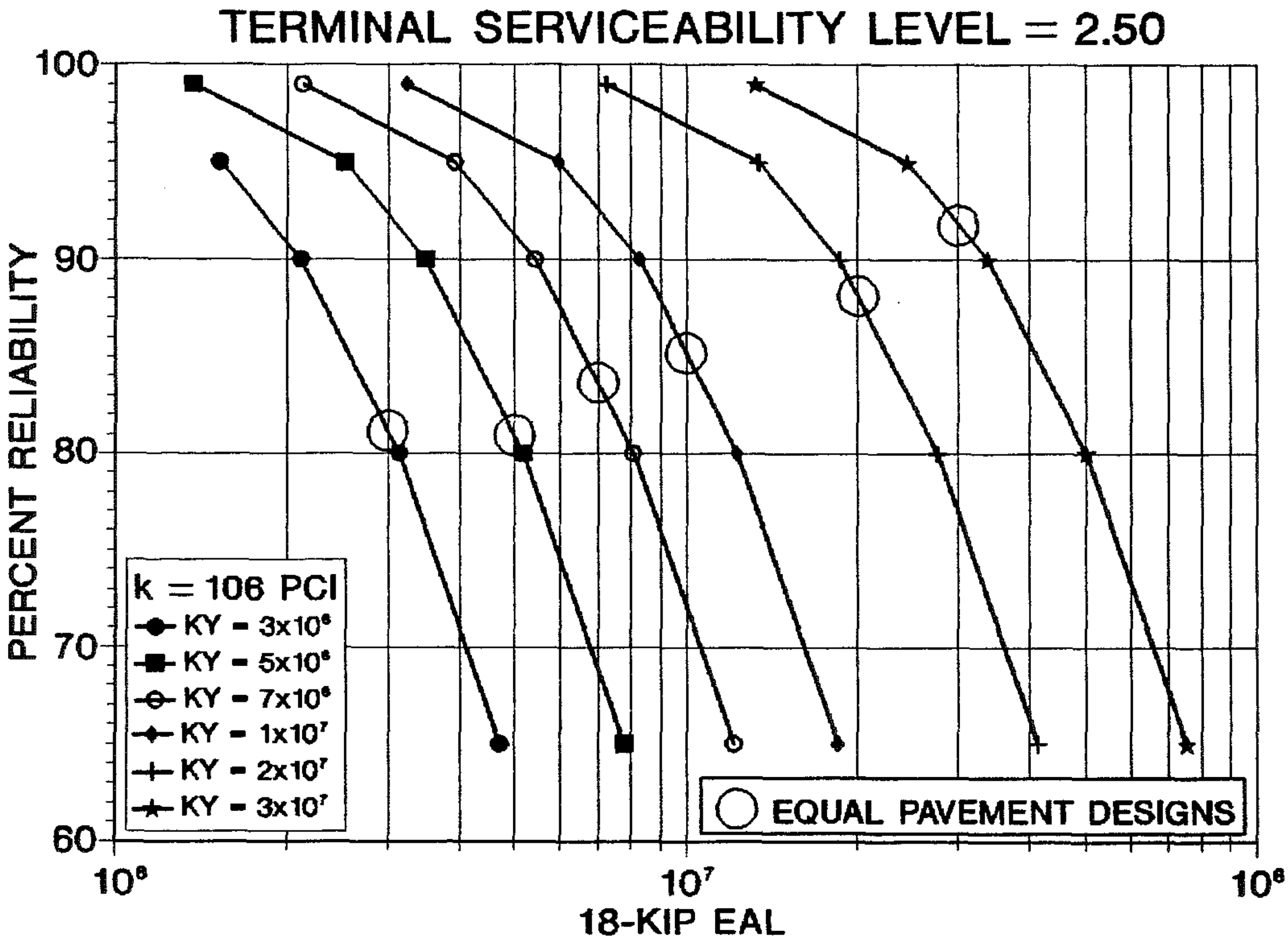


FIGURE 22. RELATIONSHIP OF 18-KIP EALS WITH PERCENT RELIABILITY FOR EQUAL PAVEMENT THICKNESSES AT A 2.50 LEVEL OF SERVICEABILITY AND k = 106 PCI.

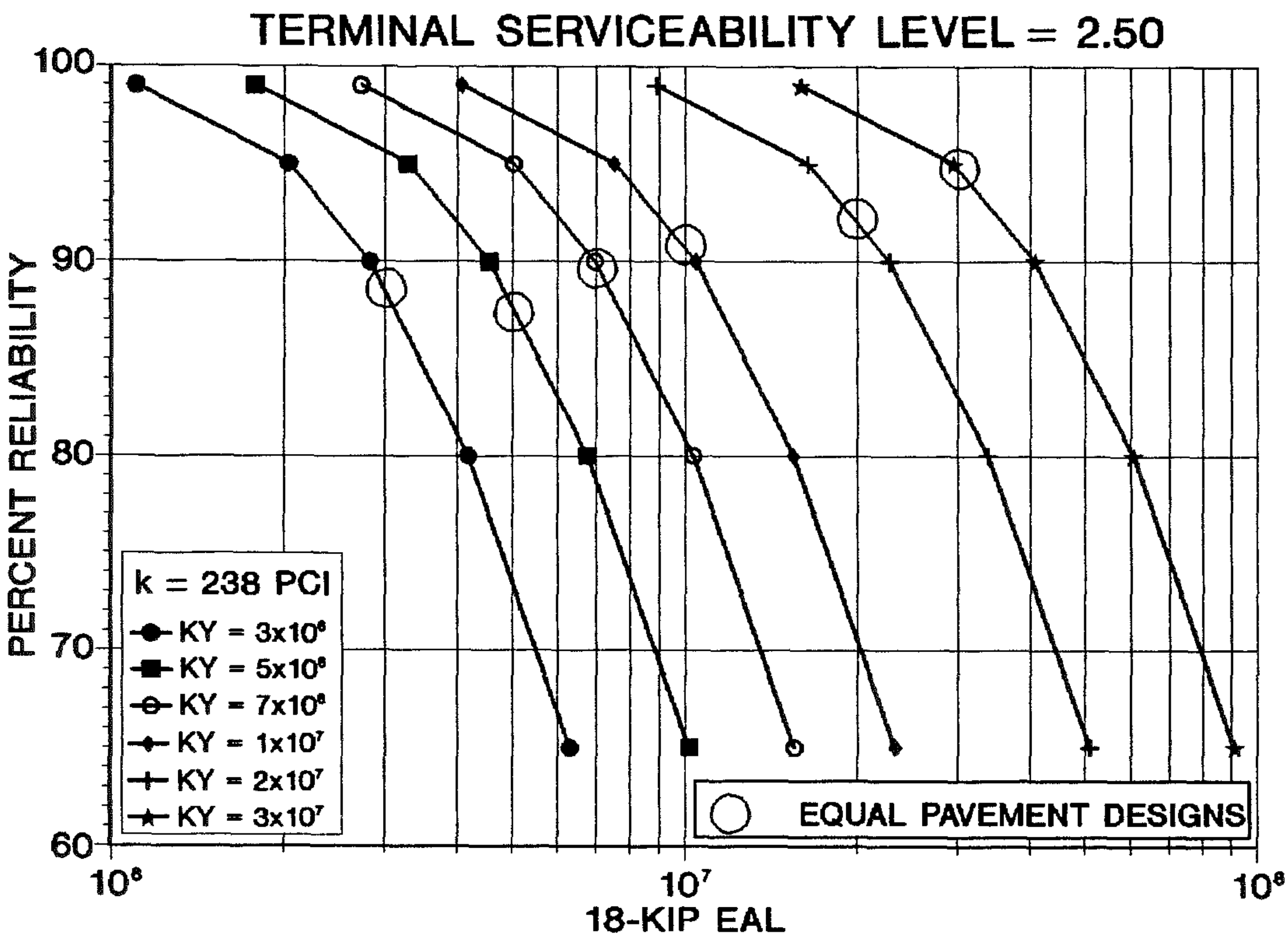


FIGURE 23. RELATIONSHIP OF 18-KIP EALS WITH PERCENT RELIABILITY FOR EQUAL PAVEMENT THICKNESSES AT A 2.50 LEVEL OF SERVICEABILITY AND k = 238 PCI.

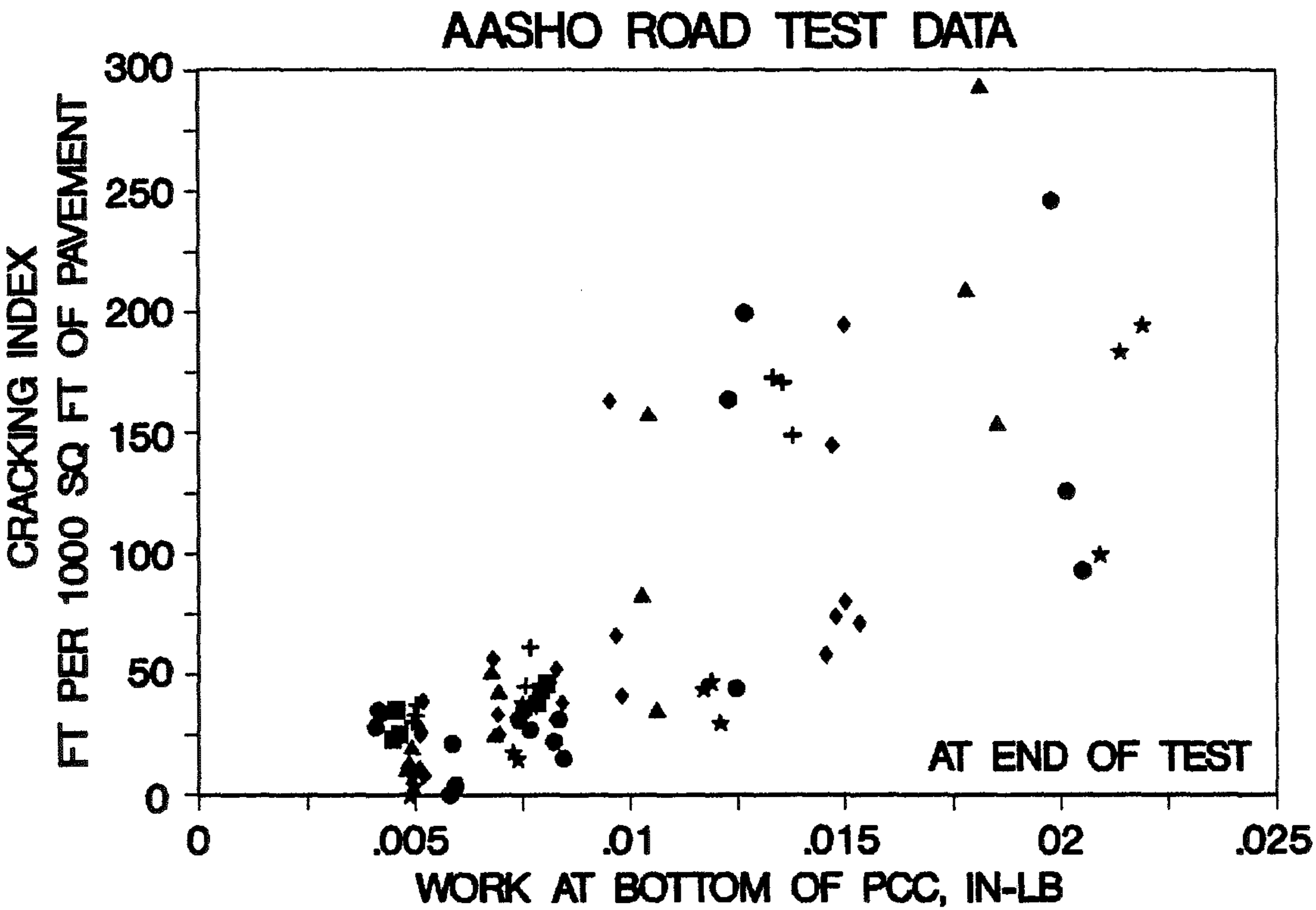


FIGURE 24. AASHO ROAD TEST CRACKING INDEX DATA VERSUS WORK AT BOTTOM OF PCC PAVEMENT (TABLE 50, REFERENCE 12).

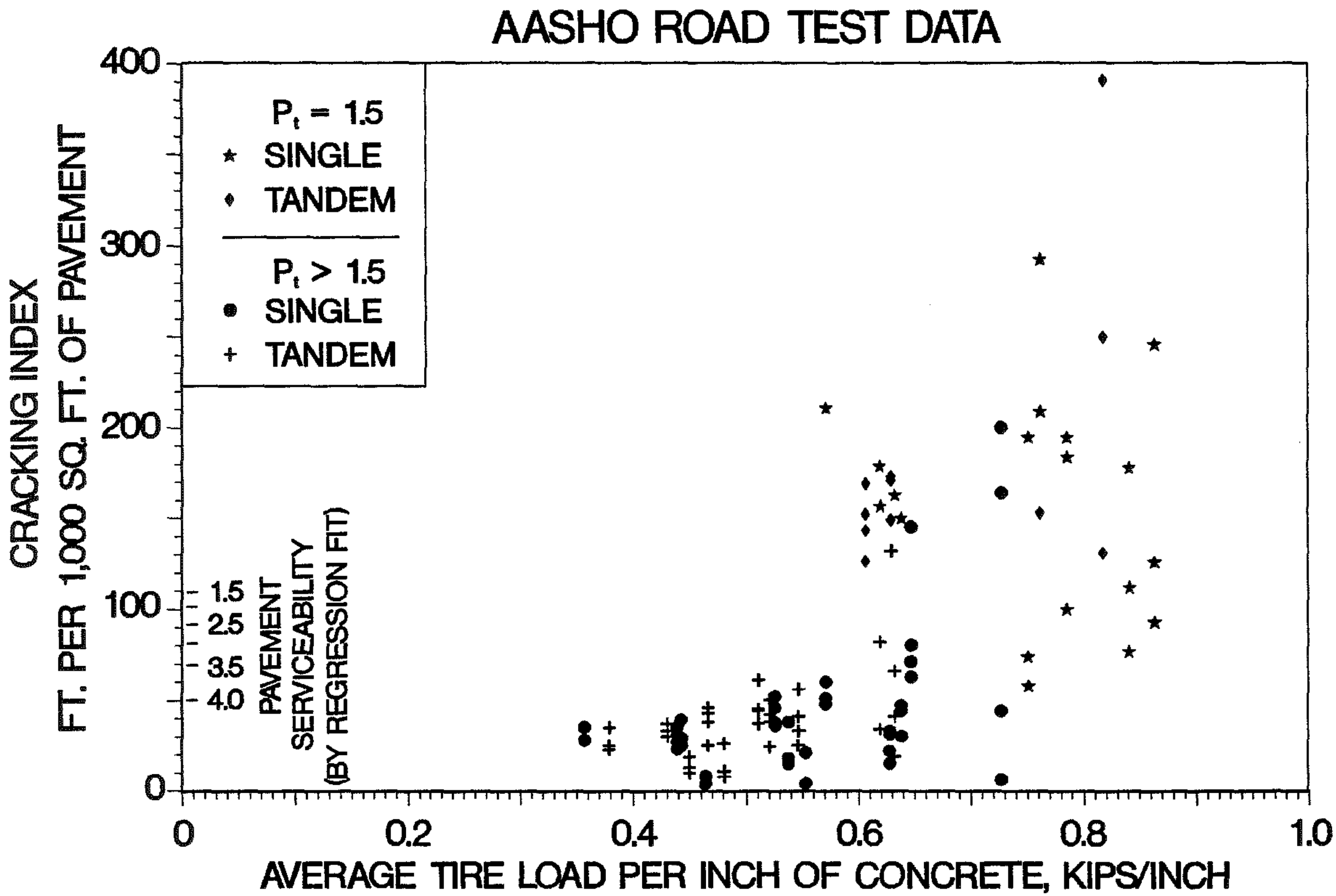


FIGURE 25. AASHO ROAD TEST CRACKING INDEX DATA VERSUS AVERAGE TIRE LOAD PER INCH OF CONCRETE (TABLE 50, REFERENCE 12).

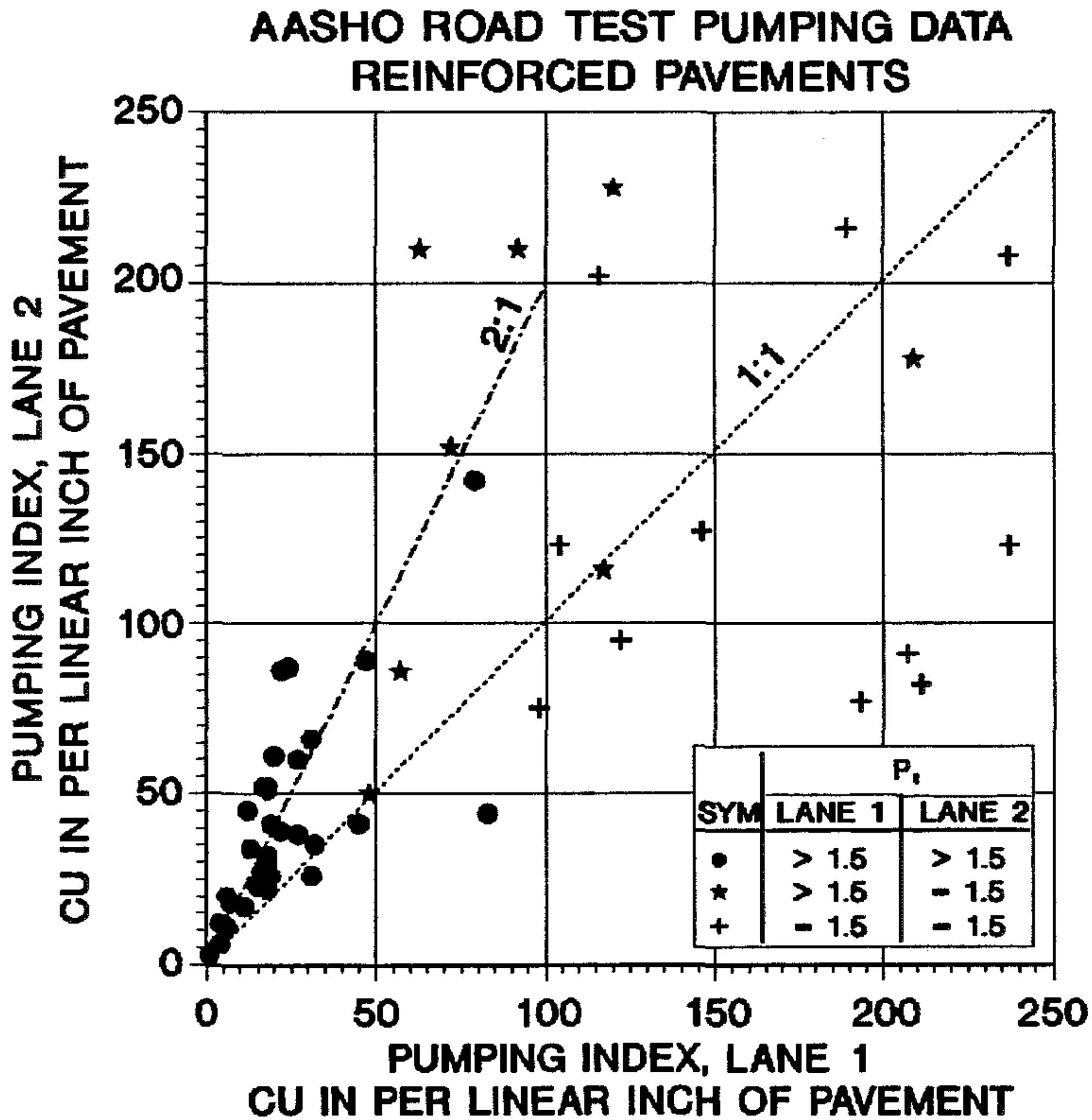


FIGURE 26. AASHO ROAD TEST PUMPING INDEX DATA, LANE 2 (TANDEM AXLES) VERSUS LANE 1 (SINGLE AXLES) (TABLE 54, REFERENCE 12).

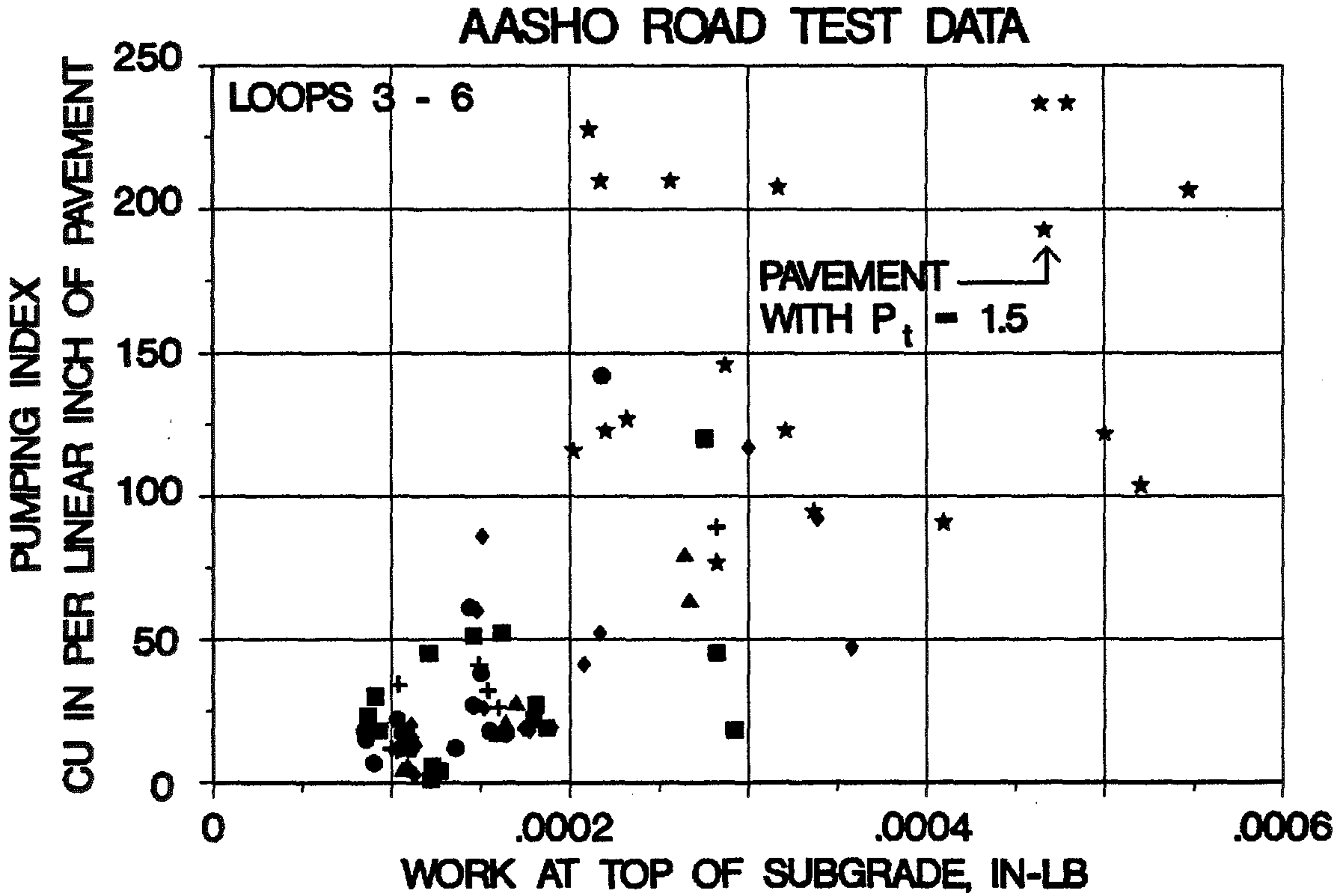


FIGURE 27. AASHO ROAD TEST PUMPING INDEX DATA VERSUS WORK AT TOP OF SUBGRADE (TABLE 54, REFERENCE 12).

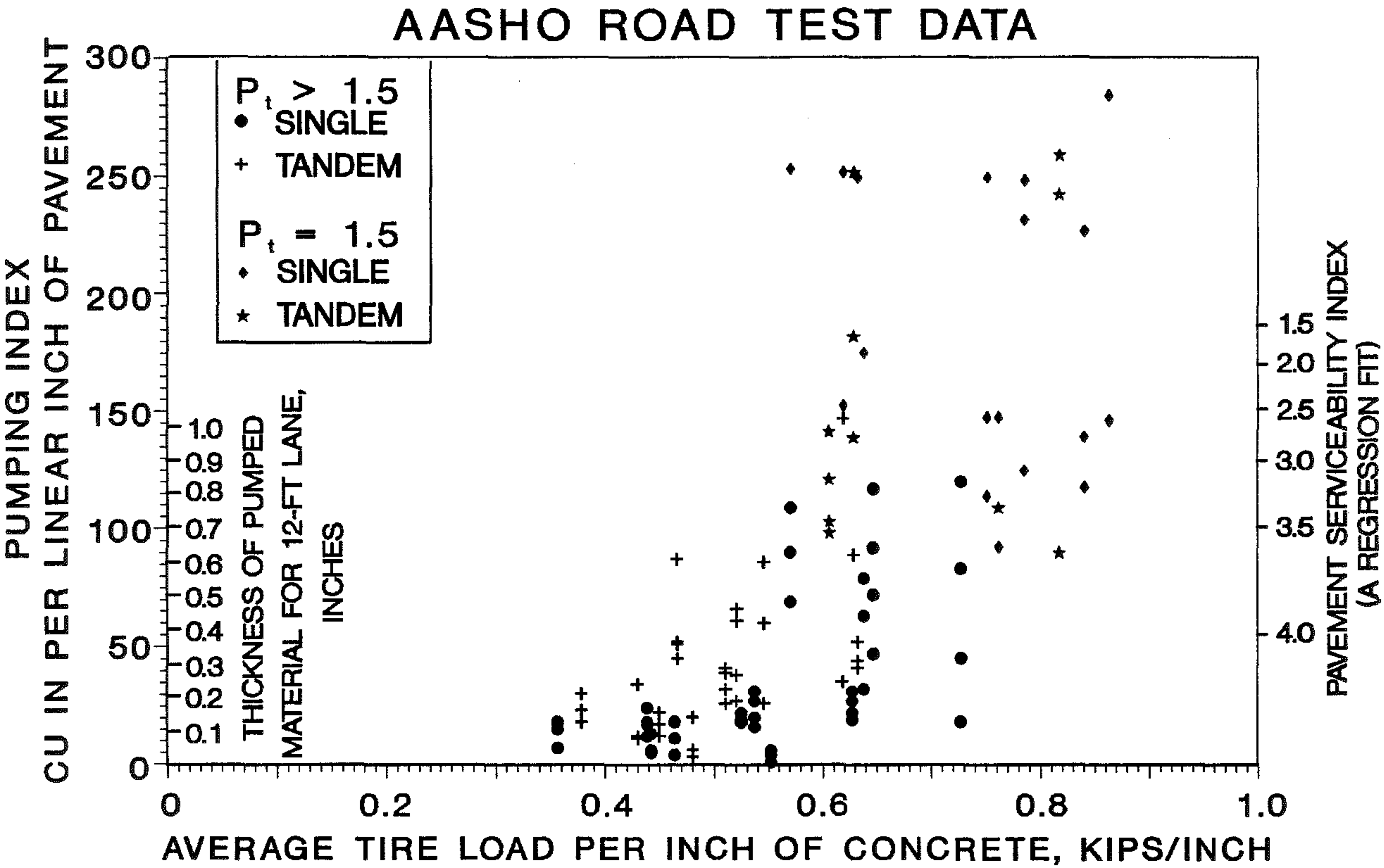


FIGURE 28. AASHO ROAD TEST PUMPING INDEX DATA VERSUS AVERAGE TIRE LOAD PER INCH OF CONCRETE (TABLE 54, REFERENCE 12).

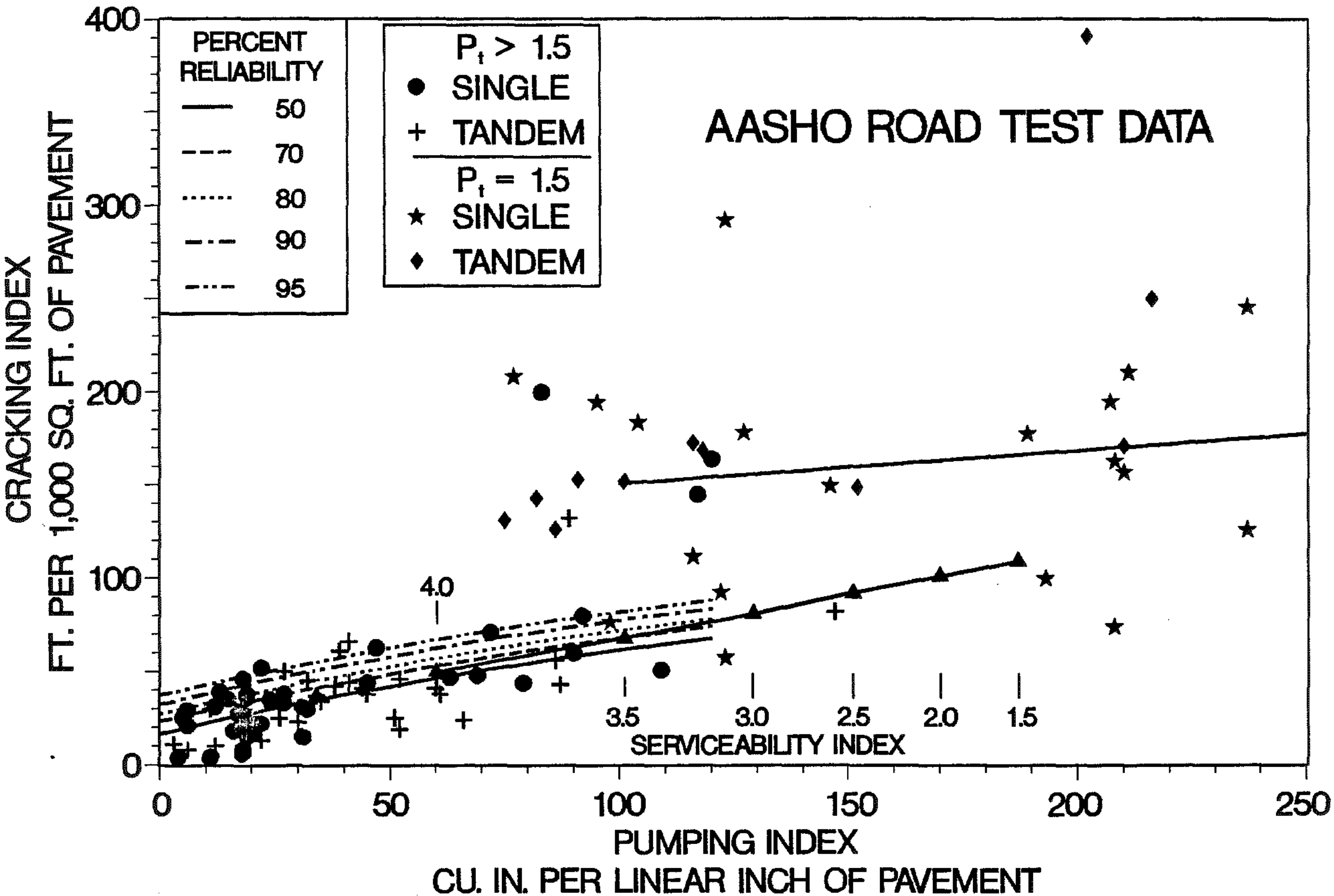


FIGURE 29. AASHO ROAD TEST DATA, CRACKING INDEX VERSUS PUMPING INDEX.

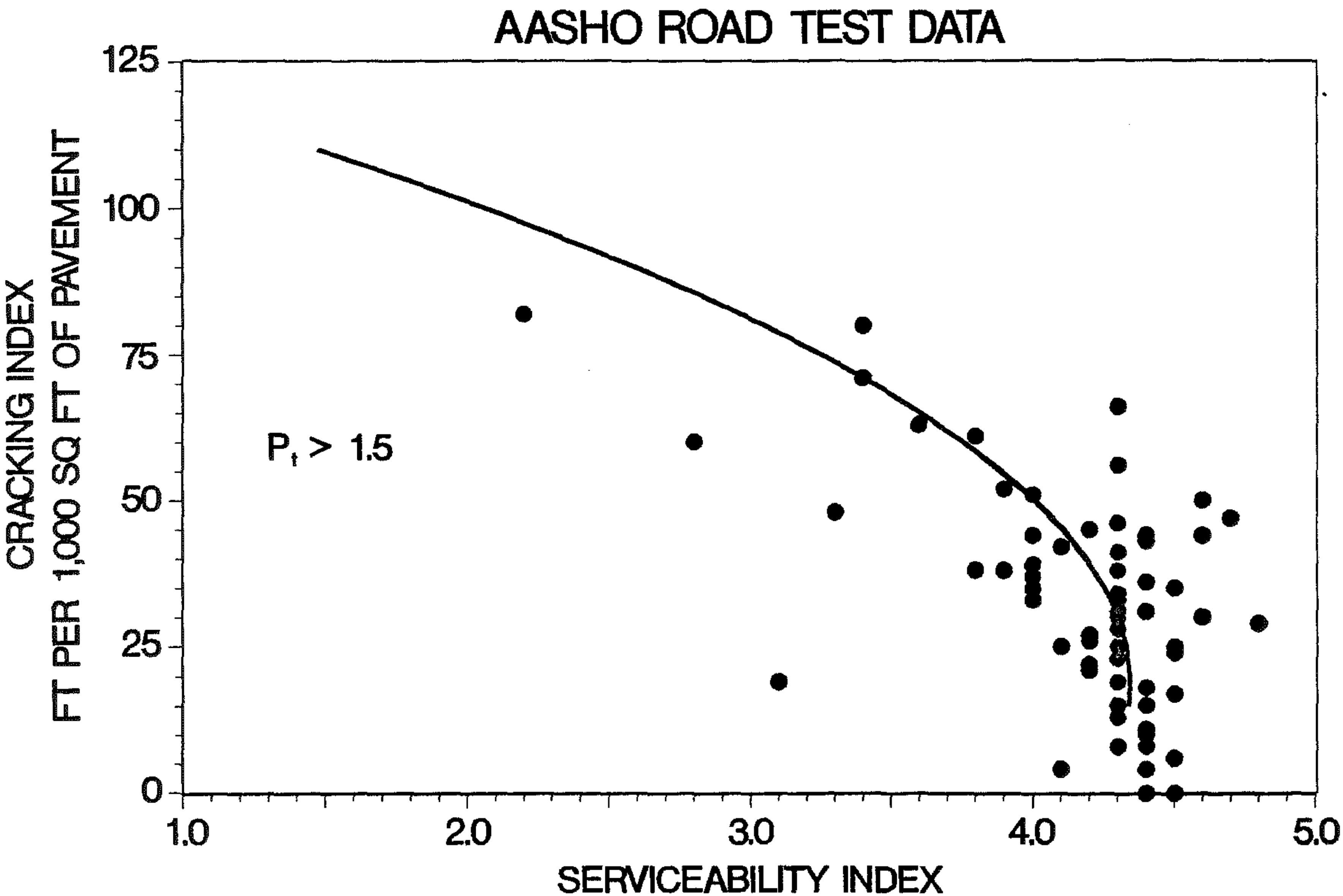


FIGURE 30. AASHO ROAD TEST DATA, CRACKING INDEX VERSUS SERVICEABILITY INDEX.

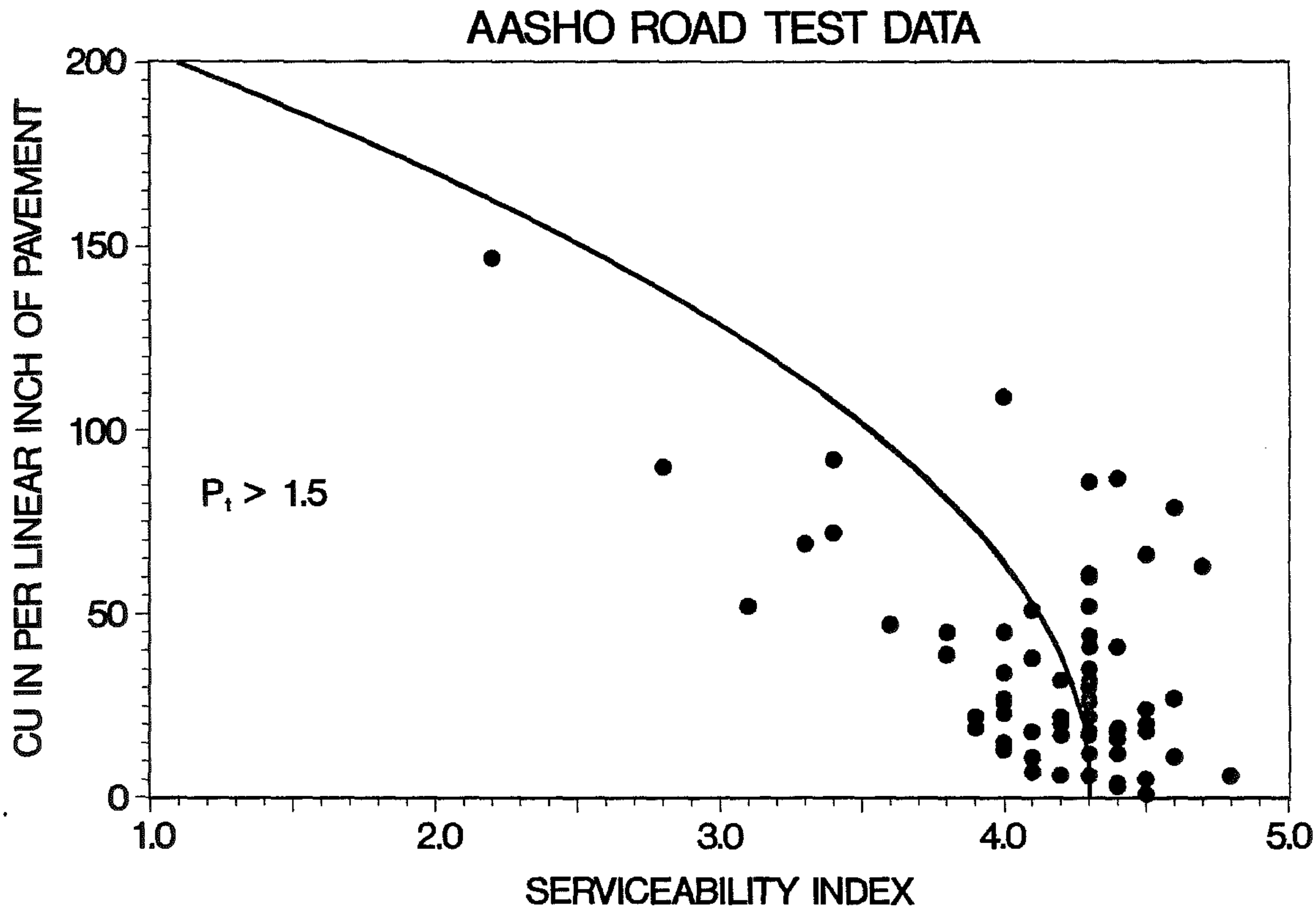


FIGURE 31. AASHO ROAD TEST DATA, PUMPING INDEX VERSUS SERVICEABILITY INDEX.

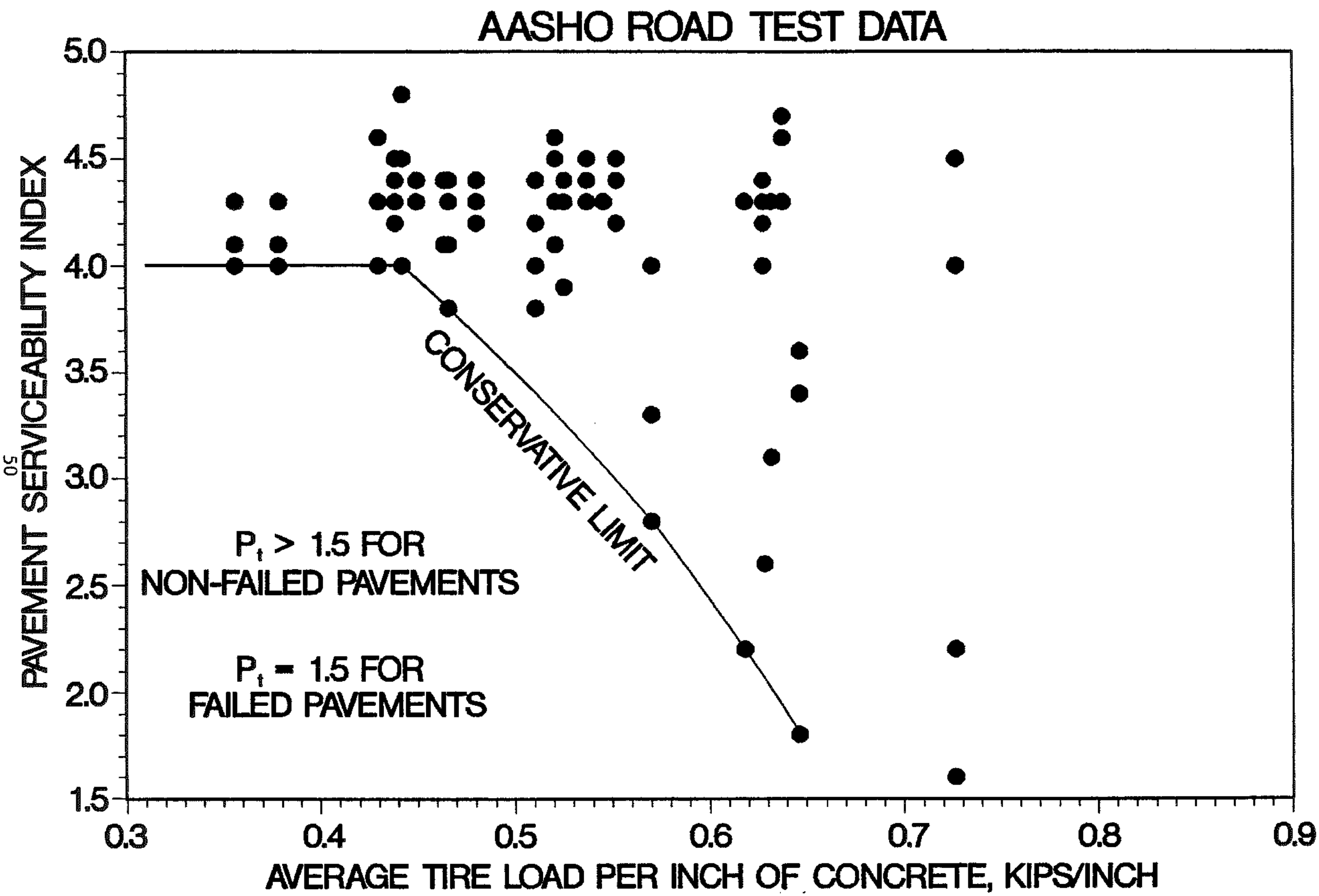


FIGURE 32. AASHO ROAD TEST SERVICEABILITY INDEX DATA VERSUS AVERAGE TIRE LOAD PER INCH OF CONCRETE--NON-FAILED PAVEMENTS AT END OF TEST.

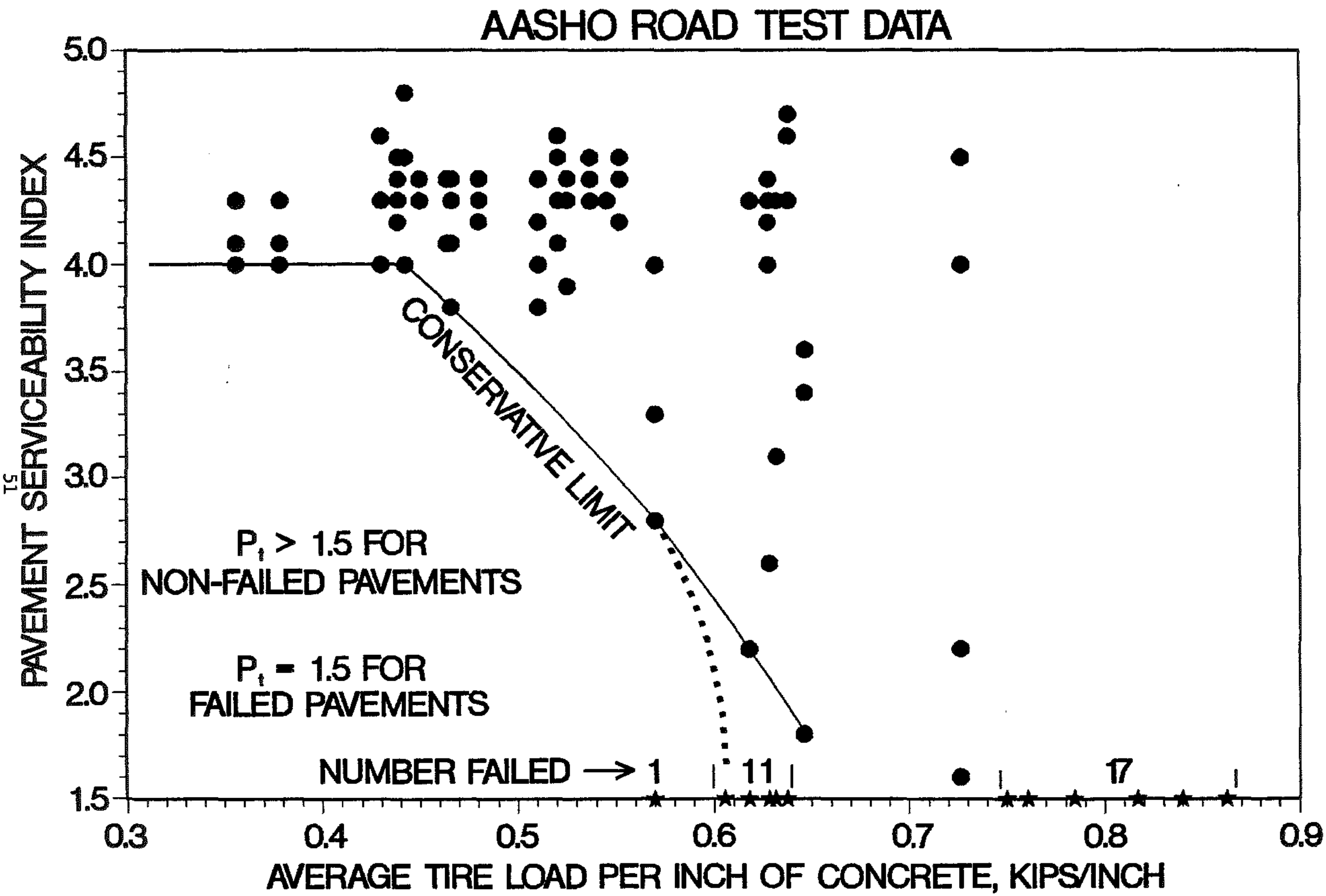


FIGURE 33. AASHO ROAD TEST SERVICEABILITY INDEX DATA VERSUS AVERAGE TIRE LOAD PER INCH OF CONCRETE--NON-FAILED PAVEMENTS AND FAILED PAVEMENTS AT END OF TEST.

TABLE 1. DESIGN THICKNESS OF PORTLAND CEMENT CONCRETE USING 1984 KENTUCKY DESIGN METHOD FOR RIGID PAVEMENT

18-KIP EAL	SUBGRADE MODULUS, PSI* 4,500	7,500	10,500	15,000
	DESIGN THICKNESS, INCHES			
3,000,000	8.35	7.75	7.30	6.75
5,000,000	9.05	8.60	8.05	7.50
7,000,000	9.70	9.15	8.60	8.00
10,000,000	10.35	9.70	9.20	8.60
20,000,000	11.70	11.10	10.50	9.80
30,000,000	12.80	11.90	11.40	10.60

* SUBGRADE MODULUS = 1,500 x CBR

TABLE 2. INPUT PARAMETERS AND VALUES USED FOR COMPARING DESIGN METHODS

PARAMETER	AASHTO	KENTUCKY
MODULUS OF ELASTICITY, PSI	4,200,000	4,200,000
MODULUS OF RUPTURE, PSI	600	600
LOAD TRANSFER COEFFICIENT	3.1	NOT A SPECIFIC VARIABLE IN DESIGN METHOD
DRAINAGE COEFFICIENT	1.0	NOT A SPECIFIC VARIABLE IN DESIGN METHOD
OVERALL STANDARD DEVIATION	0.39	NOT A SPECIFIC VARIABLE IN DESIGN METHOD
PERCENT RELIABILITY	50-98	NOT A SPECIFIC VARIABLE IN DESIGN METHOD
TERMINAL SERVICEABILITY	2.5 TO 3.5	NOT A SPECIFIC VARIABLE IN DESIGN METHOD, VALUE VARIES WITH CBR
RESILIENT MODULUS OF SOIL, PSI	4,500 7,500 10,500 15,000	4,500 7,500 10,500 15,000
DESIGN PERIOD	20 YEARS	ALREADY INCLUDED IN EAL
DESIGN EAL, MILLIONS	3 5 7 10 20 30	3 5 7 10 20 30

TABLE 3. DESIGN THICKNESS OF PORTLAND CEMENT CONCRETE USING 1986 ACPA DESIGN METHOD FOR RIGID PAVEMENTS, MODULUS OF RUPTURE = 600 PSI

		DESIGN THICKNESS, INCHES							
		k, PCI							
		106	150	186	238	106	150	186	238
		SUBGRADE MODULUS, PSI*							
	18-KIP	90 PERCENT RELIABILITY				80 PERCENT RELIABILITY			
P_t	EAL	4,500	7,500	10,500	15,000	4,500	7,500	10,500	15,000
2.50	3,000,000	8.83	8.68	8.57	8.44	8.29	8.13	8.02	7.88
	5,000,000	9.57	9.43	9.32	9.19	9.00	8.85	8.74	8.60
	7,000,000	10.09	9.94	9.84	9.71	9.49	9.034	9.23	9.10
	10,000,000	10.65	10.51	10.41	10.28	10.03	9.88	9.78	9.65
	20,000,000	11.83	11.69	11.59	11.47	11.15	11.00	10.90	10.78
	30,000,000	12.58	12.43	12.33	12.21	11.85	11.71	11.61	11.48
2.75	3,000,000	9.01	8.86	8.75	8.62	8.45	8.29	8.19	8.05
	5,000,000	9.76	9.62	9.51	9.39	9.17	9.02	8.92	8.79
	7,000,000	10.28	10.14	10.04	9.91	9.67	9.53	9.42	9.29
	10,000,000	10.86	10.72	10.62	10.50	10.22	10.08	9.98	9.85
	20,000,000	12.06	11.92	11.82	11.70	11.36	11.22	11.12	11.00
	30,000,000	12.82	12.67	12.58	12.46	12.08	11.94	11.84	11.72
3.00	3,000,000	9.21	9.07	8.96	8.84	8.64	8.49	8.38	8.25
	5,000,000	9.98	9.84	9.74	9.62	9.38	9.24	9.14	9.01
	7,000,000	10.52	10.38	10.28	10.16	9.89	9.75	9.65	9.53
	10,000,000	11.11	10.97	10.87	10.75	10.46	10.31	10.22	10.09
	20,000,000	12.33	12.19	12.10	11.98	11.62	11.48	11.38	11.27
	30,000,000	13.10	12.96	12.87	12.75	12.35	12.21	12.12	12.00
3.25	3,000,000	9.47	9.32	9.23	9.10	8.88	8.73	8.63	8.50
	5,000,000	10.26	10.12	10.02	9.90	9.64	9.50	9.40	9.28
	7,000,000	10.80	10.66	10.57	10.45	10.16	10.02	9.93	9.81
	10,000,000	11.41	11.27	11.18	11.06	10.74	10.60	10.51	10.39
	20,000,000	12.66	12.52	12.43	12.32	11.93	11.79	11.70	11.59
	30,000,000	13.44	13.31	13.22	13.11	12.68	12.54	12.45	12.34
3.50	3,000,000	9.79	9.65	9.55	9.43	9.18	9.04	8.94	8.87
	5,000,000	10.60	10.47	10.37	10.26	9.97	9.83	9.73	9.62
	7,000,000	11.16	11.03	10.94	10.82	10.50	10.37	10.28	10.16
	10,000,000	11.78	11.65	11.56	11.45	11.10	10.96	10.87	10.76
	20,000,000	13.07	12.94	12.85	12.74	12.32	12.19	12.10	11.99
	30,000,000	13.88	13.75	13.66	13.56	13.09	12.96	12.87	12.76

* k = Mr/19.4
E = 1,500 x CBR
Mr = f(E) (REF FIGURE 4)

TABLE 4. 1986 AASHTO SOIL SUPPORT-RESILIENT MODULUS RELATIONSHIP

X,LY	REGRESSION ANALYSES $Y=a+bX+cX^2$	$Y=a+bX$	INPUT DATA SOIL SUPPORT NO.	RESILIENT MODULUS, PSI
c	-0.000559157		2	2,000
b	0.16836713	0.16165725	3	3,000
a	2.972692615	2.98909455	4	4,400
SQR(RES MS)	0.00679175	0.00730034	5	6,300
R^2	0.9998235	0.9997621	6	9,300
F RATIO	16997.112	7	7	13,000
			8	19,000
			9	28,000
			10	40,000

EQUATIONS EVALUATED

SOIL SUPPORT NO.	RESILIENT MODULUS, PSI POLYNOMIAL	LINEAR	k, PCI POLYNOMIAL	LINEAR
0	939	675	48	50
1	1382	1415	71	73
2	2028	2053	105	106
3	2970	2979	153	153
4	4337	4322	224	223
5	6318	6272	326	323
6	9178	9100	473	469
7	13300	13204	686	681
8	19224	19158	991	988
9	27714	27798	1429	1473
10	39852	40334	2054	2079

TABLE 5. CRACKING INDEX DATA, TABLE 50, AASHO ROAD TEST SPECIAL REPORT 61E.

TABLE 50

CRACKING INDEX, C', AT 1,114,000 AXLE APPLICATIONS OR WHEN $p = 1.5$[1], EXPERIMENT DESIGN 1[2]

Loop	Axle Load (kips)	Subbase Thickness (in.)	Cracking Index, C': 2.5-In. Surface		3.5-In. Surface		5.0-In. Surface		6.5-In. Surface		8.0-In. Surface		9.5-In. Surface		11.0-In. Surface		12.5-In. Surface	
			R	N	R	N	R	N	R	N	R	N	R	N	R	N	R	N
2	2S	0	4	0	0	1	1	0										
		3			0			0										
			26	2	3	1	1	11										
		6	13	3	1	0	4	0										
	6S	0	129*	183*	20	60	0	0										
		3			15			0										
			118*	115	7	2	1	9										
		6	63	115	36	8	4	0										
3	12S	3					51			1								
					343*	258*	60	8	27	1	34	0						
		6						8	23									
					286*	256*	211*	16	35	1	35	1						
		9			252*	235*	48	31	31	0	28	0						
	24T	3					126*			0								
					171*	216*	169*	67*	46	0	25	1						
		6						56	25									
					194*	373*	143*	144*	43	0	35	0						
		9			212*	218*	152*	155*	38	0	23	0						
4	18S	3							63			1						
							178*	64*	71	8	38	0	39	1				
		6								0	36							
							112*	116*	80	0	52	0	25	1				
		9					77*	162*	145	26	46	0	29	1				
	32T	3							132			0						
							250*	119*	149*	154*	37	1	37	0				
		6								21	44	1						
							391*	126*	171*	155*	61		33	1				
		9					131*	205*	173*	193*	45	0	30	0				
5	22.4S	3									150*			0				
									195*	114*	30	0	38	0	4	10		
		6										0	17					
									184*	189*	47	9	15	42	8	0		
		9							100*	117*	44	88*	18	1	0	0		
	40T	3									179*			0				
									153*	122*	34	4	42	4	19	2		
		6										24	38					
									293*	123*	157*	9	24	17	13	0		
		9							209*	105*	82	98*	50	19	10	0		
6	30S	3											6			0		
											93*	92*	200	28	15	0	4	9
		6												4	33			
											126*	29	44	1	31	0	21	1
		9									246*	12	164	0	22	0	0	0
	48T	3											19			0		
											195*	68	41	30	25	0	8	8
		6												9	41			
											74*	0	66	0	33	0	26	3
		9									58*	73*	163*	2	56	0	11	0

[1] Values with asterisk are for $p = 1.5$.
[2] R = reinforced; N = nonreinforced.

TABLE 6. PUMPING INDEX DATA, TABLE 54, AASHO ROAD TEST SPECIAL REPORT 61E.

TABLE 54

PUMPING INDEX AT $p = 1.5$[1] OR $W = 1{,}114{,}000$, EXPERIMENT DESIGN 1[2]

Loop	Axle Load (kips)	Subbase Thickness (in.)	Pumping Index: 2.5-In. Surface		3.5-In. Surface		5.0-In. Surface		6.5-In. Surface		8.0-In. Surface		9.5-In. Surface		11.0-In. Surface		12.5-In. Surface	
			R	N	R	N	R	N	R	N	R	N	R	N	R	N	R	N
2	2S	0																
		3																
		6																
	6S	0	25*	11*	8	20	4	4										
		3	5*	17	10 12	7	5	6 6										
		6	11	34	7	13	2	2										
3	12S	3			315*	62*	109 90	53	17	22 19	7	18						
		6			214*	102*	211*	83 63	18 24	17	15	18						
		9			204*	149*	69	88	12	17	18	18						
	24T	3			92*	37*	86* 118*	73*	52	37 22	18	24						
		6			69*	76*	82*	65 106*	51 87	24	23	31						
		9			88*	103*	101*	146*	45	21	30	27						
4	18S	3					189*	191*	47 72	48	19	24 20	13	16				
		6					116*	91*	92	29 57	19 22	24	5	20				
		9					98*	147*	117	209	18	21	6	16				
	32T	3					216*	202*	89 152*	50*	26	35 29	34	53				
		6					202*	101*	210*	112 86*	41 39	39	12	28				
		9					75*	118*	116*	178*	32	30	11	27				
5	22.4S	3							207*	133*	146* 32	33	27	22 27	11	23		
		6							104*	301*	63	47 97	20 31	52	18	2		
		9							193*	203*	79	122*	16	28	4	3		
	40T	3							91*	108*	127* 35	37	38	29 31	17	35		
		6							123*	111*	210*	67 47	61 66	113	22	0		
		9							77*	114*	142	98*	27	84	12	12		
6	30S	3									122*	150*	18 83	32	19	15 19	4	22
		6									237*	159	45	29 52	27 31	20	6	20
		9									237*	168	120	59	22	12	1	3
	48T	3									95*	164	52 44	185	26	25 22	6	53
		6									208*	133	41	36 83	60 26	21	20	46
		9									123*	105*	228*	40	86	24	3	22

[1] Values with asterisk are for $p = 1.5$.
[2] R = reinforced; N = nonreinforced.

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APPENDIX A

DEVELOPMENT OF KENTUCKY RIGID PAVEMENT DESIGN CRITERION

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The Kentucky thickness design curves for portland cement concrete (1) are based upon the principle of work. In classical physics, work is defined as the deformation of a body due to application of an outside force. The energy within the body that resists deformation due to the outside force is called strain energy. Strain energy density is defined as the energy of deformation per unit volume (2). The summation of strain energy densities for the entire volume of the body is called strain energy and is equal and opposite to the work exerted by the outside force. The Chevron N-layer computer program (3) is based upon elastic theory and has been modified by Kentucky (4) to include the calculation of work. The units of work used throughout this report are inch-pounds.

WORK

The following equation (2, 4) was added to the Chevron N-layer computer program to calculate the strain energy density at a given point within the pavement structure:

$$W = (1/2)\lambda v^2 + \mu(\, \varepsilon_{11}^2 + \varepsilon_{22}^2 + \varepsilon_{33}^2 + 2\varepsilon_{12}^2 + 2\varepsilon_{13}^2 + 2\varepsilon_{23}^2 \,) \tag{1}$$

in which W = strain energy density, or energy of deformation per unit volume, or the volume density of strain energy,

$\in_{ij}$ = i, jth component of the strain tensor,

= $E/(2(1 + \sigma))$, the modulus of rigidity or the shear modulus, psi

E = Young's modulus, psi

σ = Poisson's ratio

$\lambda = E\sigma/((1 + \sigma)(1 - 2\sigma)$, and

$v = \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33}$.

Strain energy density accounts for all nine components of strain, or stress, four of which will have no resultant value because one shear component balances another component in two situations. All components are calculated and printed. Work is the three dimensional summation of strain energy density for the volume of material involved. It was assumed that work also was equal to the calculated strain energy density (Work = in.^{3} x psi = in.-lb) for a unit volume at a given point within the pavement structure.

The Chevron N-layer computer program was used to analyze pavement structures involving a matrix of layer thicknesses and material parameters (1). A subset of the matrix corresponding to Kentucky empirical experience is shown in Figure A1.

Tensile stresses and strains at the bottom of the portland cement concrete slab were computed for the matrix. Appropriate tensile strains were determined for fatigue criteria corresponding to the 1966 Portland Cement Association's thickness design procedure (5) and the 1972 AASHTO Interim Guide(6) as shown in Figure A2. Figure A2 also illustrates that the tensile strain fatigue relationship for the two design systems has a common transition zone between 2 and 3 million 18-kip EALs. The Portland Cement Association's design method was based upon a model developed from the full-scale Arlington tests (7) conducted in the 1940's and would be applicable to low-volume roads for today's environment. Conversely, the AASHO Road Test (8) was conducted in the early 1960's and had high volumes of trucks within relatively few years. Designs for large EALs (lower dashed line) using the PCA method resulted in excessive thicknesses as judged by Kentucky experience and designs for relatively low EALs (upper dashed line) using the AASHTO design method resulted in thicknesses that were far thinner than Kentucky experience dictated. Combining the two systems into one system resolved some discrepancies when using the individual systems.

Figure A3 shows the relationship between tensile strain and work at the bottom of the rigid pavement. The relationship of fatigue and tensile strain shown in Figure A2 was adjusted using Figure A3 to produce Figure A4. Figure A4 provides the relationship between 18-kip EAL and work at the bottom of the portland cement concrete slab. This fatigue relationship is the basis for the Kentucky thickness design curves shown in Figure A5.

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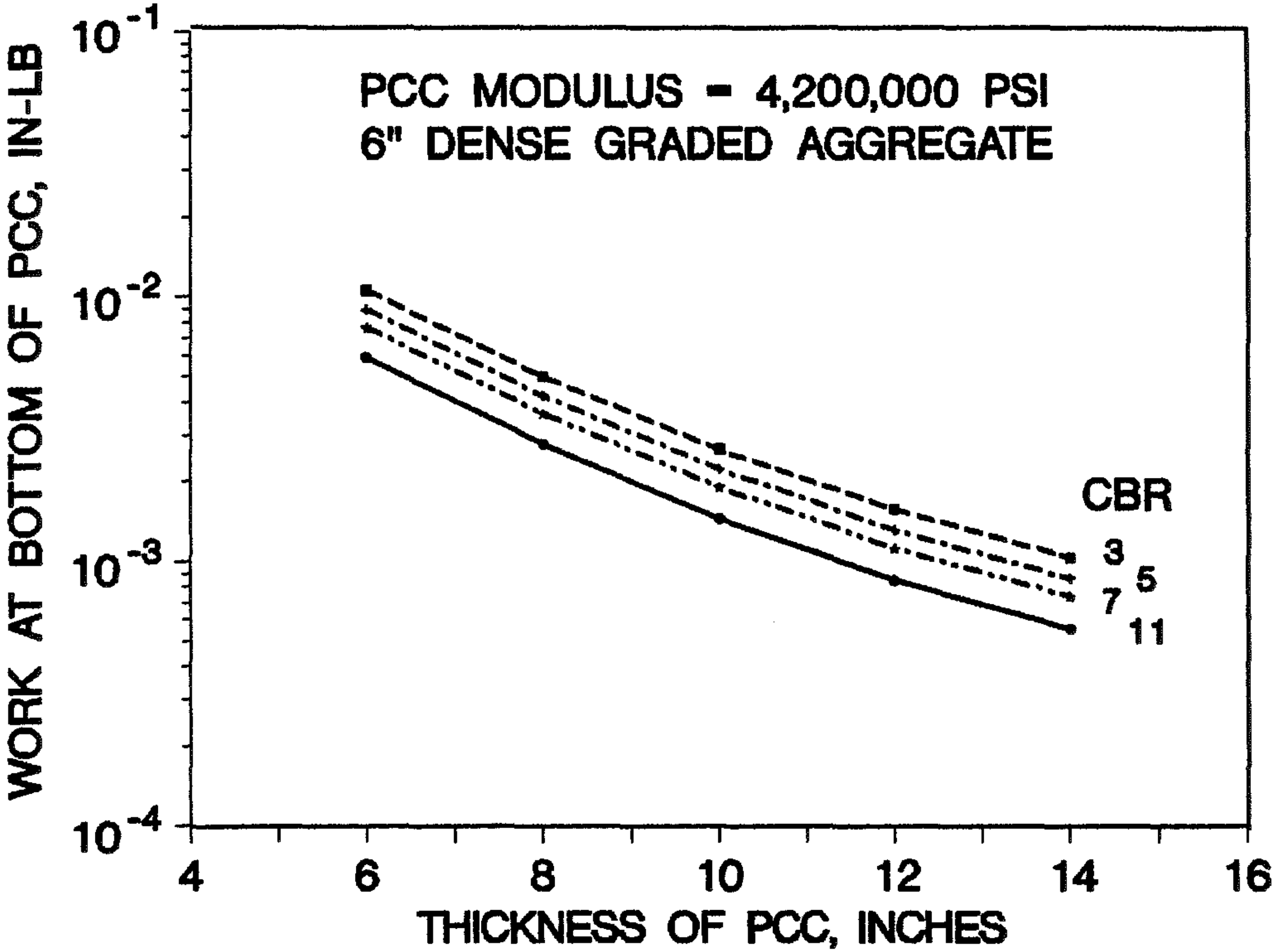

FIGURE A1. RELATIONSHIP OF THICKNESS OF PORTLAND CEMENT CONCRETE PAVEMENT AND WORK AT BOTTOM OF PORTLAND CEMENT CONCRETE PAVEMENT.

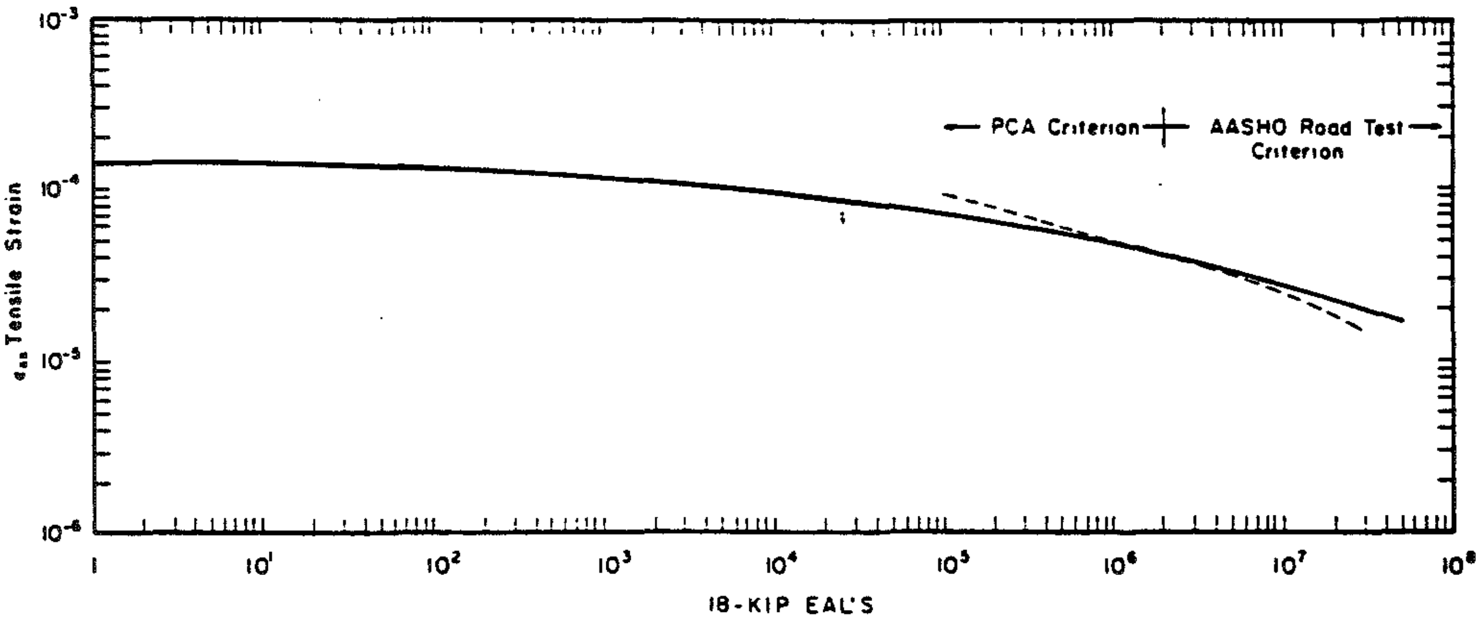

FIGURE A2. TENSILE STRAIN VS FATIGUE RELATIONSHIP FOR PORTLAND CEMENT CONCRETE PAVEMENTS.

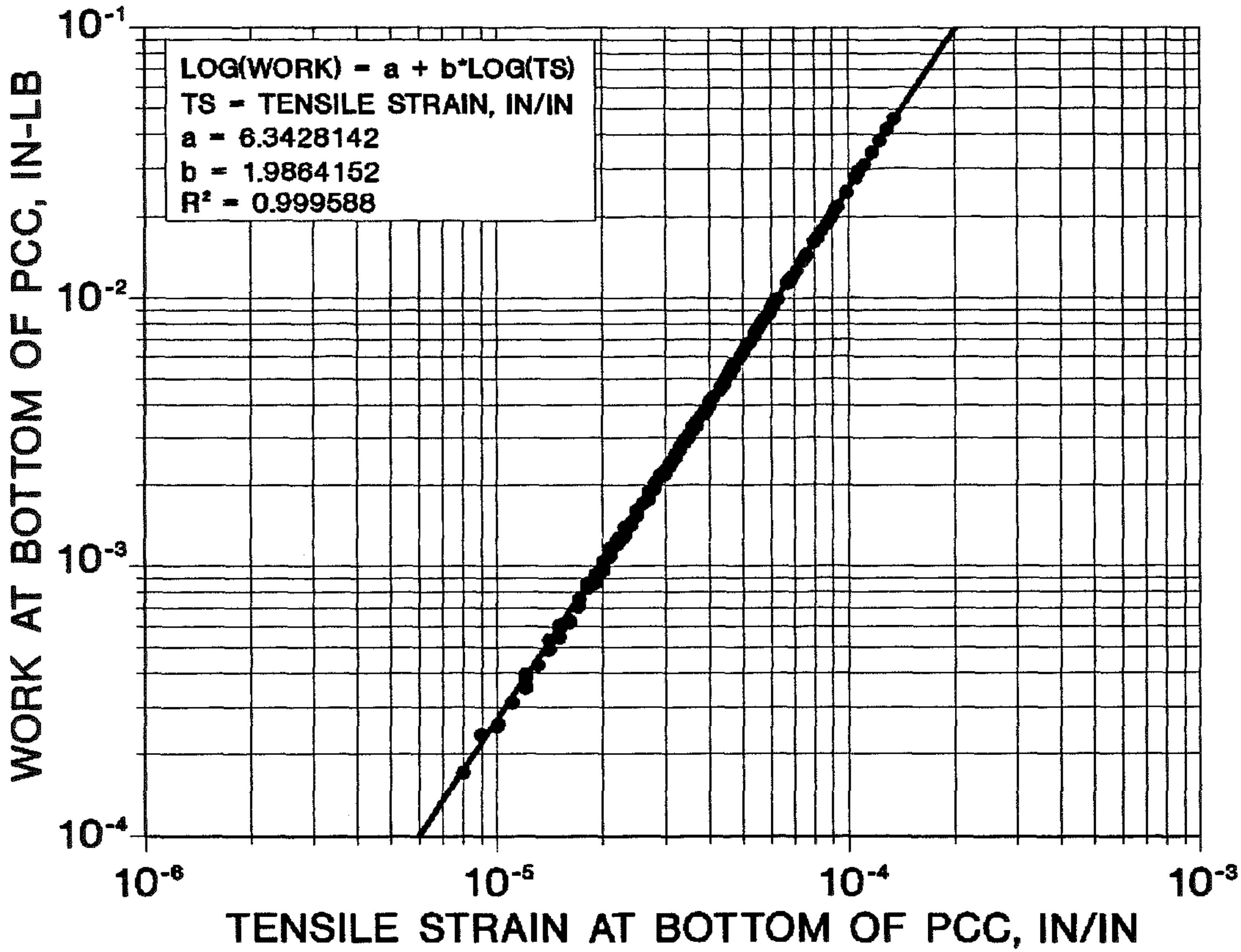

FIGURE A3. METHOD TO ADJUST TENSILE STRAIN TO EQUIVALENT WORK AT BOTTOM OF PORTLAND CEMENT CONCRETE PAVEMENT.

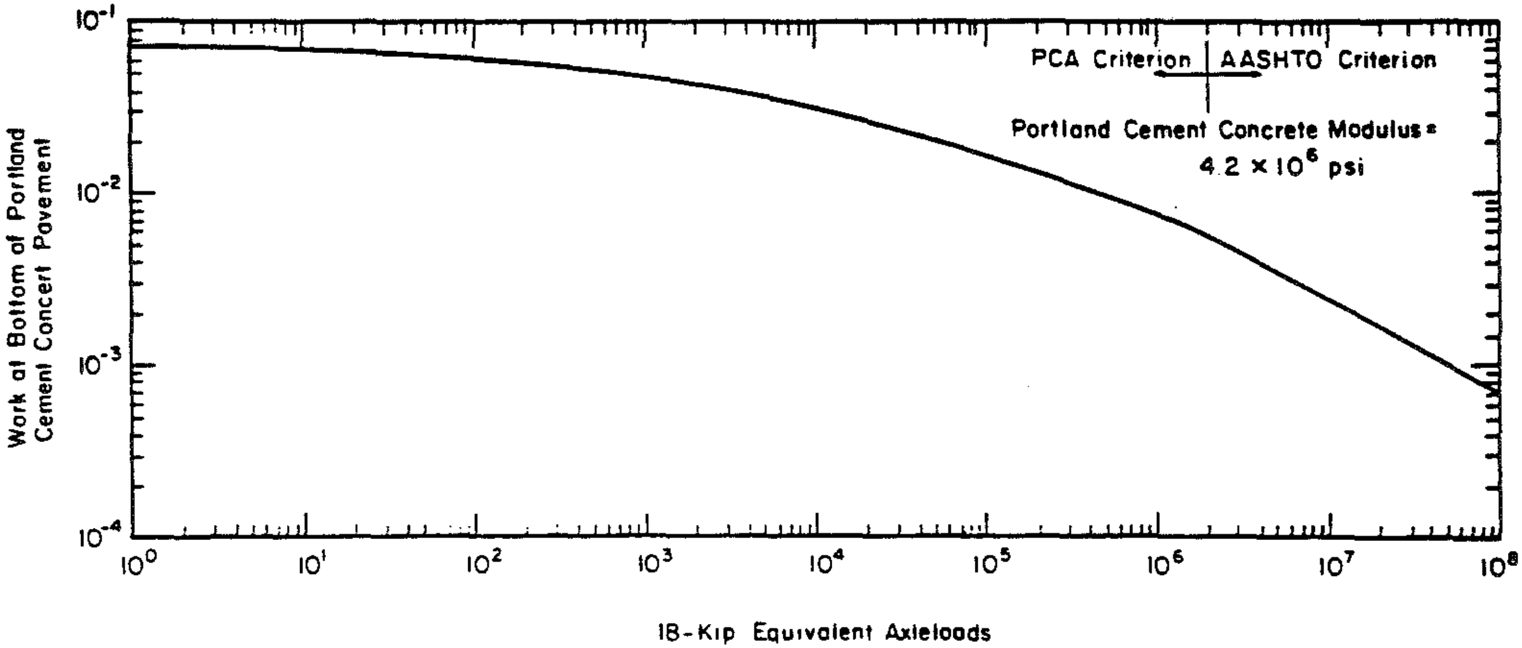

FIGURE A4. RELATIONSHIP OF WORK AND FATIGUE AT BOTTOM OF PORTLAND CEMENT CONCRETE PAVEMENT.

KY RIGID PAVEMENT THICKNESS DESIGN CURVES

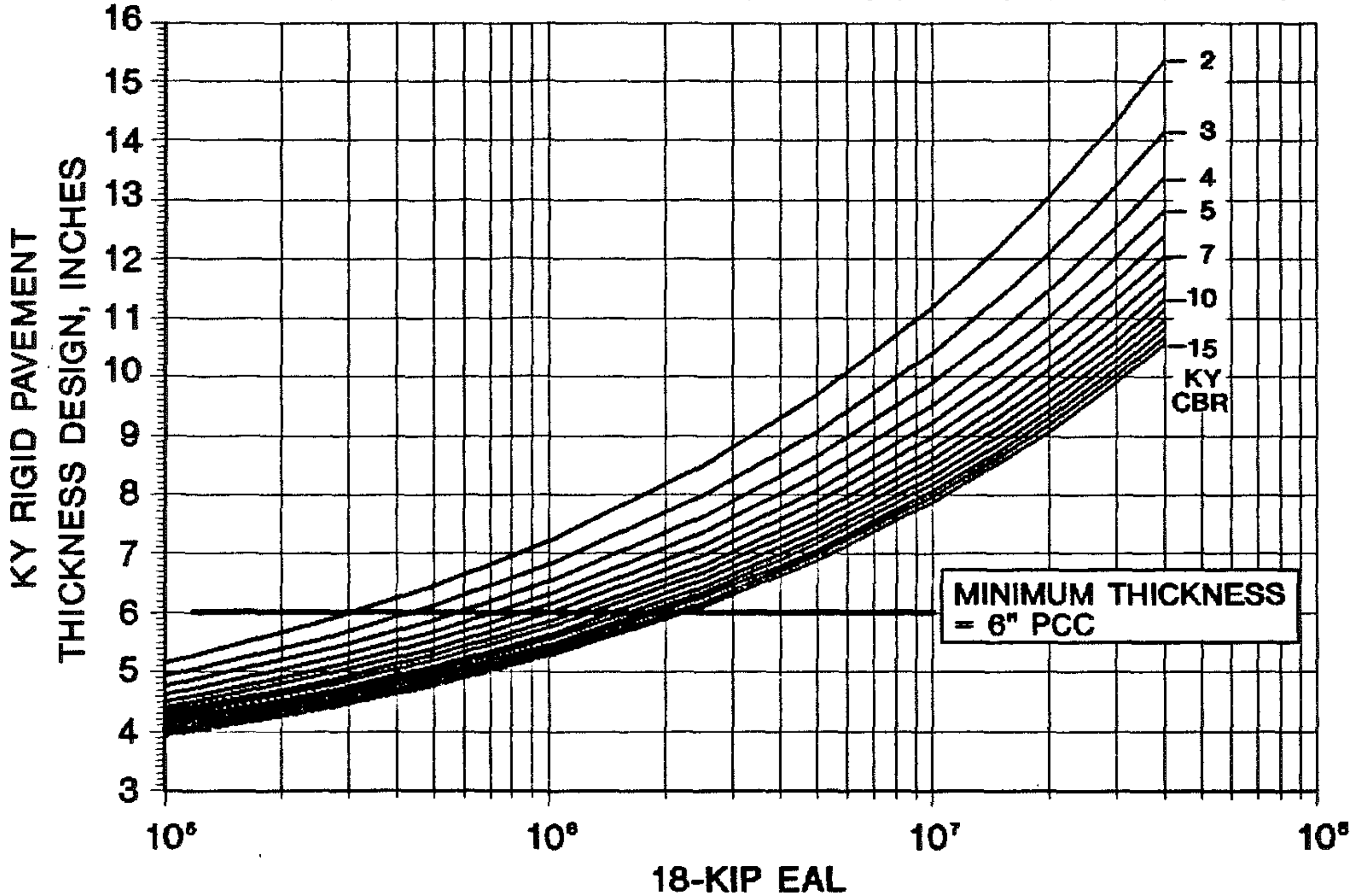

FIGURE A5. KENTUCKY PORTLAND CEMENT CONCRETE PAVEMENT THICKNESS DESIGN CURVES.

APPENDIX B

PCA RIGID PAVEMENT DESIGN CRITERION

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The basis for the 1984 PCA design method (1) is:

"The thickness design methods presented here are based on knowledge of pavement theory, performance, and research experience from the following sources:

1. Theoretical studies of pavement slab behavior by Westergaard (2), Pickett and Ray (3), and recently developed finite-element computer analyses, one of which is used as the basis for this design procedure (1).

2. Model and full-scale tests such as Arlington Tests and several research projects conducted by PCA and other agencies on subbases, joints, and concrete shoulders.

3. Experimental pavements subjected to controlled test traffic, such as the Bates Test Road (4), the Pittsbury Test Highway (5), the Maryland Road Test (6), the AASHO Road Test (7), and studies of in-service highway pavements made by various state departments of transportation.

4. The performance of normally constructed pavements subject to normal mixed traffic."

"The theoretical parts of the design procedures given here are based on a comprehensive analysis of concrete stresses and deflections by a finite-element computer program. The program models the conventional design factors of concrete properties, foundation support, and loadings, plus joint load transfer by dowels or aggregate interlock and concrete shoulder, for axleload placements at slab interior, edge, joint, and corner."

"The criteria for the design procedures are based on the pavement design, performance and research experience referenced above including relationships to performance of pavement at the AASHO Road Test and to studies of the faulting of pavements."

From Appendix A, page 34 (1), the following is quoted from the section "Fatigue":

"The flexural fatigue criterion used in the procedure presented here is shown in Fig. A3" (Figure B1 in this report). "It is similar to that used in the previous PCA method based conservatively on studies of fatigue research except that it is applied to edge-load stresses that are of higher magnitude. A modification in the high-load-repetition range has been made to eliminate the discontinuity in the previous curve that sometimes causes unrealistic effects.

The allowable number of load repetitions for a given axleload is determined based on the stress ratio (flexural stress divided by the 28-day modulus of rupture). The fatigue curve is incorporated into the design charts for use by the designer."

In Figure B1, the curves labeled "Hilsdorf and Kesler" and "PCA Curve" are the curves given in "Fig. A3". The curve marked "1966 PCA" has been added and is the fatigue criterion curve from Figure A2 used in the Kentucky method up to approximately 2 million 18-kip EALs. This curve appears to best fit the experience quoted from the 1984 PCA Thickness Design (1).

LIST OF REFERENCES

1. Packard, R. G., "Thickness Design for Concrete Highway and Street Pavements", Portland Cement Association, Skokie, Illinois, 1984.

2. Westergaard, H. M., "Theory of Concrete Pavement Design," Highway Research Board Proceedings, Part 1, Washington, D. C., 1927.

3. Pickett, G., and Ray, G. K., "Influence Charts for Concrete Pavements," American Society of Civil Engineers Transactions, Paper No. 2425, Vol. 116, 1951.

4. Older, Clifford, "Highway Research in Illinois," Proceedings of American Society of Civil Engineers, February 1924.

5. Aldrich, L., and Leonard, I. B., "Report of Highway Research at Pittsbury, California, 1921-1922," California State Printing Office.

6. Road Test One-MD, Highway Research Board Special Report No. 4, Washington, D. C., 1952.

7. The AASHO Road Test, Highway Research Board, Special Report No. 61E, Washington, D. C. 1962.

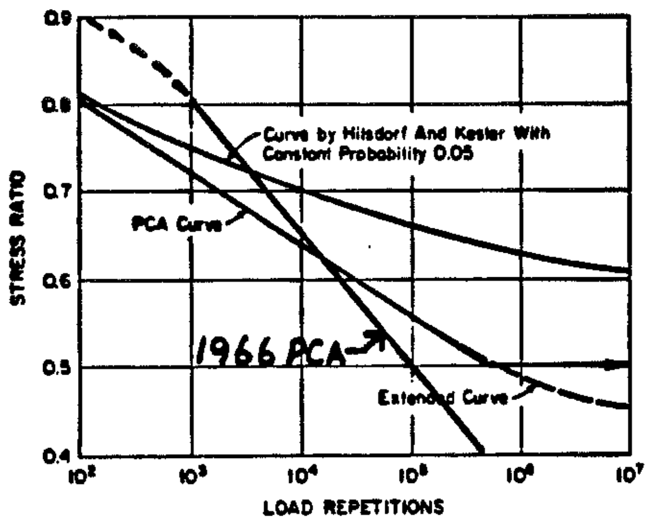

Fig. A3. Fatigue relationships.

FIGURE B1. PORTLAND CEMENT ASSOCIATION'S FATIGUE RELATIONSHIP.

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APPENDIX C

RELATIONSHIP BETWEEN SOIL SUPPORT VALUE AND RESILIENT MODULUS

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The 1986 AASHTO (1) design equation contains an additional term for reliability and standard deviation that was not included in the 1972 AASHTO Guide (2). The soil support-resilient modulus relationship (Table 4) was extracted from Appendix FF (1) and was developed for flexible pavements. The 1972 equation includes a soil support term while the 1986 equation substitutes an expression for resilient modulus and adds a term for reliability and standard error. All other terms are identical in both equations. An expression relating soil support (Si), resilient modulus (Mr), reliability (Zr), and standard error (So) may be derived as follows:

From Figure 3.1, 1986 AASHTO Guide for Design of Pavement Structures (1):

$$\log(EAL) = ZrSo + 9.36Log(SN+1) - 0.20 + G_t/(0.4 + 1094/(SN+1)^{5.19}) + 2.32Log(Mr) - 8.07 \quad (1)$$

Eq. C-12, 1972 AASHTO Interim Guide for Design of Pavement Structures (2):

$$\log(EAL) = 9.36Log(SN+1) - 0.20 + G_t/(0.4 + 1094/(SN+1)^{5.19}) + \log(1/R) + 0.372(Si - 3.0) \quad (2)$$

where R = regional factor and was assigned a value of 1.0 for the AASHO Road Test. If R = 1, then Log(1/R) = 0.0. Canceling like terms in both equations and setting them equal, results in:

$$ZrSo + 2.32Log(Mr) - 8.07 = 0.372(Si - 3.0) \quad (3)$$

A mean fit corresponds to a reliability of 50 percent which corresponds to Zr = 0. For Zr = 0, Equation 3 reduces to:

$$2.32Log(Mr) - 8.07 = 0.372(Si - 3.0) \quad (4)$$

$$Log(Mr) = (8.07 - 0.372(3.0) + 0.372Si)/2.32$$
$$= 2.997414 + 0.160345(Si) \quad (5)$$

The term ZrSo is in the 1986 equation but not in the 1972 equation, it still should not be included in Equation 3 because the design nomograph includes scales to permit changing input values for reliability and standard error separately. Including ZrSo in Equation 3 would produce a higher value for "k" resulting in thinner pavement thicknesses than intended. Therefore, Equation 5 is the AASHTO relationship to convert from Si used in the 1972 Guide (2) and Mr in the 1986 Guide 1).

LIST OF REFERENCES

1. Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1986.

2. 1972 AASHTO Interim Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1972 (1982 Printing).

APPENDIX D

SUBGRADE STIFFNESS RELATIONSHIPS

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Appendix C provides the development of the relationship of Soil Support Value with "M_r". Figure D1, Figure HH.2. (1), provides the relationship between resilient modulus, M_r, and modulus of subgrade reaction, k. However, the 1986 AASHTO Guide also defines M_r = 1,500(CBR). Figure D2 is a nomograph relating soil support value to the current Kentucky CBR relationship, both resilient moduli relationships, and the resulting "k" relationships derived by using Figure D1.

Inspection of Figure D2 shows that a wide difference in values for "k" can be obtained depending upon the chosen relationship. A sensitivity study was made for four values of CBR, levels of serviceability ranging from 2.5 to 3.5 by increments of 0.5. Figure D3 is a combination of those calculations with the range of values shown for the input variables. Figures D4 through D6 correspond to terminal serviceability values of 2.5, 3.0, and 3.5, respectively. Inspection will show that the line for the same CBR in Figures D5 through D6 overlay each other to produce the same line in Figure D3. Though the lines are labeled as values of CBR, the lines really form a family of values for "k" and are generic provided the same modulus of rupture and modulus of elasticity are held constant. No investigation was made for other values of moduli. With the above restriction and for any other k-M_r relationship that may be determined to be more appropriate in the future, the correlation equation given in the lower right corner of Figure D3 will provide the means to determine equivalent thicknesses regardless of which "k" is used.

Figure D7 is the same as Figure FF.6. (1). As a matter of clarification, the scale labeled "CBR-(KENTUCKY)" is not the one that has been used in Kentucky in the past, or currently. The document referenced in the footnote was reviewed and that Kentucky CBR vs Soil Support Value relationship is not included in the referenced document. The authors, W. B. Drake and J. H. Havens were contacted and both authors deny that this is their work and do not know the source of that relationship. Thus, this scale should be removed from Figure FF.6 (1). The referenced documents for "Scales A and B" were reviewed and their direct correlation with Soil Support Value could not be found in the documents. Also, the referenced documents were written by 1959 and the soil support scale was developed (2) after the AASHO Road Test was completed (after 1962).

Figure D8 is the same as Figure C.3-6 (3). Inspection indicates that these scales do not agree with those in Figure D7. The scale labeled Soil Support Value is common to Figures D7 and D8. Approximate values were interpolated from Figures D7 and D8 to make Figure D9. Should the interpolated values be slightly in error, the overall difference in the beginning and ending values for each scale indicates there still is a large variability. In Figure D9, each scale has been assigned an identification number at the bottom of the figure.

Scales 5-8 are a group of "R-Value" relationships. Note the large variation in the location of the "zero" end of these scales compared to the scale labeled "Soil

Support Value". Scales 2, 3, 9, and 10 are a group of "CBR" relationships also indicating a wide variation in beginning and ending values compared to the "Soil Support Value" scale. Scales 11-14 are duplicates of scales shown in Figure D2.

Figure D7 originally came from NCHRP Report 128 (2). This document was reviewed. The following is quoted starting on Page 27 and items that might be open for comment are marked with an asterisk and number (Example: *1):

SOIL SUPPORT

The correlation of the soil support scale in the Interim Guide for flexible pavements with local conditions and procedures has also presented problems to the highway engineer. In this section layered theory is used to develop a rational procedure for correlating local materials with the soil support scales in the Guides, and a procedure is presented whereby a soil support value may be developed on the basis of resilient modulus tests. Using data collected from the highway departments, scales are also provided for estimating soil support from currently used strength tests.

Development of Scale F

Using relationships between W_{t18} and pavement and subgrade strain derived from layered theory, a series of tables of pavement component strains and load applications were developed for subgrade modular values other than those found at the AASHO Road Test and for surface thickness of 3 and 5 in. and surface modulus of 150,000 and 600,000 psi. For each structural number, subgrade modulus, and surface modulus, a corresponding vertical strain on subgrade and tensile strain in the bottom fiber of asphaltic concrete surface was derived (Appendix C).

Using the strain versus W_{t18} data discussed previously, a theoretical soil support scale was developed. For a given structural number and a given number of equivalent daily 18-kip single-axle load applications, the location of soil support points for subgrade modular values of 3,000, 7,500, and 15,000 **(*1)** were established graphically. The theoretical soil support curves based on vertical compressive strain on the subgrade, shown in Appendix C, take a shape similar to that of the assumed curve (i.e., approximately vertical). It was found that surface thickness does not play as significant a role in determining the soil support scale as does surface modulus. After scales were established for several different values of the surface modulus of elasticity, it was concluded that the assumption of a linear soil support scale is valid, and the establishment of a relationship between soil support and resilient modulus would follow. (See *1 for a detailed discussion.)

Recommendations

It is concluded that vertical compressive strain on the subgrade was the most significant factor affecting the performance of the roads at the AASHO Road Test. As a result of the work shown in Appendix C, a relationship was established between soil support and resilient modulus of the subgrade soil. Using 3,000 psi as the modulus of the subgrade soil at the AASHO Road Test, a relationship between modulus and soil support was developed. This relationship is summarized in Figure 28 (Figure D2). After comparing the modulus scale, F, with R-value scale, A, and CBR scale, C, as a check of the validity of the soil support scale, the following comments are made:

1. In available literature the modulus of a good crushed stone or aggregate base is reported to range from 15,000 to 35,000, depending on the magnitude of the vertical stresses applied. This would correspond to an R-value of the range of about 60 to 85 and a CBR of about 20 to 80. Both of these ranges are in line with what is usually considered to be the range from a good aggregate subbase to a good aggregate base. Thus, the scale F appears to be reasonable in the upper ranges.
2. For subgrade soil, a 3,000-psi modulus would be considered good. When one compares these values with scales A and C, it can be seen that, for the range of modular values from 3,000 to 10,000 psi,**(*1)** the corresponding range of R-value would be from 10 to about 45, and CBR from 2 to about 10. This indicates that the scale F appears reasonable in the lower range also.

On the basis of this investigation, it appears that the soil support scale assumed in the Interim Guide is reasonably valid. However, when R-value, CBR, and modulus as determined in this section are compared with the relationships between R-value, CBR, and modulus developed in the structural layer coefficient analysis, there is a slight difference, particularly at the higher values of modulus. This difference is attributable to the different method of analysis. In the case of the soil support scale, the relationship between soil support and modulus was determined on the basis of vertical strain in the subgrade.

The following is quoted from NCHRP Report 128, Appendix C (2), on Page 78:

"DEVELOPMENT OF A RATIONAL SOIL SUPPORT SCALE

As mentioned in the Interim Guide for design of flexible pavements, many basic assumptions were made in the development of the design charts in the Guides. One of these assumptions was:

It has been necessary to assume a scale for the soil support value on (the design) charts...3.0 on the scale represents the silty clay roadbed soils on the Road Test, it is a firm and valid point. 10.0 represents crushed rock base material such as used on the AASHO Road Test. It is a reasonably valid point. All other points on the scale are assumed.

Following is a discussion of the approach used to check validity of the soil support scale for use in the design of flexible pavements.

The need for planned satellite studies subsequent to the Road Test was clearly emphasized in HRB Special Report 61-E (4), particularly from the standpoint of strengthening the soil support scale. Satellite studies on soils differing from those at the Road Test would make it possible to establish empirically a stronger based and a more reliable soil support scale. Because of the limited number of satellite studies that have been conducted, it was apparent that some other means must be used to strengthen the soil support scale. One such means is through application of theory, such as layered elastic analysis.

Several investigators have established the applicability of layered elastic theory to the prediction of deflections and of stresses and strains in a pavement structure. These investigators have indicated the reliability of these predicted responses through comparisons of measured responses on either prototype pavements or full-scale test roads. On the basis of these investigations it was concluded that a first step toward a rational soil support scale should be the application of layered elastic theory, and that additional refinements should be made as new developments and new methods for characterizing the pertinent properties of pavement components become available.

The response of the pavement to one dual wheel of an 18-kip axle load (i.e. two 4,500-lb wheel loads) is used for this analysis. The contact area for each of the loads is assumed to be circular, and the spacing between the tires is assumed to be equal to one load radius. The variables considered in the analysis are:

1. The modulus of the surface layer (E_1), 150,000 and 600,000 psi.**(*2)**
2. The modulus of the base layer (E_2), 15,000 psi.**(*2)**
3. The modulus of the subgrade layer (E_3), 3,000, 7,500, and 15,000 psi.**(*1)**
4. The thickness of the surface layer (D_1), 3, 4, 5, 6, 8, and 10 in.
5. The thickness of the base layer (D_2).

The surface and base moduli, and one level of subgrade modulus (E_3=3,000 psi), are similar to that established at the AASHO Road Test. The other values of subgrade modulus, 7,500 and 15,000 psi, were selected primarily to represent a side range of subgrades from poor to good, with assumed correlation with CBR values about as follows:

SUBGRADE TYPE	MODULUS (PSI)	CBR
Poor	3,000	2
Fair	7,500	5
Good	15,000	10+ (*1)

Also considered in the analysis were six levels of surface thickness, ranging from 3 to 10 in., to cover the broad range of surfacing thickness used on heavy-duty highways. The corresponding base thicknesses used for each surface level were determined from

$$SN = a_1D_1 + a_2D_2 \qquad \text{(C-17)}$$

in which SN is the structural number; a_1 and a_2 are structural coefficients for the surface and base, respectively; and D_1 and D_2 are the thickness of the surface and base, respectively.

Several investigators have indicated that two of the most critical responses in the pavement are the tensile strain on the bottom fiber of the asphaltic concrete (E_{ac}) and the vertical compressive strain on the subgrade (E_{sg}). The first is generally associated with fatigue cracking, and the second is associated with distortion of the pavement, such as rutting or corrugating. For this analysis, E_{ac} and E_{sg} were calculated for each of the combinations of variables with the aid of an IBM 6400 digital computer and Chevron Research Corporation's program for solution of the layered elastic equation. Calculations were made for one 4,500-lb tire load, and, in order to obtain the effect of the dual tires, the response of a second 4,500-lb tire spaced at three load radii was superimposed on it. The results of the calculation are show in Figures C-19 and C-20 (Figures D10 and D11 herein) (*3). Note that E_{ac} and E_{sg} are functions of the structural number, the subgrade modulus, and the surface modulus.

The values for equivalent 18-kip single-axle load applications to a given level of serviceability were calculated using the following equation from the Interim Guides:

$$\log W_{18} = 9.36\log(SN+1) - 0.20 + \frac{G}{0.40 + \frac{1094}{(SN+1)^{5.19}}} \qquad \text{(C-18)}$$

For each structural number, unadjusted for climatic and soil conditions, the number of equivalent 18-kip single-axle load applications (W_{t18}) was calculated for terminal serviceability indices of 2.5 and 2.0, with results as follows:

(*4) EQUIVALENT 18-KIP SINGLE-AXLE LOAD APPLICATIONS, W_{t18}

SN	$p_t = 2.5$	$p_t = 2.0$
1.5	3,193	3,278
2.0	16,454	17,534
3.0	186,514	230,335
4.0	1,088,780	1,610,795
5.0	4,805,546	8,044,522
6.0	18,138,485	32,365,071

On the basis of the relationships established here, and the calculated strains summarized in Figures C-19 and C-20 (Figures D10 and D11 herein), figures"..."were prepared to show the relationships of both vertical compressive strain and tensile strain in the bottom fiber of the asphaltic concrete as functions of W_{t18} for terminal serviceability indices of 2.5 and 2.0 for the AASHO Road Test conditions, and two levels of surface modulus (150,000 and 600,000 psi)."

The following is quoted from NCHRP Report 128, Appendix C (2), on Page 98:

Determination of k for Use with the Rigid Pavement Design Equations

The k-value (modulus of subgrade reaction) used on the AASHO design chart for rigid pavements is somewhat smaller than the k-value to which engineers are accustomed. That used in rigid pavement design is usually the so-called "elastic k." The k used as a basis for development of the Interim Guide for rigid pavements is the "gross k." The gross k is smaller than the elastic k because the total deflection of the plate is considered in the calculations.

The elastic k was used in this development because its values are generally in the range with which engineers are familiar, and it comes closer to duplicating the original Westergaard assumptions. Therefore, when one is comparing the results of the design charts with the AASHO design charts, this difference in the k-value should be taken into consideration. The studies at the AASHO Road Test showed the following correlation between the two k-values:

$$k_E = 1.77\ k_G \qquad \text{(C-45)}$$

in which
k_E = elastic modulus of support, pci; and
k_G = gross modulus of support, pci. ***5**

The problem of determining a k-value for use in rigid pavement design is compounded by other factors, such as the ability of a material to maintain its initial value over the life of the pavement. As an indication of the range of k-values to be expected, one can look at the supporting materials used at the AASHO Road Test. The basic subgrade material was an A-6 clay, Texas Triaxial Class 5.6. When used directly, this material had a gross k of 20 to 30. A subbase material was provided for most of the sections of the AASHO Road Test. This subbase material was a sandy gravel, Texas Triaxial Class 3.7. Six to 9 in. of this material resulted in a gross k-value of 50 to 75, with an average of 60, equivalent to an elastic k-value of about 108. The Interim Guide is based primarily on the performance of these sections.

From this information it appears that, for use with the Guide, an elastic k of 100 to 200 pci might be expected from good granular subbases about 6 in. thick, and an elastic k of 200 to 400 might be expected from stabilized material about 6 in. thick."

DISCUSSION

The following discussion is presented in the spirit of the Appendix C (2) statement, "...additional refinements should be made as new developments...become available."

***1** First, the term "Resilient Modulus" has been used interchangeably with two mathematical relationships, a) 1,500 x CBR, and b) as a function of Soil Support Value as included in the design equation shown on the nomograph of the 1986 AASHTO Guide (4) and Figure D2.

Second, the numerical value for resilient modulus is quoted as 10,000 psi or 15,000 psi in a confusing manner in both the main portion and Appendix HH of the 1986 AASHTO Guide (4) and in NCHRP Report 128 (2). As mentioned earlier, the scale labeled as "Kentucky CBR" in Figure D7 has an unknown origin and definitely is not in the reference given at the bottom of that Figure. The relationship between Soil Support Value and Kentucky CBR shown in Figure D2 was developed from results of the "round robin" laboratory tests of soil samples from the AASHO Road Test in the late 1960's (5). In reference 19, the main extended portion of the regression line for a range of CBRs from 2 to approximately 30 passed through CBR 100 at a Soil Support Value of 8.25 and this corresponds with test results determined by Utah and shown in Figure D12. Correlation of these separate test results suggests that Figure D2 may have more validity than D1. Also, the resilient modulus-soil support value relationships will be altered significantly if analyses suggested in ***2** are used.

***2** The assumption that crushed stone has a fixed value for "modulus" was tried Kentucky (6). The modulus relationship used as input value for elastic layer analyses using the Chevron N-layer computer program (7) was fixed at 25,000 psi. The relationship of 1,500 x CBR was used to assign subgrade moduli. For CBRs greater than 18, reasonable trends for lesser CBRs became very strange until it was realized that the moduli for the asphaltic concrete and subgrade were greater than

for the crushed stone intermediate layer. The following rationale is an over-simplification but is used to illustrate the problem. First, assume that the crushed stone material is placed between two thick steel plates, the crushed stone is restrained laterally to prevent lateral movement, the assembly is placed in a compression testing machine, and then loaded until the some of the crushed stone particles fracture. That load would be very high. Second, keep the same assembly and assumptions used in the first example except replace the bottom steel plate by a thick layer of gelatin, then load the sample. The sample will fail long before the maximum load for the first example can be obtained because the stone will penetrate the gelatin and the gelatin support will be lost. Similarly, the stiffness of the layer above the crushed stone layer also affects the load carrying and distributing capabilities of the stone layer. In summary, the load distributing characteristics, or modulus, is a direct function of the stiffness of the layer above and below the unbound crushed stone layer and as such will **NEVER** be a constant.
Kentucky (8) resolved the problem by analyzing various pavement structures where the thickness varied as a function of CBR, the proportion of the crushed stone layer was always 67 percent of the total thickness, and these structures were to carry the same design 18-kip EAL based on empirical results. The modulus of the asphaltic concrete was held constant and the modulus of the subgrade was assigned using the relationship of 1500 x CBR. The average CBR of 7 for Kentucky soils and the Benkelman beam measured deflection of 0.015 inches for a 21-inch pavement subjected to an 18-kip single axleload were the resultant benchmark values determined during a series of field tests conducted in 1957. By trial and error, a modulus of 27,500 psi was required to match the surface deflection of 0.015 inch (due to an 18-kip single axleload) for 21 inches and CBR 7 conditions, or a ratio of 2.8. The second criterion point was assumed to be a Bousinesq solution corresponding to a CBR 320 (a ratio of 1.0) which is the equivalent of 480,000 psi based on the 1500 x CBR relationship. A straight line connecting these two criterion points on a log-log plot formed the moduluar relationship between crushed stone and subgrade. The modulus of the crushed stone layer was varied according to this ratio-CBR relationship. Analyses were made for structures ranging from CBR 3 to 40 that corresponded to the same design EAL of 8 million 18-kip, 4-tired single axleload. The calculated vertical compressive strain was almost a constant value for these structures. Additional analyses (9) indicated that a slight modification in this straight line log-log relationship would be required so that these structures would behave according to strain-energy principals which is another way of saying that they would behave "according to classical physics".

***3** Figures D10 and D11 indicate that the Structural Number, SN, were calculated using values of 0.44 for the asphaltic concrete coefficient, a_1, and 0.14 for base material coefficient, a_2. Rationale would indicate that the value of 0.44 would have to be reduced for a reduction in elastic modulus from 600,000 psi to 150,000 psi. However, only one calculated SN is given and the text and figures do not indicate that another SN was calculated for the reduced asphaltic concrete modulus, or how the modification was made--if it was made. In any case, changing the moduli will change the values shown in Figures D10 and D11 significantly.

Unpublished preliminary analyses indicate that principals of work can be used to develop a variable relationship of structural coefficient for base material as a function of the thickness of subbase and vice versa. The key is the resulting work, or work strain, at the top of the subgrade due to the applied load at the top of the pavement structure. The SN for each pavement structure on Loops 3-6 of the AASHO Road Test was recalculated using the adjusted coefficient value as a function of the layer thicknesses. For the same number of 18-kip EALs, two plots were made of SN versus 18-kip EAL. The first used the traditional structural coefficient values of 0.44, 0.14, and 0.11 and the second used structural coefficient values that varied according to layer thicknesses. The scatter in SN for variable coefficient values was less than 15 percent of the scatter for the AASHTO recommended constant values. In summary, modulus is not a constant for any material and even if it is, it would vary effectively according to the other materials and their thicknesses used to make the pavement structure. To accomplish the task of defining these variations is not a simple assignment and would require tremendous effort.

*4 Appendix A, Report 61 E (4) contains the number of load repetitions for a number of flexible pavement structures at five fixed levels of serviceability. SN was calculated for each structure using the AASHTO recommended values for structural coefficients. The load repetitions were converted to 18-kip EALs according to the actual loads employed on each pavement section in that respective loop and for Loops 3-6 and single and tandem axleloads using the published AASHTO load equivalency factors. Regression analyses were performed using log-log straight-line and polynomial equations and the standard deviations were recorded. The R^2 coefficient value was highest for the straight-line equation form. Regressions were made for each of the five serviceability levels. The regression equation was plotted for a given serviceability level and percent reliability. The 1986 AASHTO design equation was evaluated for various values of SN, and respective axleload and level of serviceability and results were superimposed on the same figure. Figures D12 and D13 are two illustrations. Close inspection indicates the 1986 AASHTO equation yields a higher permissible fatigue value than the regression equation indicates the same SN at the Road Test could tolerate. Figures similar to D12 and D13 were made for levels of serviceability of 2.0, 2.5, 3.0, and 3.5 and for each percent reliability of 50, 80, 85, 90, and 95 percent. To summarize the results, the least discrepancy between the two equations occurred at the 2.5 serviceability level and 50 percent reliability. The discrepancy increased as the serviceability level was increased or decreased from the 2.50 level and as the percent reliability increased. As an example, a 20 million 18-kip EAL using the AASHTO design equation would require a SN for which the regression equation through the Road Test data would correspond to an 18-kip EAL of approximately 10 million. Another way of expressing it is that the regression equation through the Road Test data would require an additional value 1.0 more than the design equation at a design EAL of 20 million. Using 0.44 as the structural coefficient, the regression equation would require another 2.25 inches of asphaltic concrete.

*5 Figure D9 is a compilation of Figures D7 and D8. Footnote 3 of Figure D14 (Figure C-32 (2)) provides the reference (10) that contains the following quote from Page 151:

> "USE OF THE SOIL TEST BY AVIATION ENGINEER TROOPS
>
> ...the use of the soil constants derived from the tests would have to be accomplished principally by field idntification of the soils. For this reason, identification of soils has been stressed...and design curves have been included which give the range of bearing ratio and "k" values for various types of soils. Figure 2" (which has the same CBR-k scales shown in Figure D15) "shows the tentative design curves for total thickness of flexible pavements, on which there has been super-imposed Dr. Casagrande's new soil classification which is being taught the aviation engineer officers at Harvard. The Bureau of Public Road's classifcation is included only for the information of those who are familiar with this classification. You will note that the"k" values range from 100 for the fat clay to approximately 800 for an excellent well graded gravel. These values are considered only approximate, although to date some very good checks have been obtained."

As discussed earlier, Kentucky conducted CBR tests on a series of soils taken at the AASHO Road Test. The average CBR was determined to be approximately 5.3 to correspond with a Soil Support Value of 3.0. A regression analysis was made for CBR test data from Kentucky soils combined with the Road Test data and extrapolated to CBR 100 which corresponded to a Soil Support Value of 8.25--the value reported by Utah (3). Because there is correspondence of values from two sources, regression analyses were made between the CBR-k relationship shown in Figure D15 with the relationships of Soil Support Value and "Kentucky CBR" (Scales 1 and 10 in Figure D9) to position Scale 15. The following regression equations were used to establish the position of Scale 15.

$$k = 10^{(1.733958 + 0.568048(\log(CBR))}, \text{ and} \qquad \text{D-1}$$

$$k = 10^{(1.730783 + 0.138033(\text{Soil Support Value})}. \qquad \text{D-2}$$

Note that there is reasonably good agreement between Scales 14 and 15, especially for Soil Support Values up to approximately 5. Using the 1988 ACPA computer program, the design thicknesses for k values corresponding to a Soil Support Value of 5 differ by less than 0.20 inch. Closer inspection of Scales 14 and 15 shows that the k of Scale 15 is less than the k of Scale 14 which indicates that the

1.77 quoted above is insufficient to produce a scale comparable to Scale 15. Also, the ratio between Scales 13 and 15 is not a constant but is defined as:

$$k = 10^{(0.983924 + 1.163377(\log(Mr))}$$

where:

M_r = resilient modulus defined as:
$M_r = 10^{(2.997 + 0.160(SSV))}$, and
SSV = Soil Support Value.

Therefore, it is recommended that Equation D1 be used to adjust Kentucky CBR to a k-value (Scale 15, Figure D9) and D2 to adjust from Soil Support Value to a k-value (Scale 15, Figure D9) for use in the 1986 AASHTO Nomograph, or 1988 ACPA Computer Program, when comparing rigid pavement thickness designs.

LIST OF REFERENCES

1. Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1986.

2. C. J. Van Til, B. F. McCullough, B. A. Vallerga, and R. G. Hicks, "Evaluation of AASHO Interim Guide for Design of Pavement Structures", NCHRP Report 128, Highway Research Board, Washington, D. C. 1972.

3. 1972 AASHTO Interim Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1972 (1982 Printing).

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5. T. C. Hopkins, "Relationship between Soil Support Value and Kentucky CBR," Research Report 297, Division of Research, Kentucky Department of Highways, Lexington, KY, 1970.

6. H. F. Southgate, R. C. Deen, and J. H. Havens, "Rational Analysis of Kentucky Flexible Pavement Design Criterion", Research Report 293, Division of Research, Kentucky Department of Highways, Lexington, KY, 1968.

7. J. Michelow, "Analysis of Stresses and Displacements in an n-Layered Elastic System under a Load Uniformly Distributed on a Circular Area," unpublished, California Research Corporation, Chevron Oil Company, Richmond, CA, September 24, 1963.

8. R. C. Deen, H. F. Southgate, and J. H. Havens, "Structural Analysis of Bituminous Concrete Pavements", Research Report 305, Division of Research, Kentucky Department of Highways, Lexington, KY, 1971

9. H. F. Southgate, and R. C. Deen,"Flexible Pavement Designs Based on Work", Proceedings, Volume 57, Association of Asphalt Paving Technologists, St. Paul, MN, 1988.

10. T. A. Middlebrooks, and G. E. Bertram, "Soil Tests for Design of Runway Pavements", Proceedings, Highway Research Board, Washington, D. C., 1942.

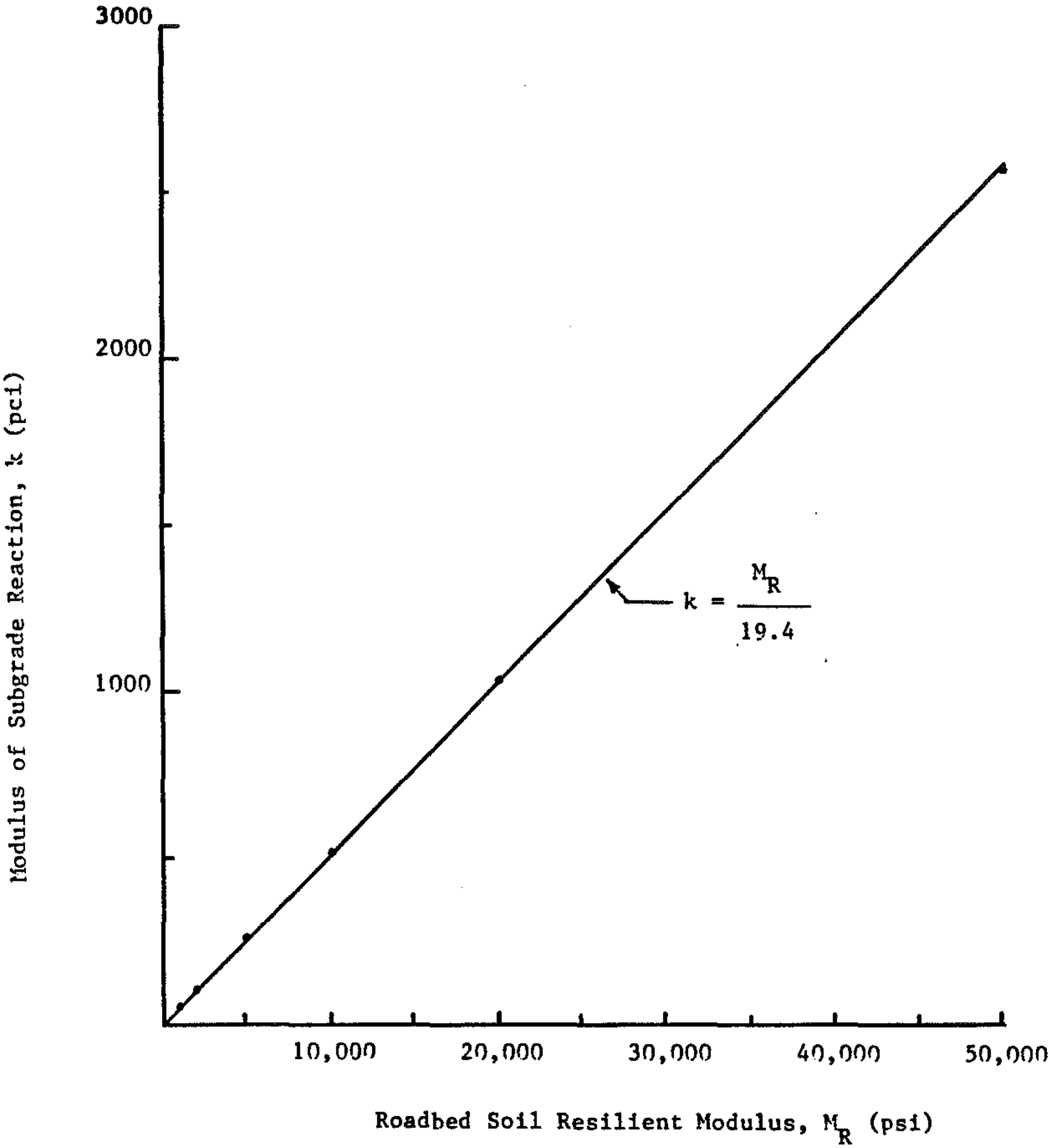


FIGURE D1. THEORETICAL RELATIONSHIP BETWEEN MODULUS OF SUBGRADE REACTION AND ROADBED SOIL RESILIENT MODULUS.

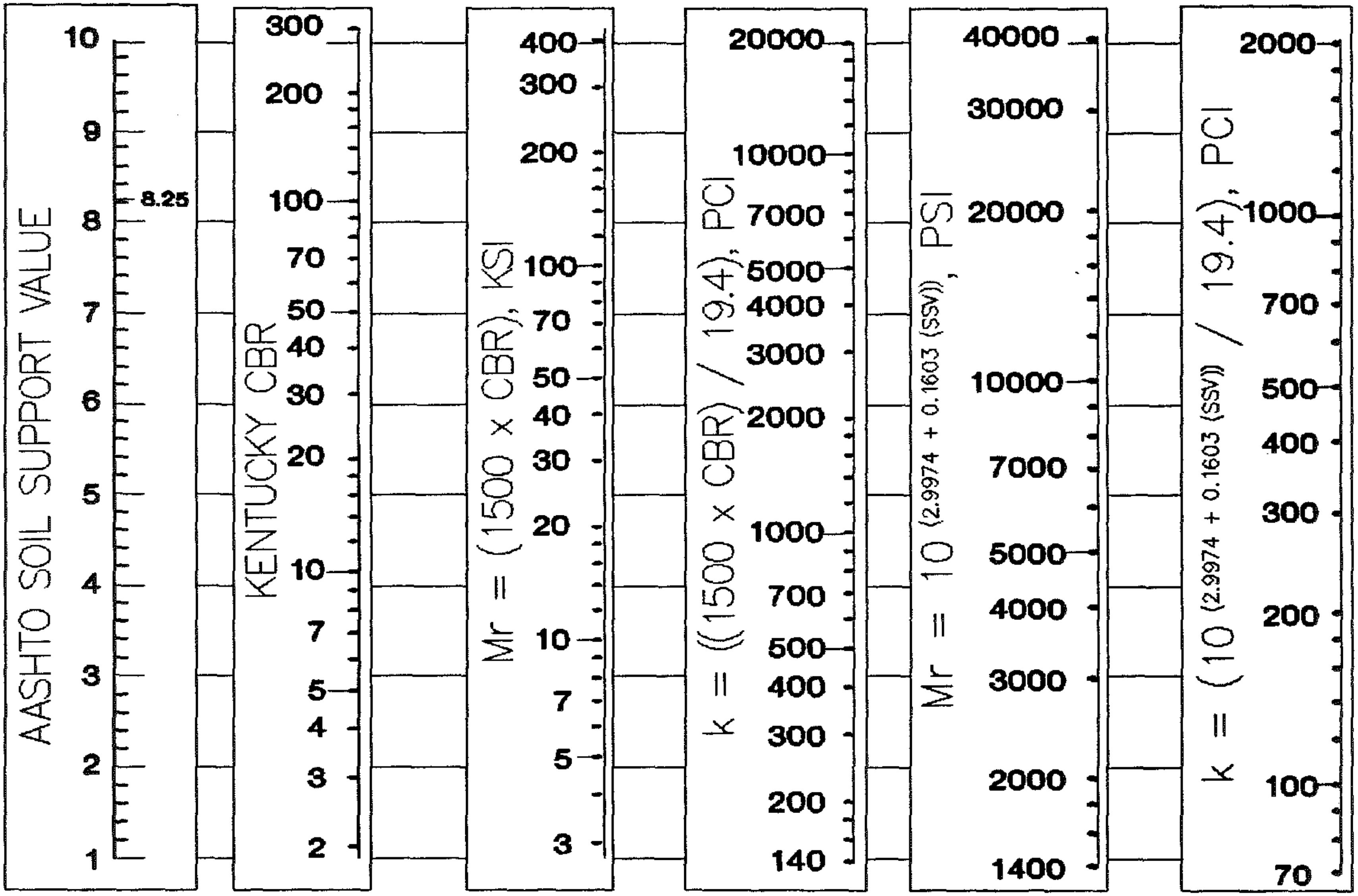


FIGURE D2. RELATIONSHIPS BETWEEN SOIL SUPPORT VALUE, KENTUCKY CBR, RESILIENT MODULUS, AND MODULUS OF SUBGRADE REACTION.

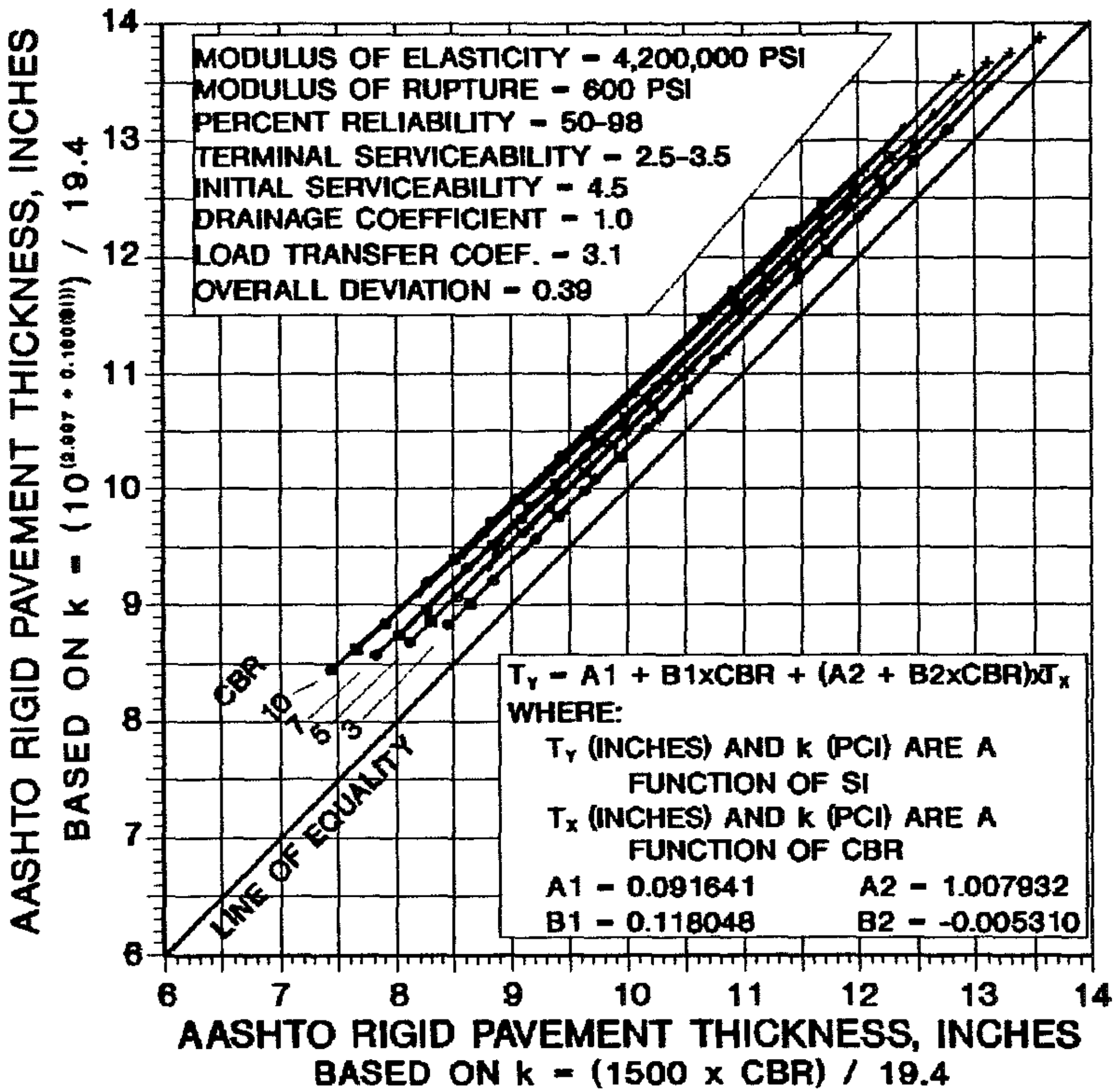


FIGURE D3. RELATIONSHIPS BETWEEN AASHTO RIGID PAVEMENT THICKNESSES AS A FUNCTION OF VALUES OF MODULUS OF SUBGRADE REACTION.

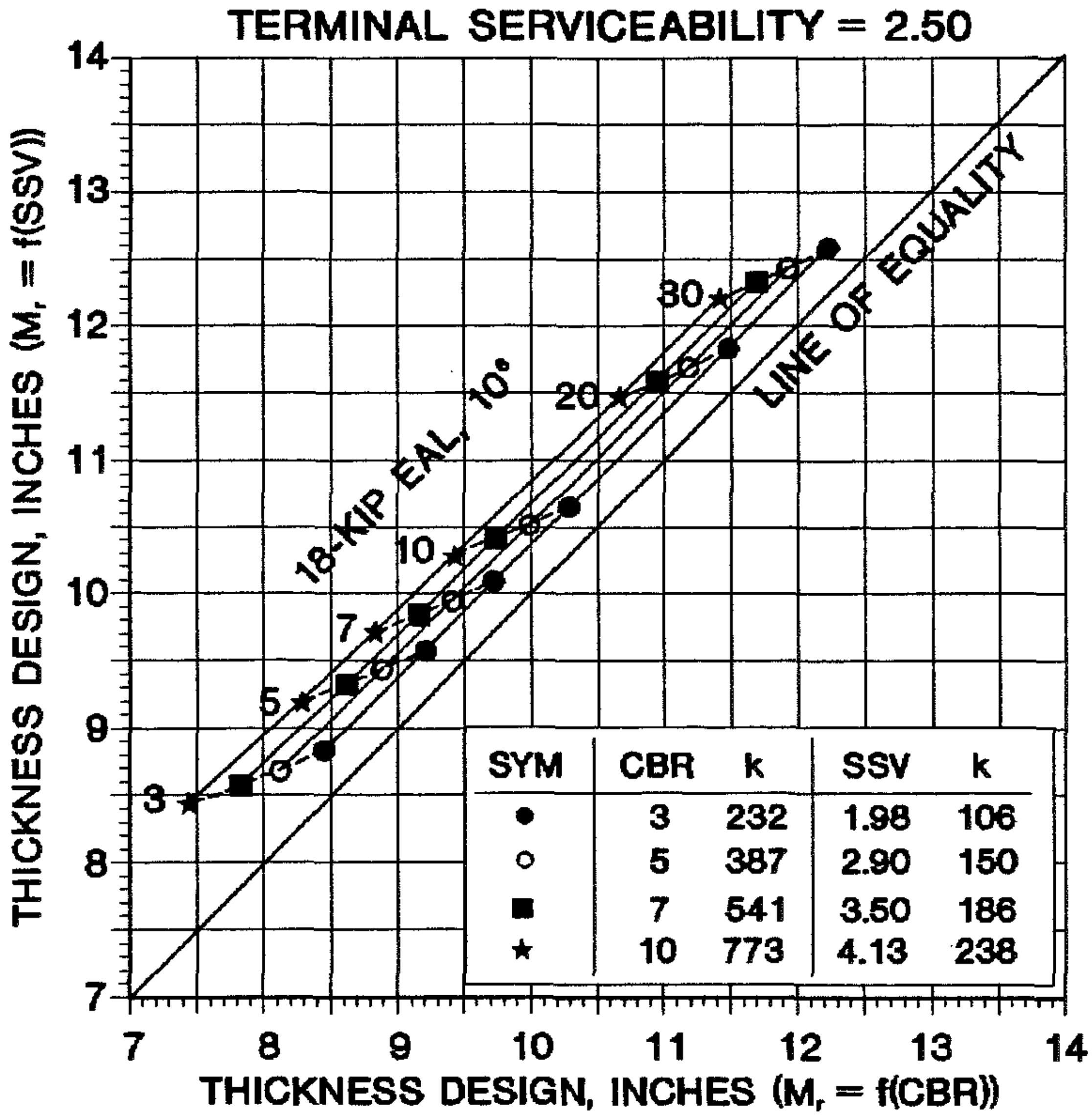


FIGURE D4. RELATIONSHIP OF RIGID PAVEMENT THICKNESS DESIGNS AT A 2.50 TERMINAL SERVICEABILITY LEVEL AND AS A FUNCTION OF VALUES OF RESILIENT MODULUS.

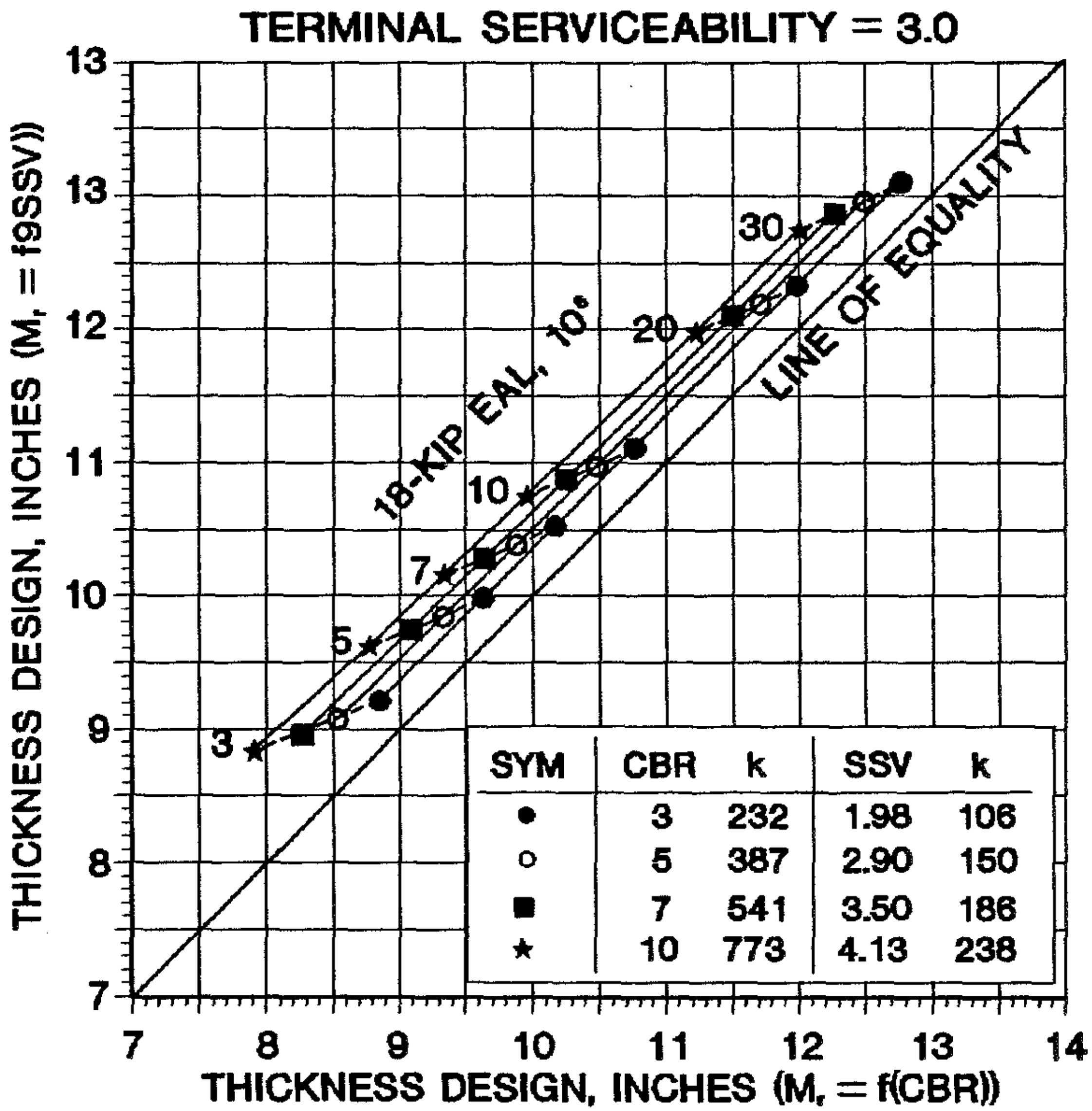


FIGURE D5. RELATIONSHIP OF RIGID PAVEMENT THICKNESS DESIGNS AT A 3.00 TERMINAL SERVICEABILITY LEVEL AND AS A FUNCTION OF VALUES OF RESILIENT MODULUS.

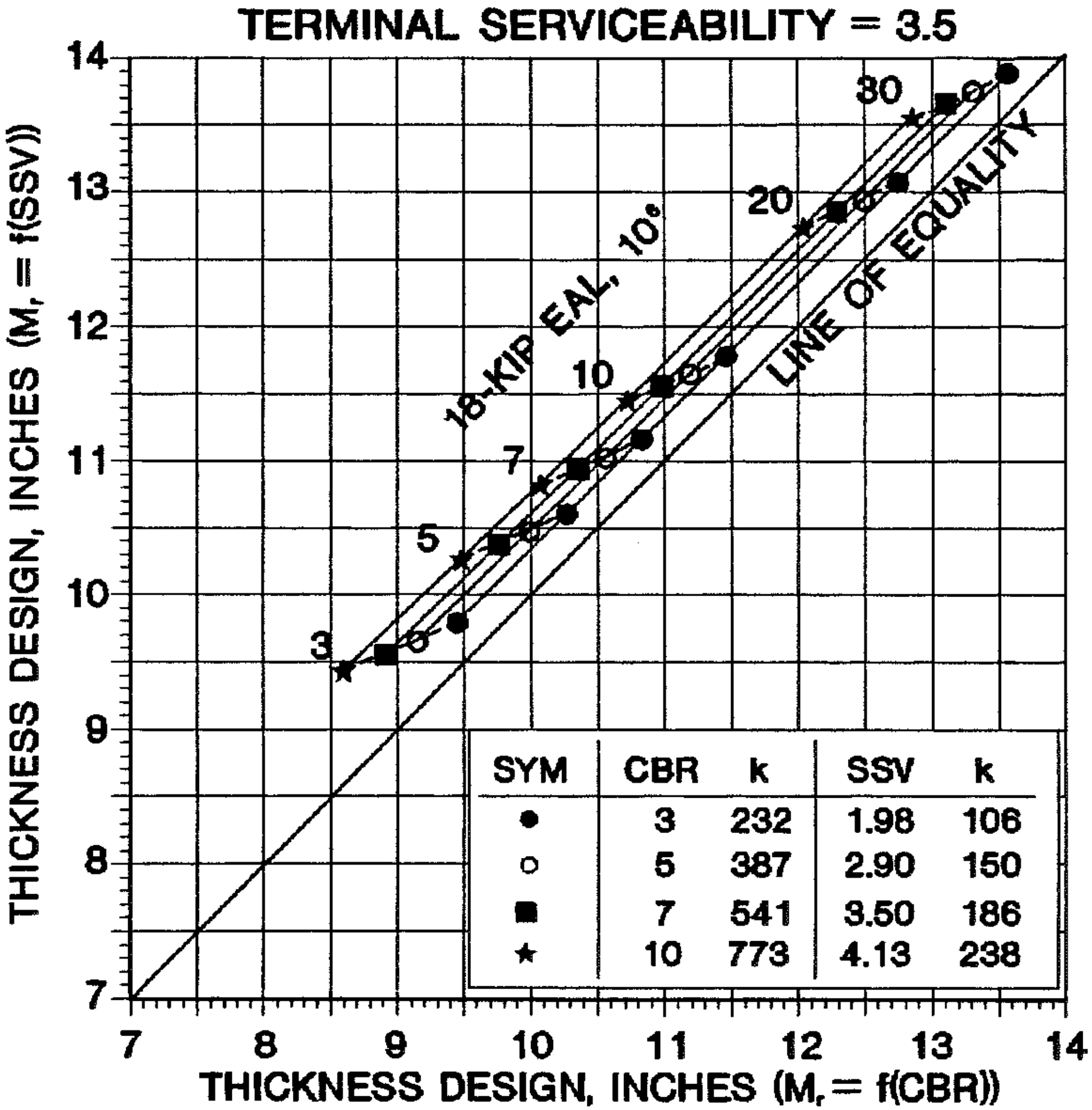
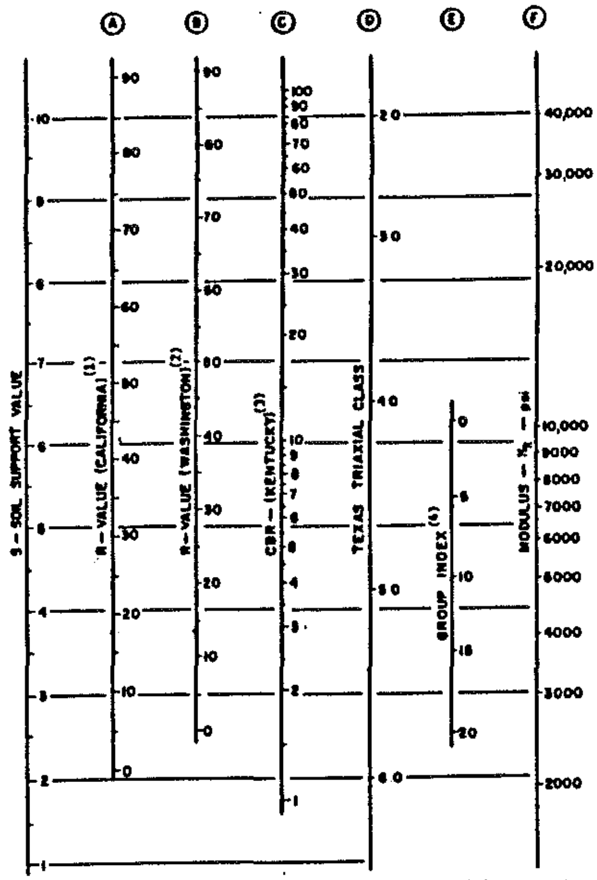


FIGURE D6. RELATIONSHIP OF RIGID PAVEMENT THICKNESS DESIGNS AT A 3.50 TERMINAL SERVICEABILITY LEVEL AND AS A FUNCTION OF VALUES OF RESILIENT MODULUS.



(1) The correlation is with the design curves used by California; AASHO designation is T-173-60, and exudation pressure is 240 psi. See Hveem, F.N., and Carmany, R.M., "The Factors Underlying the Rational Design of Pavements." Proc. HRB, Vol. 28 (1948) pp. 101-136.
(2) The correlation is with the design curves used by Washington Dept. of Highways; exudation pressure is 300 psi. See "Flexible Pavement Design Correlation Study." HRB Bull. 133 (1956).
(3) The correlation is with the CBR design curves developed by Kentucky. See Drake, W.B., and Havens, J.H., "Re-Evaluation of Kentucky Flexible Pavement Design Criterion." HRB Bull. 233 (1959) pp. 33-56. The following conditions apply to the laboratory-modified CBR: specimen is to be molded at or near the optimum moisture content as determined by AASHO T-99; dynamic compaction is to be used with a hammer weight of 10 lb dropped from a height of 18 in.; specimen is to be compacted in five equal layers with each layer receiving 10 blows; specimen is to be soaked for 4 days.
(4) This scale has been developed by comparison between the California R-value and the Group Index determined by the procedure in Proc. HRB Vol. 25 (1945) pp. 376-392.

FIGURE D7. CORRELATION CHART FOR ESTIMATING SOIL SUPPORT (1).

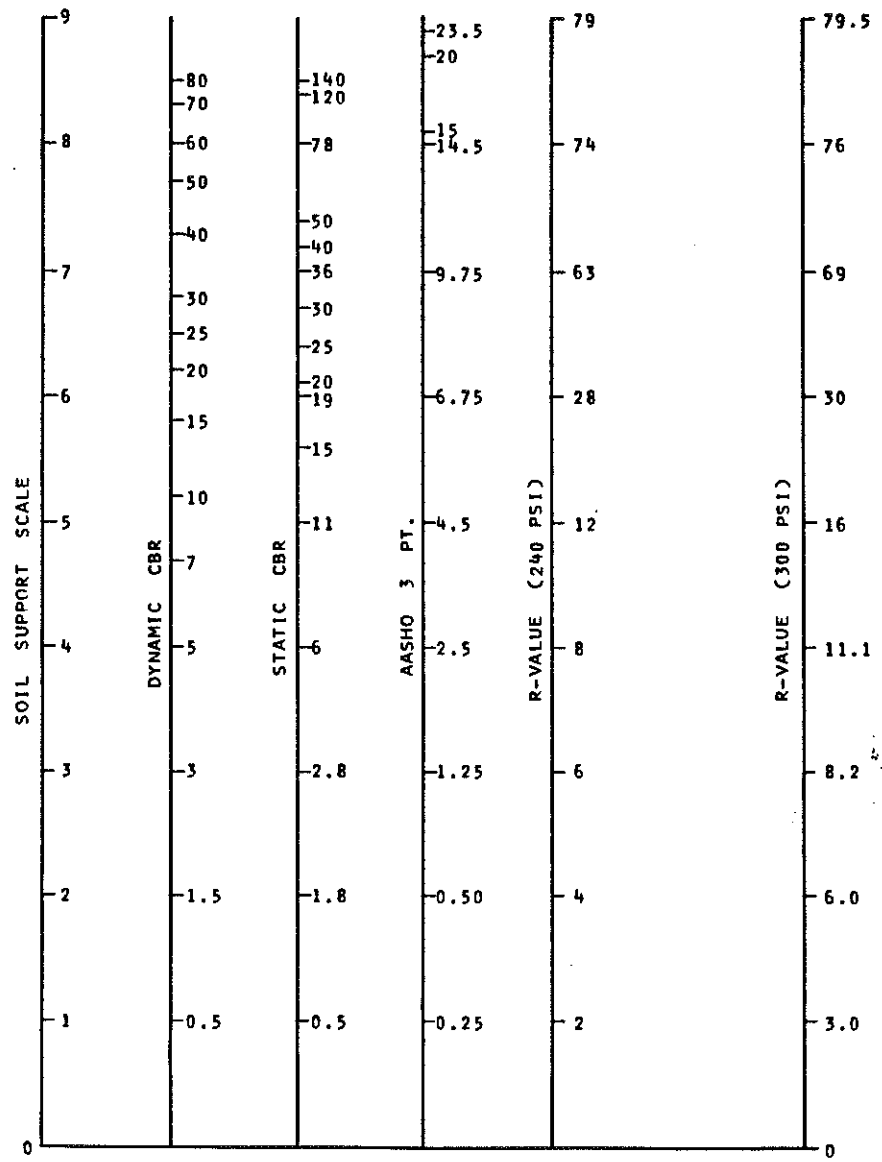


FIGURE D8. TEST VALUES AS DETERMINED FROM THE GRAPHS IN FIGURES C.3-2 THROUGH C.3-5, 1972 AASHTO INTERIM GUIDE.

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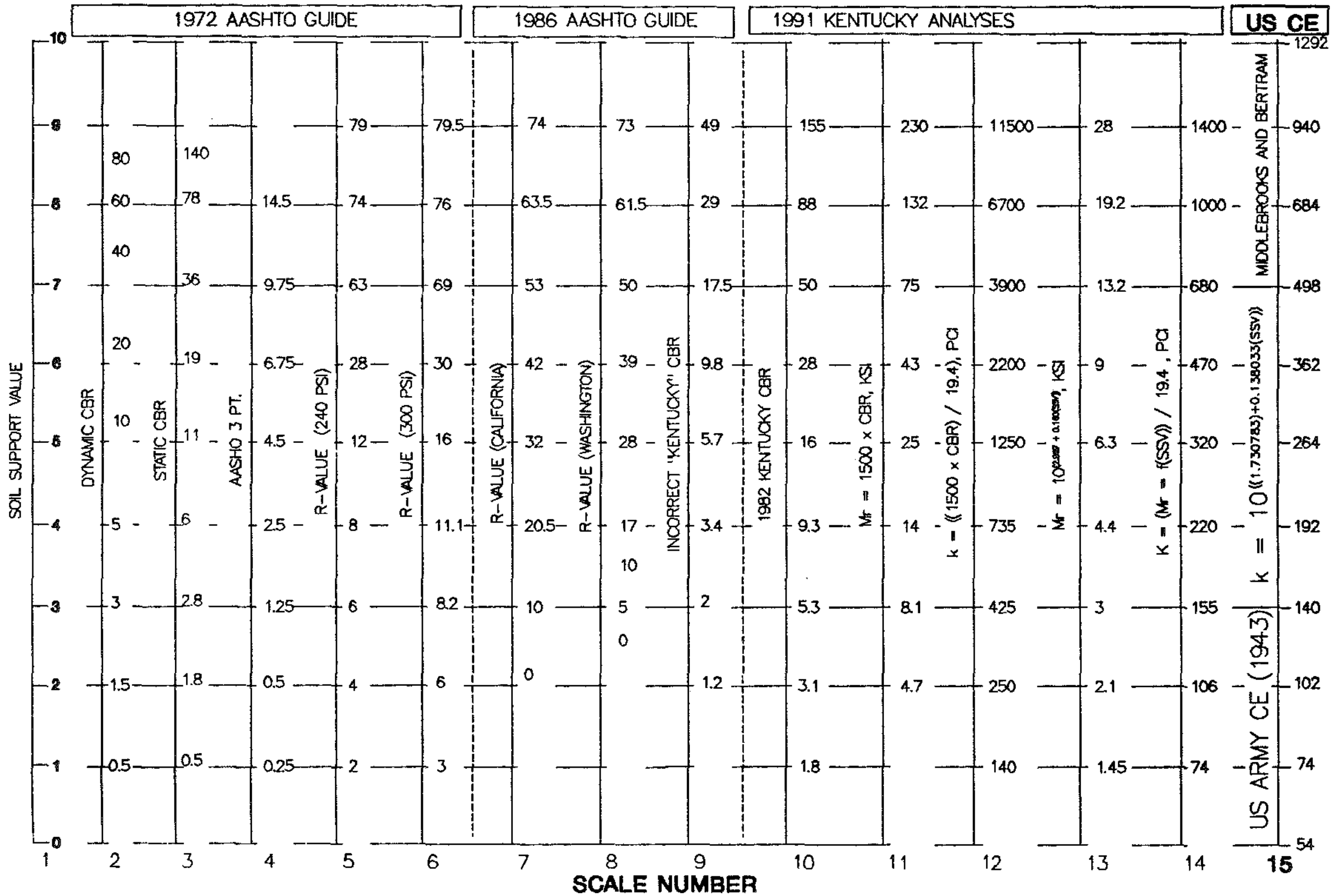


FIGURE D9. RELATIONSHIPS BETWEEN SOIL SUPPORT VALUE, R-VALUE, CBR, RESILIENT MODULUS, MODULUS OF SUBGRADE REACTION.

Thickness Layer 1	Thickness Layer 2							SN
Modulus-Layer 2		15000						
Modulus-Layer 1		150000			600000			
Modulus-Layer 3		3×10^3	7.5×10^3	15×10^3	3×10^3	7.5×10^3	15×10^3	
	1.5	.0011758	.0008289	.0006381	.0005585	.0004175	.0003314	1.5
3	14.0	.0006290	.0006282	.0006272	.0003578	.0003402	.0003287	3
	22.3	.0006118	.0006212	.0006263	.0003357	.0003314	.0003283	4
	30.7	.0006103	.0006205	.0006261	.0003287	.0003285	.0003281	5
	39.0	.0006112	.0006207	.0006259	.0003262	.0003275	.0003280	6
4	2.0	.0009054	.0006760	.0005458	.0004017	.0003133	.0002564	2
	10.3	.0005883	.0005580	.0005397	.0003056	.0002753	.0002576	3
	18.7	.0005402	.0005395	.0005358	.0002728	.0002627	.0002559	4
	27.0	.0005303	.0005355	.0005380	.0002615	.0002581	.0002557	5
	35.3	.0005281	.0005346	.0005378	.0002570	.0002562	.0002555	6
5	6.7	.0005671	.0004978	.0004577	.0002674	.0002288	.0002045	3
	15.0	.0004774	.0004644	.0004559	.0002294	.0002141	.0002039	4
	23.3	.0004565	.0004561	.0004554	.0002146	.0002081	.0002037	5
	31.7	.0004502	.0004536	.0004552	.0002081	.0002053	.0002036	6
6	3.0	.0005793	.0004595	.0003888	.0002350	.0001943	.0001663	3
	11.3	.0004321	.0004035	.0003859	.0001979	.0001784	.0001657	4
	19.7	.0003963	.0003897	.0003851	.0001806	.0001714	.0001654	5
	28.0	.0003852	.0003853	.0003848	.0001727	.0001682	.0001654	6
8	12.3	.0003180	.0002951	.0002808	.0001358	.0001226	.0001143	5
	20.7	.0002931	.0002854	.0002803	.0001263	.0001187	.0001141	6
10	5.0	.0002856	.0002409	.0002122	.0001076	.0000926	.0000828	5
	13.3	.0002402	.0002224	.0002113	.0000986	.0000885	.0000827	6

Figure C-19. Summary of calculations for tensile strain in the bottom fiber of the asphaltic concrete (response due to both tires).

FIGURE D10. SUMMARY OF CALCULATIONS FOR TENSILE STRAIN IN THE BOTTOM FIBER OF THE ASPHALTIC CONCRETE (2).

Thickness Layer 1	Thickness Layer 2	Modulus-Layer 2: 15000						SN
		Modulus-Layer 1: 150000			600000			
		Modulus-Layer 3: 3×10^3	7.5×10^3	15×10^3	3×10^3	7.5×10^3	15×10^3	
3	1.5	.004560	.002612	.001594	.002456	.001743	.000936	1.5
	14.0	.001236	.000752	.000480	.001043	.000641	.000399	3
	22.3	.000706	.000433	.000280	.000624	.000387	.000252	4
	30.7	.000459	.000280	.000180	.000412	.000255	.000166	5
	39.0	.000323	.000196	.000125	.000290	.000180	.000116	6
4	2.0	.003202	.000089	+.000036	.001650	.001021	.000655	2
	10.3	.001457	.000877	.000541	.001063	.000648	.000391	3
	18.7	.000793	.000528	.000316	.000660	.000429	.000257	4
	27.0	.000492	.000300	.000194	.000428	.000263	.000170	5
	35.3	.000351	.000214	.000137	.000310	.000191	.000123	6
5	6.7	.001689	.000994	.000603	.001013	.000617	.000383	3
	15.0	.000892	.000547	.000348	.000673	.000411	.000256	4
	23.3	.000559	.000342	.000222	.000459	.000282	.000178	5
	31.7	.000381	.000232	.000150	.000325	.000199	.000127	6
6	3.0	.001838	.001109	.000694	.000879	.000571	.000376	3
	11.3	.000999	.000611	.000381	.000650	.000400	.000251	4
	19.7	.000611	.000374	.000241	.000461	.000281	.000176	5
	28.0	.000415	.000253	.000163	.000321	.000204	.000128	6
8	12.3	.000726	.000447	.000281	.000432	.000267	.000170	5
	20.7	.000482	.000294	.000187	.000332	.000199	.000125	6
10	5.0	.000820	.000521	.000377	.000376	.000246	.000167	5
	13.3	.000549	.000339	.000215	.000312	.000190	.000122	6

Figure C-20. Summary of calculations for vertical compressive strain on the subgrade (response due to both tires).

FIGURE D11. SUMMARY OF CALCULATIONS FOR VERTICAL COMPRESSIVE STAIN ON THE SUBGRADE(2).

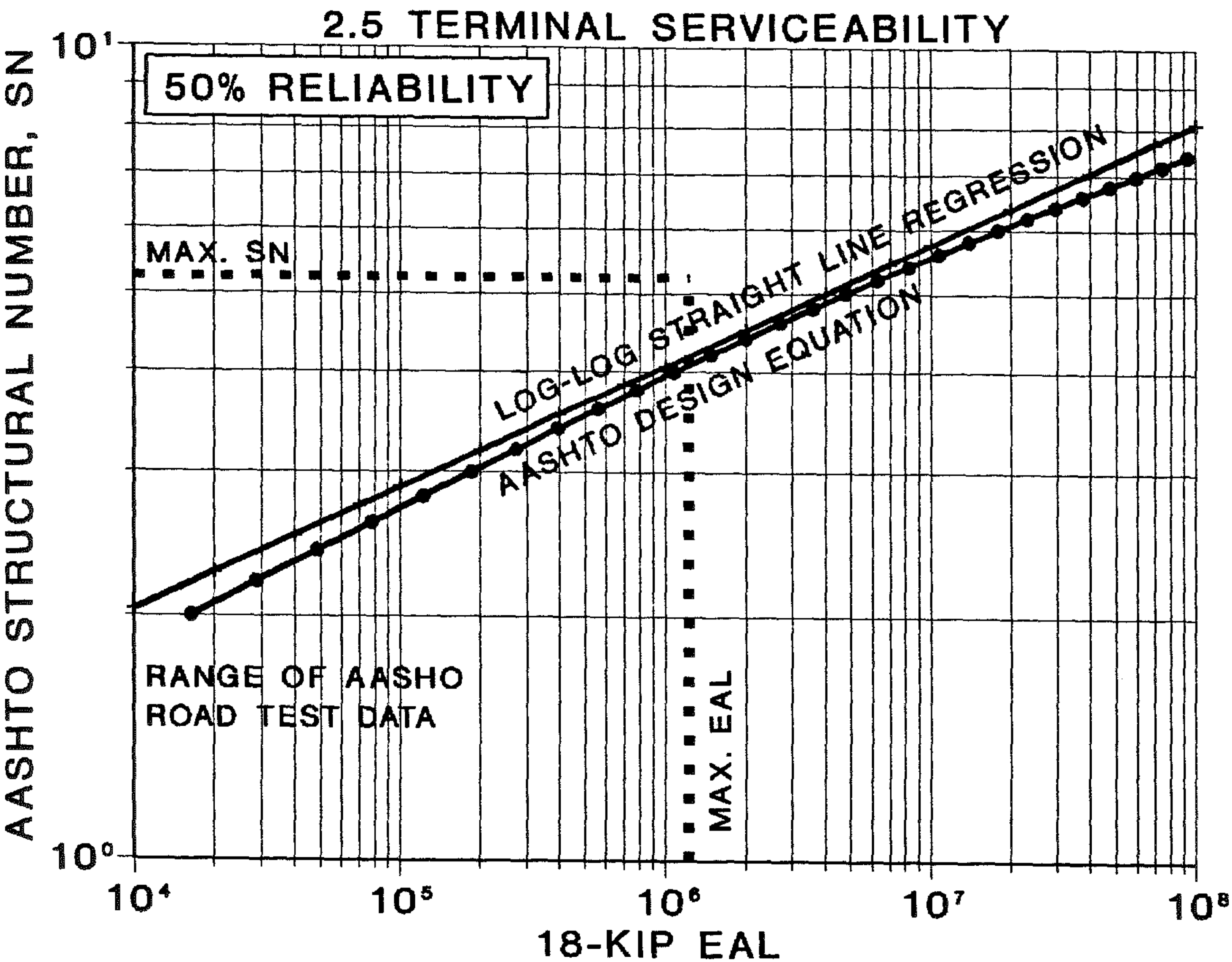


FIGURE D12. RELATIONSHIP OF 18-KIP EAL AND AASHTO STRUCTURAL NUMBER FOR 50 PERCENT RELIABILITY AND 2.50 TERMINAL SERVICEABILITY.

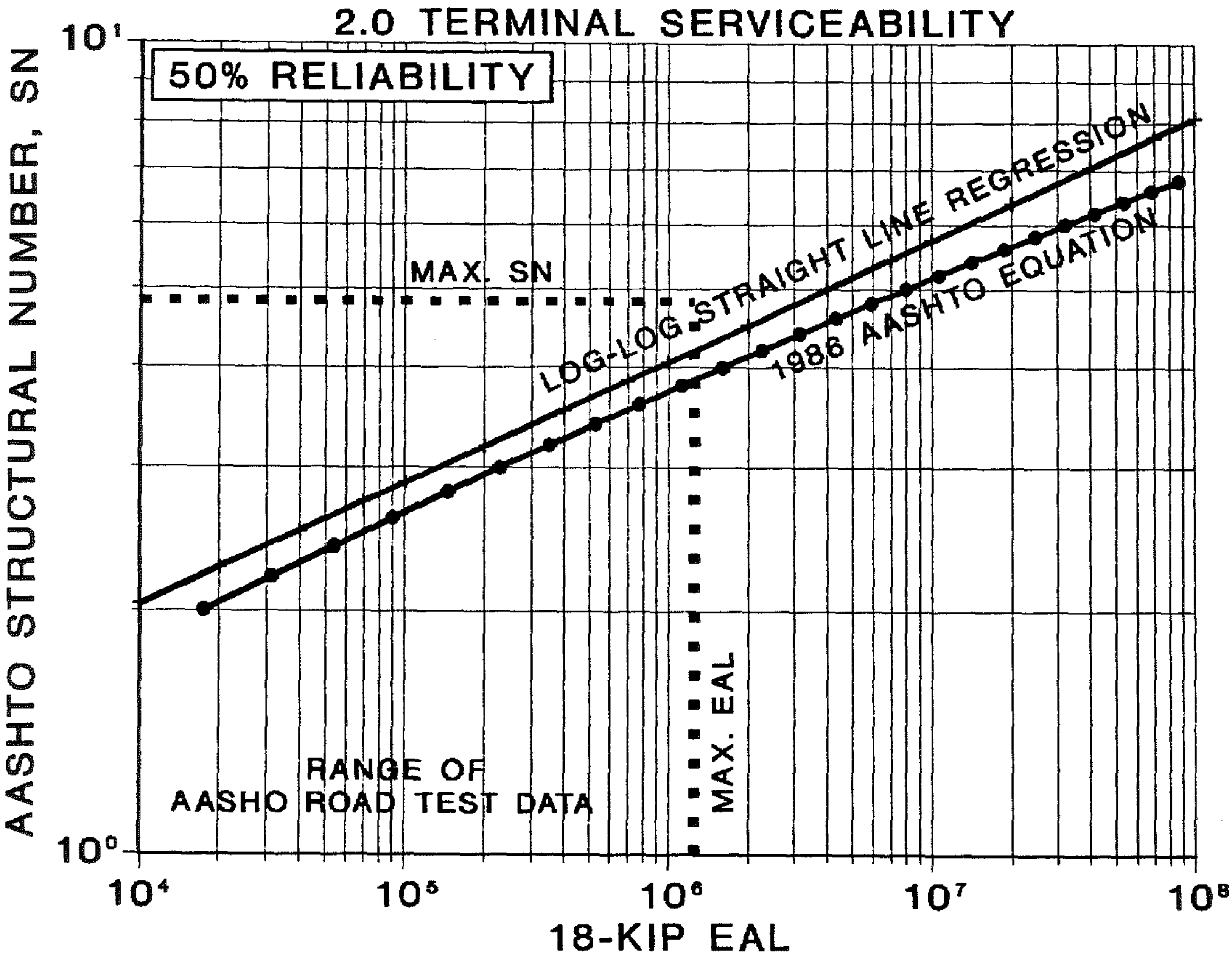
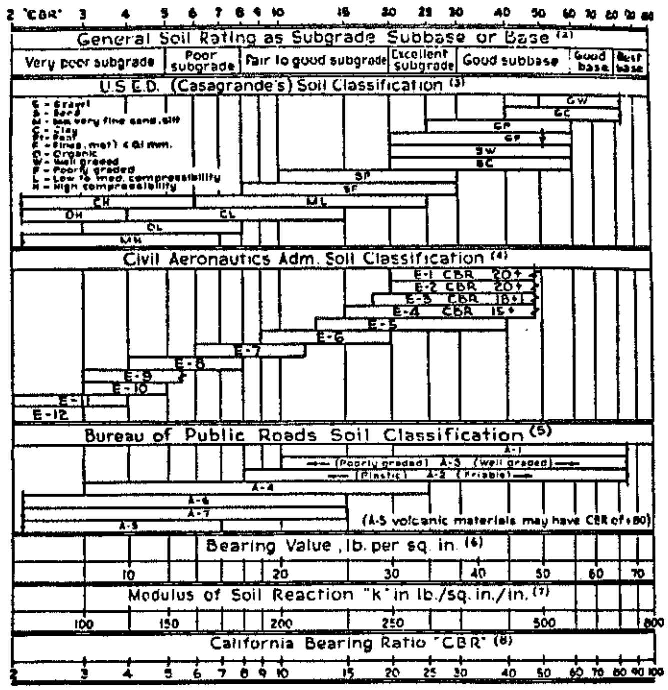


FIGURE D13. RELATIONSHIP OF 18-KIP EAL AND AASHTO STRUCTURAL NUMBER FOR 50 PERCENT RELIABILITY AND 2.00 TERMINAL SERVICEABILITY.



(1) All interrelationships are approximate. Actual tests are required to determine CBR, k, etc.
(2) For basic idea, see Porter, O.J., "Foundations for Flexible Pavements." Proc. HRB, Vol. 22 (1942) pp. 100-136.
(3) Relationships from Engineering Manual. Chap.XX, Office of the Chief of Engineers, War Dept. (March 1943). See also Middlebrooks, T.A., and Bertram, G.E., "Soil Tests for Design of Runway Pavements." Proc. HRB, Vol. 22 (1942) pp. 144-173.
(4) CBR values estimated from old CAA classification system.
(5) See Engineering Manual. A-6 and A-7 soils cut off at CBR 15 rather than at 25 as indicated in Chap. XX.
(6) See Middlebrooks and Bertram, p. 184. Bearing values at 0.1 in. deflection; bearing area not given.
(7) See item (3). k is factor used in Westergaard's Analysis for thickness of portland cement concrete pavement.
(8) See items (2) and (3).

FIGURE D14. INTERRELATIONSHIPS OF SOIL CLASSIFICATION, CALIFORNIA BEARING RATIOS, BEARING VALUES, AND k-VALUES (2).

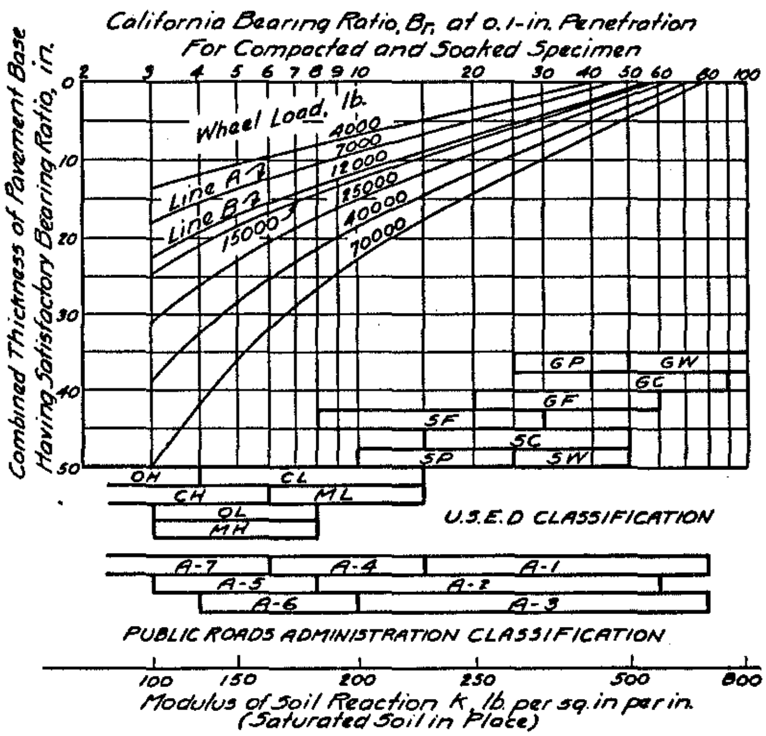


FIGURE D15. CURVES FOR DESIGN OF FOUNDATIONS FOR FLEXIBLE PAVEMENTS (10).

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APPENDIX E

LOAD EQUIVALENCIES

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Comparison of thickness designs furnished in Tables 2 and 3 and illustrated in Figures 5-10 indicates the possibility that some of the discrepancies might be a result of differences between Kentucky's and AASHTO's load equivalency factors. Figures E1-E3 are tables of load equivalency factors for a terminal serviceability level of 2.5 included in the 1986 AASHTO Guide (1). The pattern of load equivalency values indicates that factors are greater for thin and thick pavements than for some intermediate thicknesses. Close inspection indicates that the minimum values vary as a function of load and pavement thickness. The pattern is the same for single, tandem, and triaxle configurations, but the combinations of load and thickness where the minimum value changes to another thickness varies according to the number of axles. The minimum values have been enclosed by rectangles to illustrate the relationship.

Computer solutions for a matrix of loads, axles, and pavement thicknesses were obtained during the late 1970's to study the effects of loads on rigid pavements. Load equivalency factors were not developed fully at that time. Computer solutions were retrieved and load equivalencies were developed for rigid pavements in the same manner as for flexible pavements. In Figure E4, the portion of the curve labeled as "PCA" (up to approximately 2 million EALs) is a straight line of the format logX,Y and greater than 2 million is the format logX,logY. To minimize computer programming problems, a log-log polynomial equation was fitted to Figure E4 and the calculated load equivalencies were based upon the resulting relationship shown in Figure E5. Figures E6-E8 illustrate the relationship for rigid pavement load equivalencies for Kentucky and 1986 AASHTO design methods. Note that Figure E8 is for a two-tired single axle for which the 1986 AASHTO Guide does not contain a set of load equivalencies. The Kentucky analyses indicate the relationships are a function of subgrade as well as load as shown in Figures E6-E8. Load equivalencies provided in the 1986 AASHTO Guide (1) vary as a function of pavement thickness but not for changes in subgrade moduli.

The Kentucky set of load equivalencies shown in Figures E6-E8 are those expressed as a function of work at the bottom of a 10-inch rigid pavement and are based on the relationship of work vs repetitions shown in Figure E4. Chevron computer solutions of the 1970's mentioned earlier were used to determine the relationship between work at the top of the subgrade and work at the bottom of the concrete. Figure E9 shows these relationships for a four-tired single axle and for three subgrade stiffnesses. The resulting log-log polynomial equation fitted to Figure E4 is shown as the top curve in Figure E10. Figure E9 was used to determine the equivalent work at the top of the subgrade as a function of the work at the bottom of the concrete for the same fatigue level. These three sets of data were subjected to regression analyses of work vs repetitions and the resultant curves at the bottom of Figure E10 are the relationships of log-log regression analyses. Load equivalency relationships for a four-tired single axleload were calculated based on work at the top of the subgrade resulted in the load equivalencies shown in Figure E11. Figures

E12 and E13 are similar to Figure E9. Figure E14 is a combination of the relationships for a two-tired steering axle, a four-tired single axle, and an eight-tired tandem axle arrangements. Note that Figures E6 and E11 are nearly identical, but the load equivalency factors are based on behavior at two different locations--bottom of the concrete, or top of the subgrade. Does this infer that the crushed stone base is useful primarily as a construction platform and does not have any significant use related to pavement fatigue?

The W4 Tables for loadometer data contain the AASHTO set of load equivalencies of SN=5 for flexible pavements and 9-inch concrete pavements. These sets of load equivalencies were used to calculate the fatigue for each weight group and then summed to obtain the total EALs for the weighed axles. It was noticed that the totals were not equal for each type of pavement, the total for the rigid pavement was always greater than for the flexible pavement, and that the ratio of rigid to flexible was not constant. Because the two totals are not equal, it can be stated that the structures chosen for comparison were not equal in fatigue. Nevertheless, the available data representing the total of 11 loadometer stations were collected starting with 1959 and the ratios are shown in Figure E15. Note that the ratio is increasing with calendar year and is thought to be a reflection of increasing axleloads with time.

To determine the magnitude of differences between the load equivalency relationships, the 1989 Weigh-In-Motion data file consisting of 34,025 trucks in seven vehicle classifications was analyzed using the Kentucky flexible factors, AASHTO rigid factors for 6-, 8-, and 10-inch pavements, and Kentucky rigid pavement factors for 6-, 8-, and 10-inch pavements on each of Kentucky CBRs 3, 7.5, and 15 (elastic moduli of 4.5, 11.5, and 22.5 ksi, respectively). Results are shown in Tables E1-E3 for rigid pavement thicknesses of 6, 8, and 10 inches, respectively. To determine the pattern of load equivalencies, the ratio of the total EALs for Kentucky rigid pavement to the Kentucky flexible pavement was calculated for each thickness and CBR and shown in Figure E16. Figure E17 illustrates the ratios of EALs for AASHTO rigid pavements to calculated EALs using Kentucky flexible pavement relationships. Table E4 contains the calculated ratios shown in Figures E16 and E17.

The ACPA computer program (2) based upon the 1986 AASHTO Guide was used to determine the concrete pavement design thicknesses for the adjusted AASHTO EALs shown in Table E4. If the 1986 AASHTO and Kentucky load equivalencies were identical, the combination of a given EAL and CBR would correspond to the same thickness obtained from both design systems. Table E5 contains the AASHTO rigid pavement design thicknesses at four percent reliabilities for the adjusted AASHTO EALs given in Table E4. Therefore, differences in load equivalency relationships have definite influences upon thickness designs between the two design methods.

LIST OF REFERENCES

1. Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D. C., 1986.

2. Simplified Guide for the Design of Concrete Pavements," American Concrete Pavement Association, Arlington Heights, IL, 1988.

Table D.13. Axle load equivalency factors for rigid pavements, single axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
4	.003	.002	.002	.002	.002	.002	.002	.002	.002
6	.012	.011	.010	.010	.010	.010	.010	.010	.010
8	.039	.035	.033	.032	.032	.032	.032	.032	.032
10	.097	.089	.084	.082	.081	.080	.080	.080	.080
12	.203	.189	.181	.176	.175	.174	.174	.173	.173
14	.376	.360	.347	.341	.338	.337	.336	.336	.336
16	.634	.623	.610	.604	.601	.599	.599	.599	.598
18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.51	1.52	1.55	1.57	1.58	1.58	1.59	1.59	1.59
22	2.21	2.20	2.28	2.34	2.38	2.40	2.41	2.41	2.41
24	3.16	3.10	3.22	3.36	3.45	3.50	3.53	3.54	3.55
26	4.41	4.26	4.42	4.67	4.85	4.95	5.01	5.04	5.05
28	6.05	5.76	5.92	6.29	6.61	6.81	6.92	6.98	7.01
30	8.16	7.67	7.79	8.28	8.79	9.14	9.35	9.46	9.52
32	10.8	10.1	10.1	10.7	11.4	12.0	12.3	12.6	12.7
34	14.1	13.0	12.9	13.6	14.6	15.4	16.0	16.4	16.5
36	18.2	16.7	16.4	17.1	18.3	19.5	20.4	21.0	21.3
38	23.1	21.1	20.6	21.3	22.7	24.3	25.6	26.4	27.0
40	29.1	26.5	25.7	26.3	27.9	29.9	31.6	32.9	33.7
42	36.2	32.9	31.7	32.2	34.0	36.3	38.7	40.4	41.6
44	44.6	40.4	38.8	39.2	41.0	43.8	46.7	49.1	50.8
46	54.5	49.3	47.1	47.3	49.2	52.3	55.9	59.0	61.4
48	66.1	59.7	56.9	56.8	58.7	62.1	66.3	70.3	73.4
50	79.4	71.7	68.2	67.8	69.6	73.3	78.1	83.0	87.1

FIGURE E1. 1986 AASHTO AXLELOAD EQUIVALENCY FACTORS FOR RIGID PAVEMENTS, SINGLE AXLES AND P_t OF 2.5 (1).

Table D.14. Axle load equivalency factors for rigid pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0006	.0006	.0005	.0005	.0005	.0005	.0005	.0005	.0005
6	.002	.002	.002	.002	.002	.002	.002	.002	.002
8	.007	.006	.006	.005	.005	.005	.005	.005	.005
10	.015	.014	.013	.013	.012	.012	.012	.012	.012
12	.031	.028	.026	.026	.025	.025	.025	.025	.025
14	.057	.052	.049	.048	.047	.047	.047	.047	.047
16	.097	.089	.084	.082	.081	.081	.080	.080	.080
18	.155	.143	.136	.133	.132	.131	.131	.131	.131
20	.234	.220	.211	.206	.204	.203	.203	.203	.203
22	.340	.325	.313	.308	.305	.304	.303	.303	.303
24	.475	.462	.450	.444	.441	.440	.439	.439	.439
26	.644	.637	.627	.622	.620	.619	.618	.618	.618
28	.855	.854	.852	.850	.850	.850	.849	.849	.849
30	1.11	1.12	1.13	1.14	1.14	1.14	1.14	1.14	1.14
32	1.43	1.44	1.47	1.49	1.50	1.51	1.51	1.51	1.51
34	1.82	1.82	1.87	1.92	1.95	1.96	1.97	1.97	1.97
36	2.29	2.27	2.35	2.43	2.48	2.51	2.52	2.52	2.53
38	2.85	2.80	2.91	3.03	3.12	3.16	3.18	3.20	3.20
40	3.52	3.42	3.55	3.74	3.87	3.94	3.98	4.00	4.01
42	4.32	4.16	4.30	4.55	4.74	4.86	4.91	4.95	4.96
44	5.26	5.01	5.16	5.48	5.75	5.92	6.01	6.06	6.09
46	6.36	6.01	6.14	6.53	6.90	7.14	7.28	7.36	7.40
48	7.64	7.16	7.27	7.73	8.21	8.55	8.75	8.86	8.92
50	9.11	8.50	8.55	9.07	9.68	10.14	10.42	10.58	10.66
52	10.8	10.0	10.0	10.6	11.3	11.9	12.3	12.5	12.7
54	12.8	11.8	11.7	12.3	13.2	13.9	14.5	14.8	14.9
56	15.0	13.8	13.6	14.2	15.2	16.2	16.8	17.3	17.5
58	17.5	16.0	15.7	16.3	17.5	18.6	19.5	20.1	20.4
60	20.3	18.5	18.1	18.7	20.0	21.4	22.5	23.2	23.6
62	23.5	21.4	20.8	21.4	22.8	24.4	25.7	26.7	27.3
64	27.0	24.6	23.8	24.4	25.8	27.7	29.3	30.5	31.3
66	31.0	28.1	27.1	27.6	29.2	31.3	33.2	34.7	35.7
68	35.4	32.1	30.9	31.3	32.9	35.2	37.5	39.3	40.5
70	40.3	36.5	35.0	35.3	37.0	39.5	42.1	44.3	45.9
72	45.7	41.4	39.6	39.8	41.5	44.2	47.2	49.8	51.7
74	51.7	46.7	44.6	44.7	46.4	49.3	52.7	55.7	58.0
76	58.3	52.6	50.2	50.1	51.8	54.9	58.6	62.1	64.8
78	65.5	59.1	56.3	56.1	57.7	60.9	65.0	69.0	72.3
80	73.4	66.2	62.9	62.5	64.2	67.5	71.9	76.4	80.2
82	82.0	73.9	70.2	69.6	71.2	74.7	79.4	84.4	88.8
84	91.4	82.4	78.1	77.3	78.9	82.4	87.4	93.0	98.1
86	102.	92.	87.	86.	87.	91.	96.	102.	108.
88	113.	102.	96.	95.	96.	100.	105.	112.	119.
90	125.	112.	106.	105.	106.	110.	115.	123.	130.

FIGURE E2. 1986 AASHTO AXLELOAD EQUIVALENCY FACTORS FOR RIGID PAVEMENTS, TANDEM AXLES AND P_t OF 2.5 (1).

Table D.15. Axle load equivalency factors for rigid pavements, triple axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003
6	.001	.001	.001	.001	.001	.001	.001	.001	.001
8	.003	.002	.002	.002	.002	.002	.002	.002	.002
10	.006	.005	.005	.005	.005	.005	.005	.005	.005
12	.011	.010	.010	.009	.009	.009	.009	.009	.009
14	.020	.018	.017	.017	.016	.016	.016	.016	.016
16	.033	.030	.029	.028	.027	.027	.027	.027	.027
18	.053	.048	.045	.044	.044	.043	.043	.043	.043
20	.080	.073	.069	.067	.066	.066	.066	.066	.066
22	.116	.107	.101	.099	.098	.097	.097	.097	.097
24	.163	.151	.144	.141	.139	.139	.138	.138	.138
26	.222	.209	.200	.195	.194	.193	.192	.192	.192
28	.295	.281	.271	.265	.263	.262	.262	.262	.262
30	.384	.371	.359	.354	.351	.350	.349	.349	.349
32	.490	.480	.468	.463	.460	.459	.458	.458	.458
34	.616	.609	.601	.596	.594	.593	.592	.592	.592
36	.765	.762	.759	.757	.756	.755	.755	.755	.755
38	.939	.941	.946	.948	.950	.951	.951	.951	.951
40	1.14	1.15	1.16	1.17	1.18	1.18	1.18	1.18	1.18
42	1.38	1.38	1.41	1.44	1.45	1.46	1.46	1.46	1.46
44	1.65	1.65	1.70	1.74	1.77	1.78	1.78	1.78	1.79
46	1.97	1.96	2.03	2.09	2.13	2.15	2.16	2.16	2.16
48	2.34	2.31	2.40	2.49	2.55	2.58	2.59	2.60	2.60
50	2.76	2.71	2.81	2.94	3.02	3.07	3.09	3.10	3.11
52	3.24	3.15	3.27	3.44	3.56	3.62	3.66	3.68	3.68
54	3.79	3.66	3.79	4.00	4.16	4.26	4.30	4.33	4.34
56	4.41	4.23	4.37	4.63	4.84	4.97	5.03	5.07	5.09
58	5.12	4.87	5.00	5.32	5.59	5.76	5.85	5.90	5.93
60	5.91	5.59	5.71	6.08	6.42	6.64	6.77	6.84	6.87
62	6.80	6.39	6.50	6.91	7.33	7.62	7.79	7.88	7.93
64	7.79	7.29	7.37	7.82	8.33	8.70	8.92	9.04	9.11
66	8.90	8.28	8.33	8.83	9.42	9.88	10.17	10.33	10.42
68	10.1	9.4	9.4	9.9	10.6	11.2	11.5	11.7	11.9
70	11.5	10.6	10.6	11.1	11.9	12.6	13.0	13.3	13.5
72	13.0	12.0	11.8	12.4	13.3	14.1	14.7	15.0	15.2
74	14.6	13.5	13.2	13.8	14.8	15.8	16.5	16.9	17.1
76	16.5	15.1	14.8	15.4	16.5	17.6	18.4	18.9	19.2
78	18.5	16.9	16.5	17.1	18.2	19.5	20.5	21.1	21.5
80	20.6	18.8	18.3	18.9	20.2	21.6	22.7	23.5	24.0
82	23.0	21.0	20.3	20.9	22.2	23.8	25.2	26.1	26.7
84	25.6	23.3	22.5	23.1	24.5	26.2	27.8	28.9	29.6
86	28.4	25.8	24.9	25.4	26.9	28.8	30.5	31.9	32.8
88	31.5	28.6	27.5	27.9	29.4	31.5	33.5	35.1	36.1
90	34.8	31.5	30.3	30.7	32.2	34.4	36.7	38.5	39.8

FIGURE E3. 1986 AASHTO AXLELOAD EQUIVALENCY FACTORS FOR RIGID PAVEMENTS, TRIDEM AXLES AND P_t OF 2.5 (1).

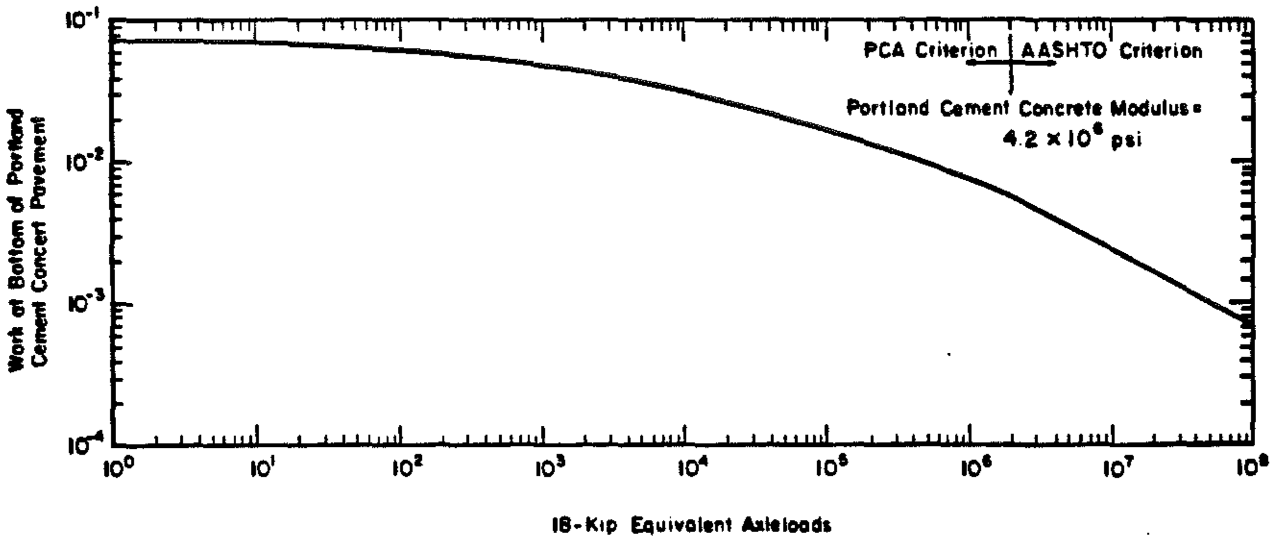

FIGURE E4. WORK AT THE BOTTOM OF THE PORTLAND CEMENT CONCRETE PAVEMENT AS A FUNCTION OF 18-KIP EQUIVALENT AXLELOADS.

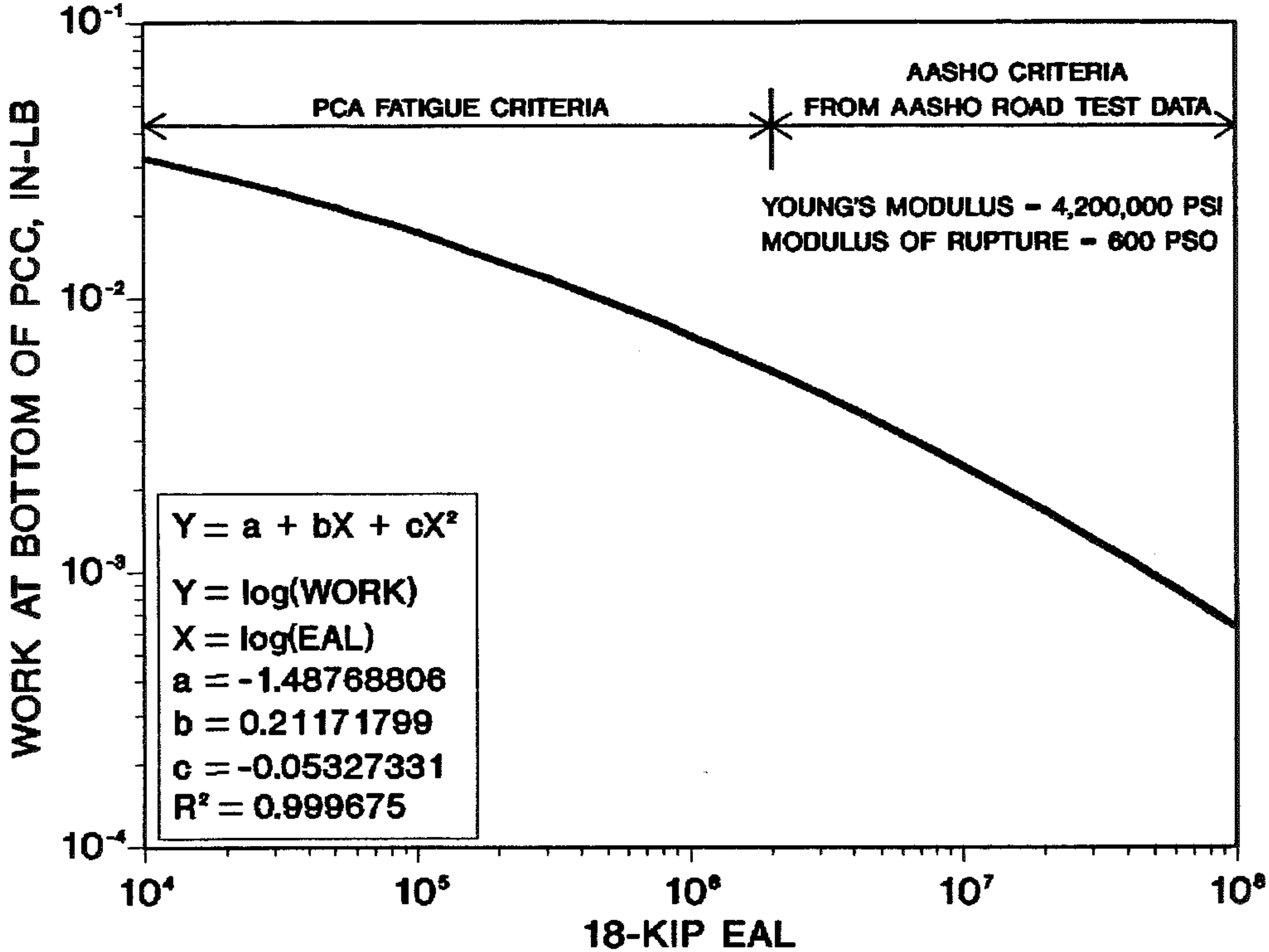

FIGURE E5. POLYNOMIAL EQUATION FITTED TO DATA USED IN FIGURE A4 TO DESCRIBE WORK AT BOTTOM OF PCC VERSUS 18-KIP EAL.

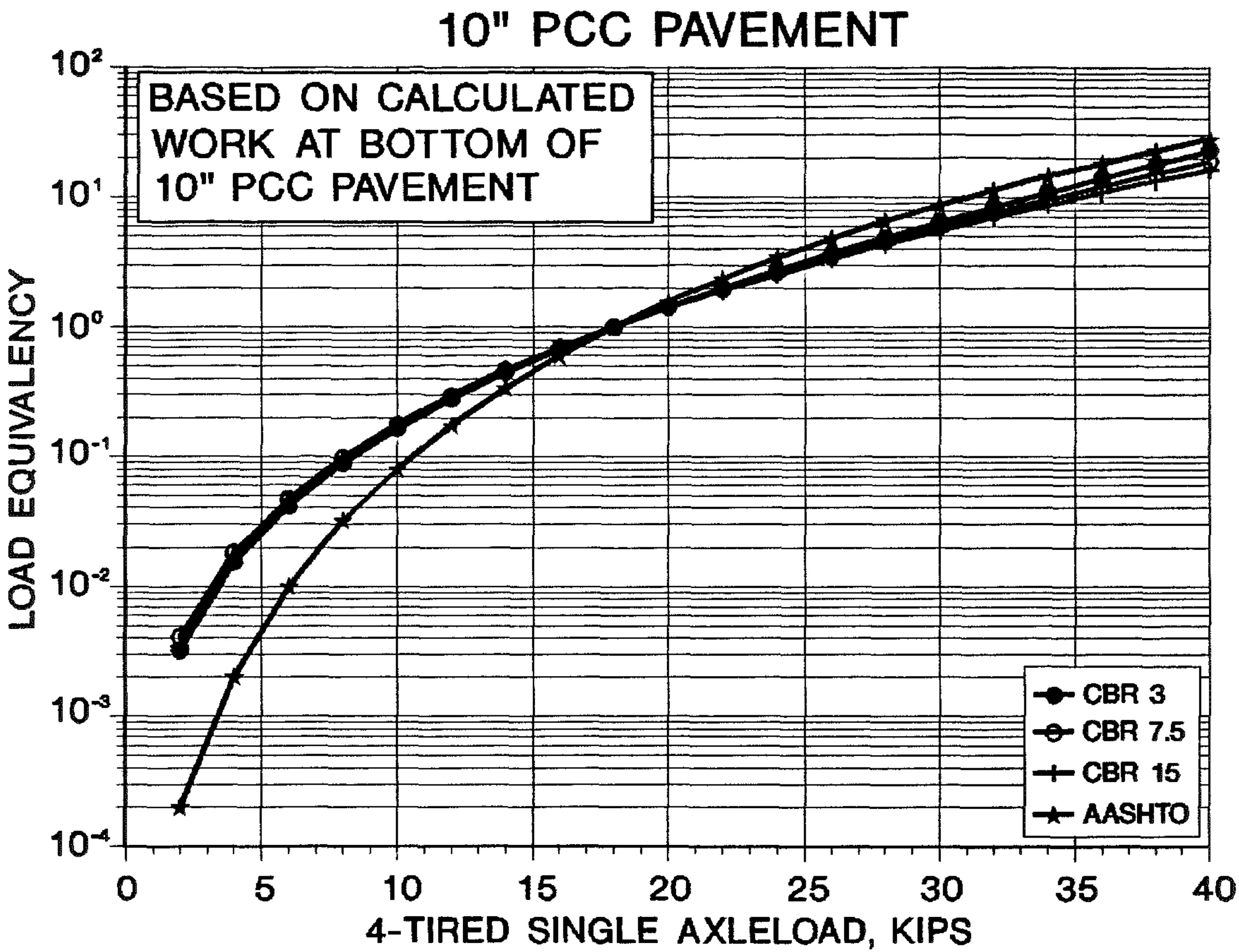

FIGURE E6. LOAD EQUIVALENCY FACTOR RELATIONSHIP FOR 4-TIRED SINGLE AXLELOAD FOR 10-INCH PCC PAVEMENT.

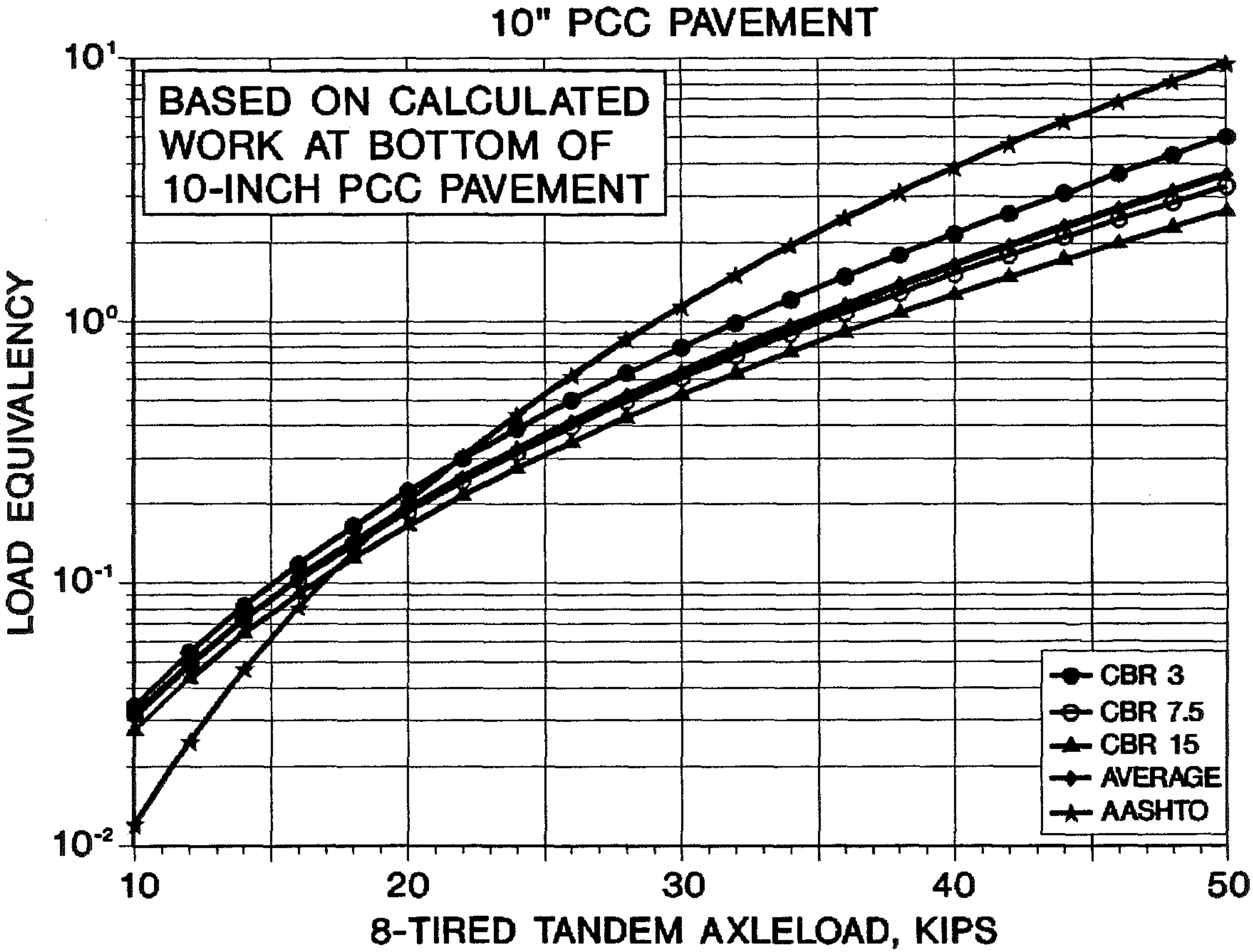

FIGURE E7. LOAD EQUIVALENCY FACTOR RELATIONSHIP FOR 8-TIRED TANDEM AXLELOAD FOR 10-INCH PCC PAVEMENT.

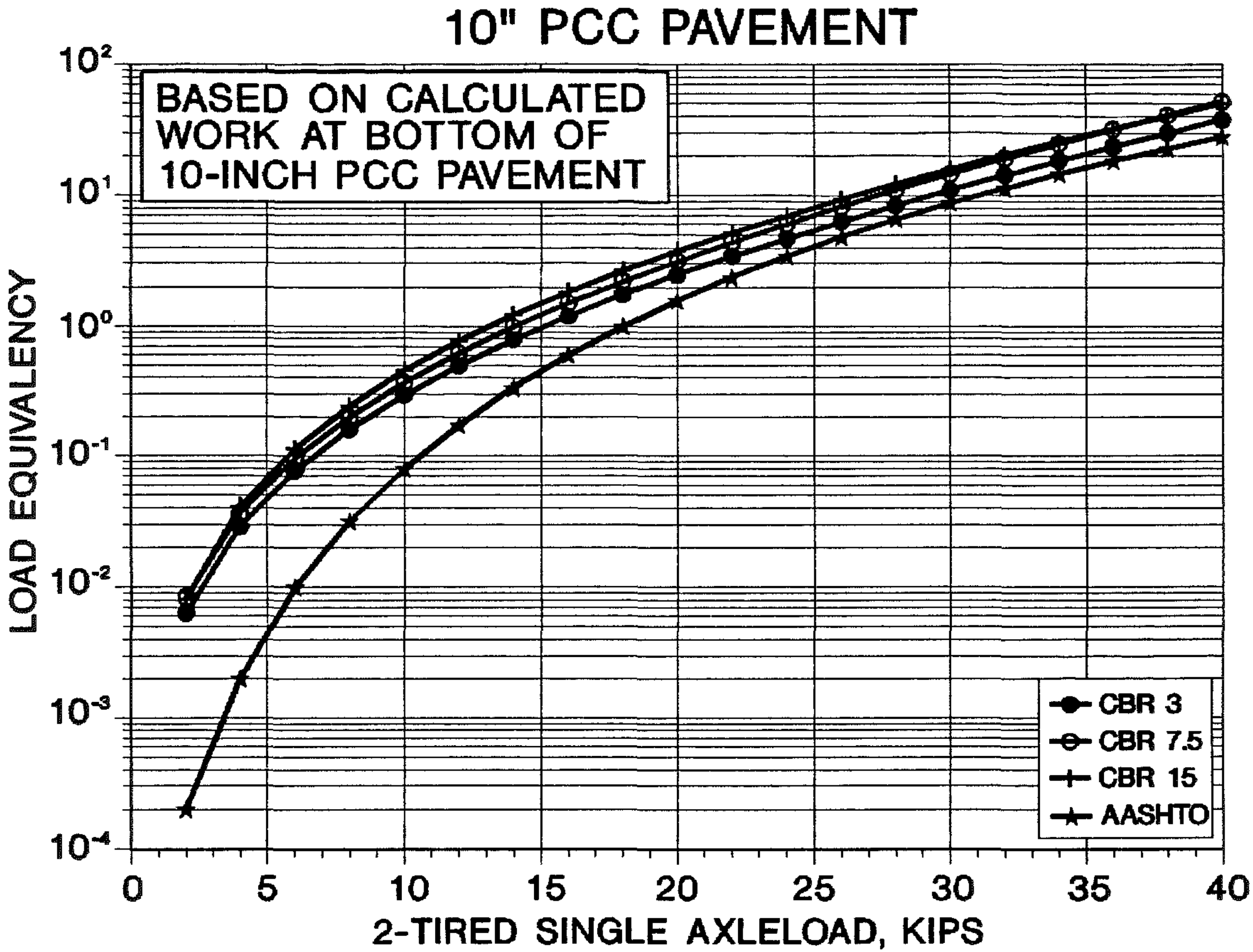

FIGURE E8. LOAD EQUIVALENCY FACTOR RELATIONSHIP FOR 2-TIRED SINGLE AXLELOAD FOR 10-INCH PCC PAVEMENT.

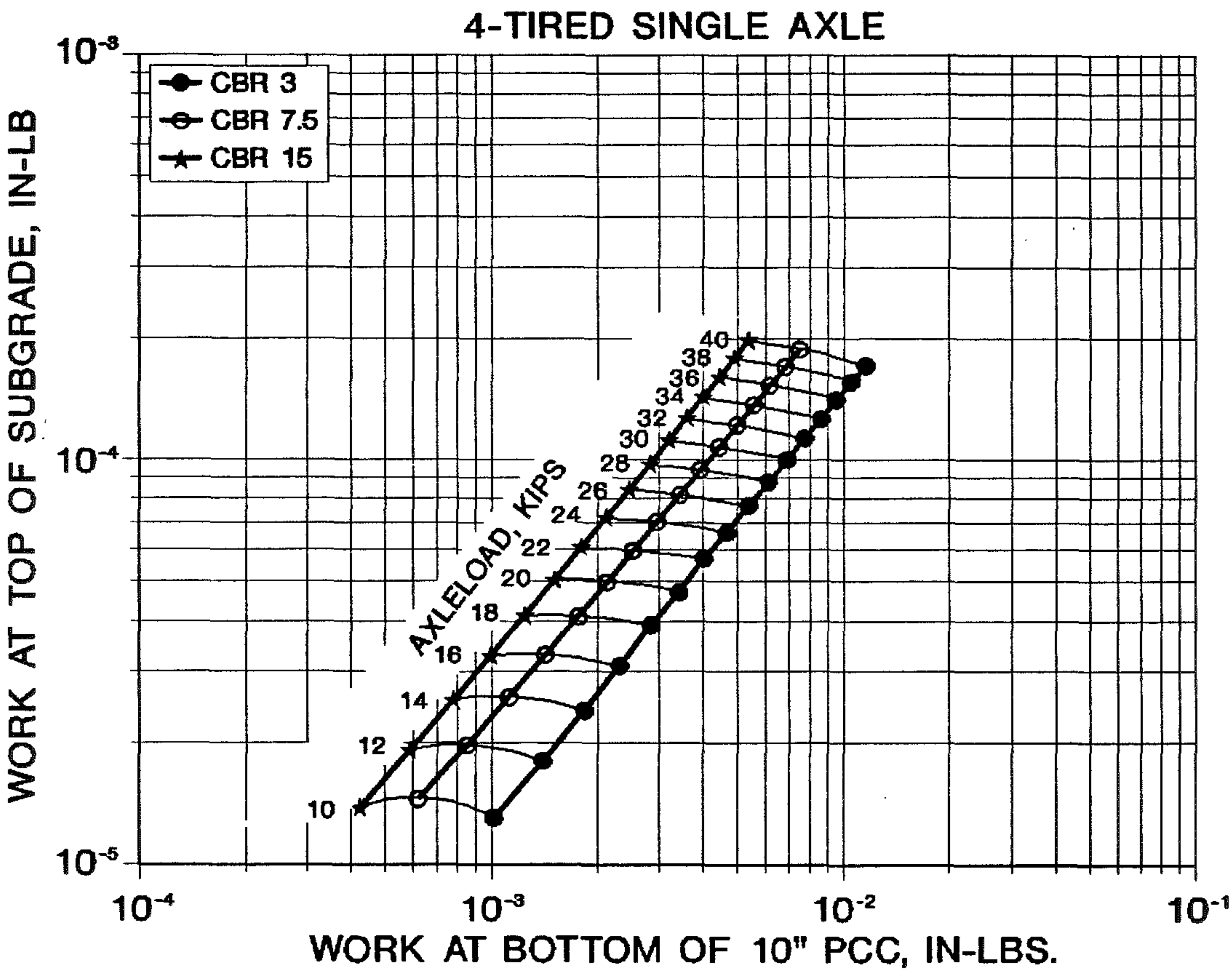

FIGURE E9. RELATIONSHIP BETWEEN WORK AT THE BOTTOM OF 10-INCH PCC PAVEMENT AND WORK AT THE TOP OF THE SUBGRADE FOR 4-TIRED SINGLE AXLE.

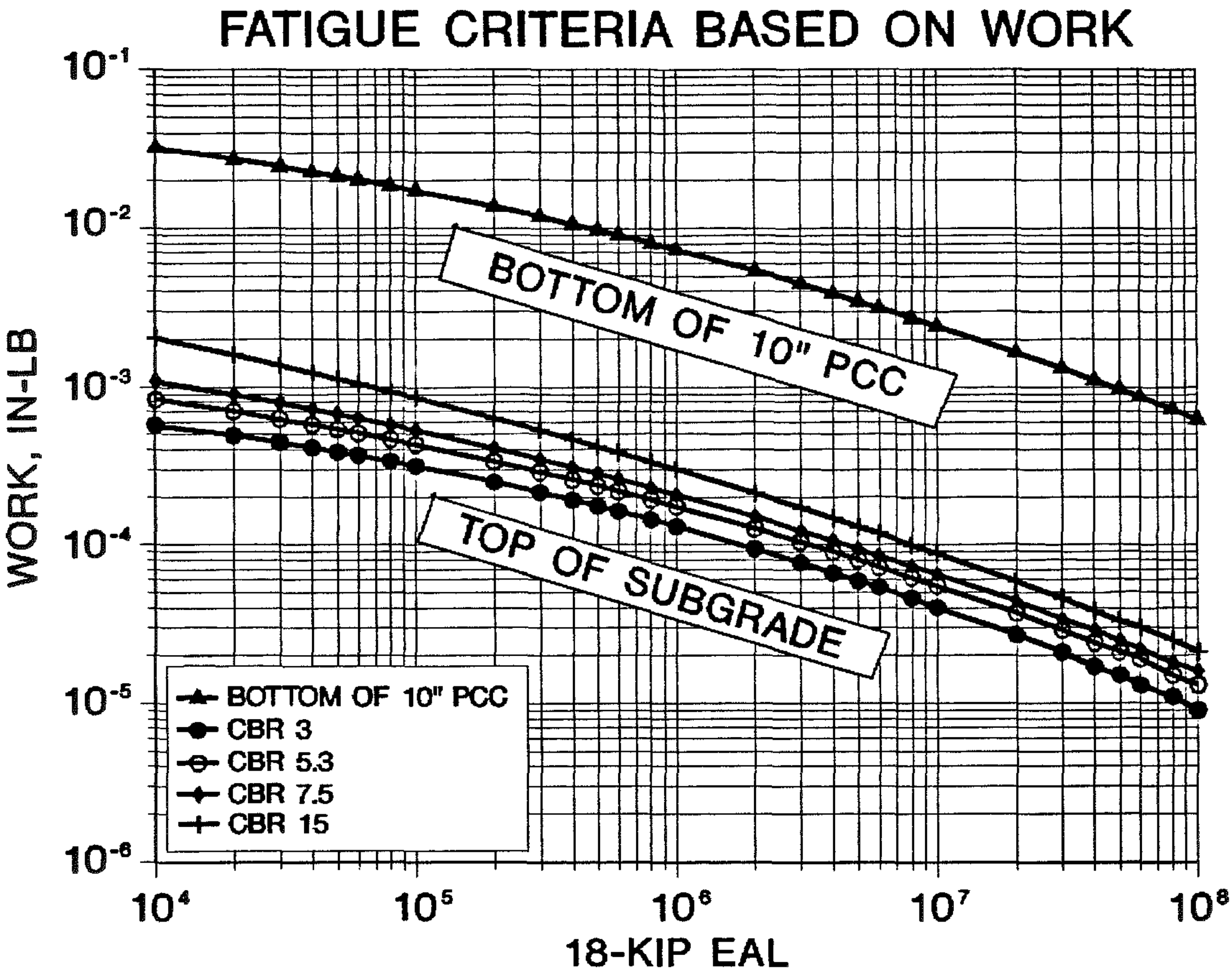

FIGURE E10. INTERRELATIONSHIP OF 18-KIP EAL FATIGUE AND WORK AT BOTTOM OF 10-INCH PCC PAVEMENT AND WORK AT TOP OF SUBGRADE FOR CBRs 3, 7.5, AND 15.

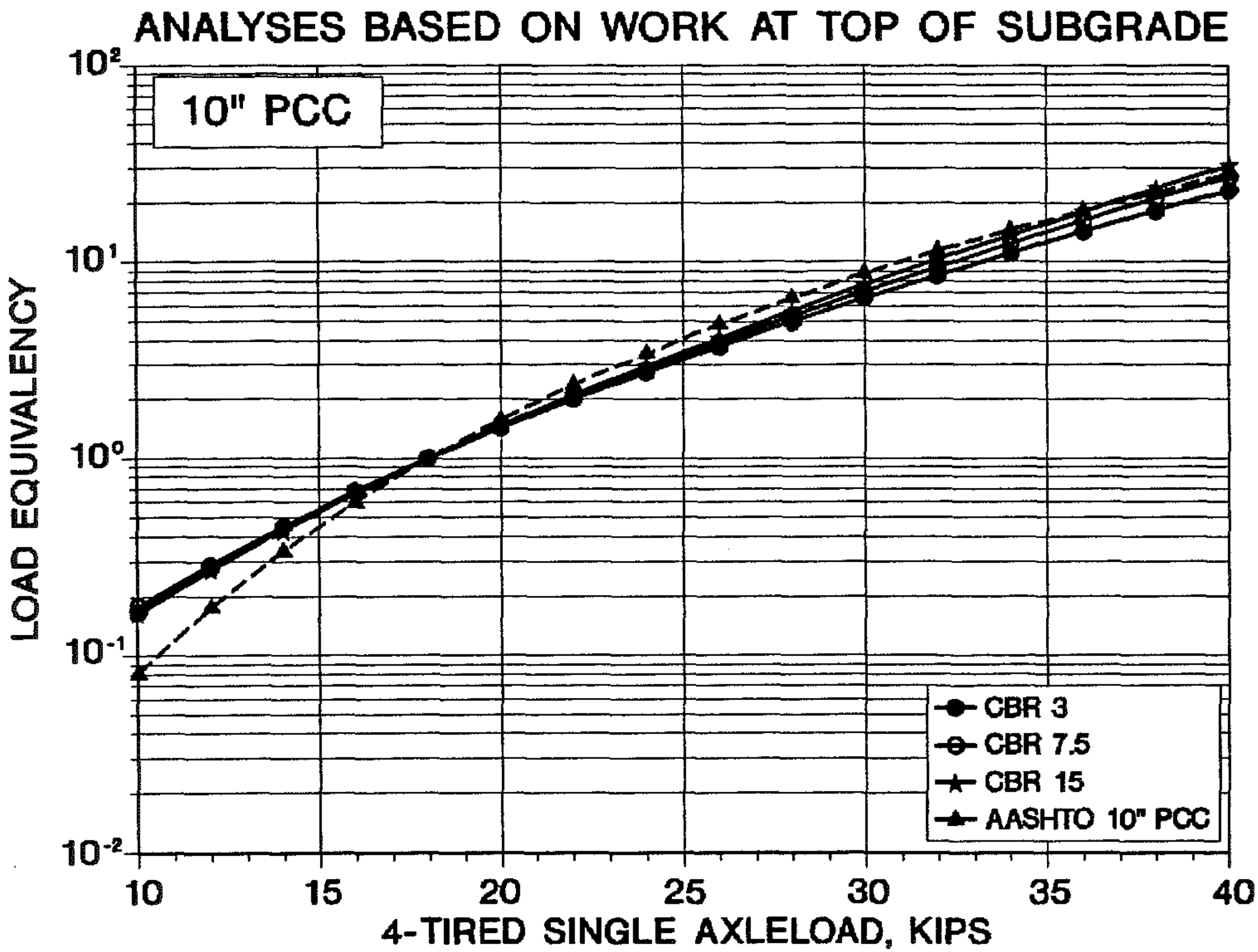

FIGURE E11. LOAD EQUIVALENCY FACTOR RELATIONSHIP FOR 4-TIRED SINGLE AXLELOAD BASED ON WORK AT THE TOP OF THE SUBGRADE.

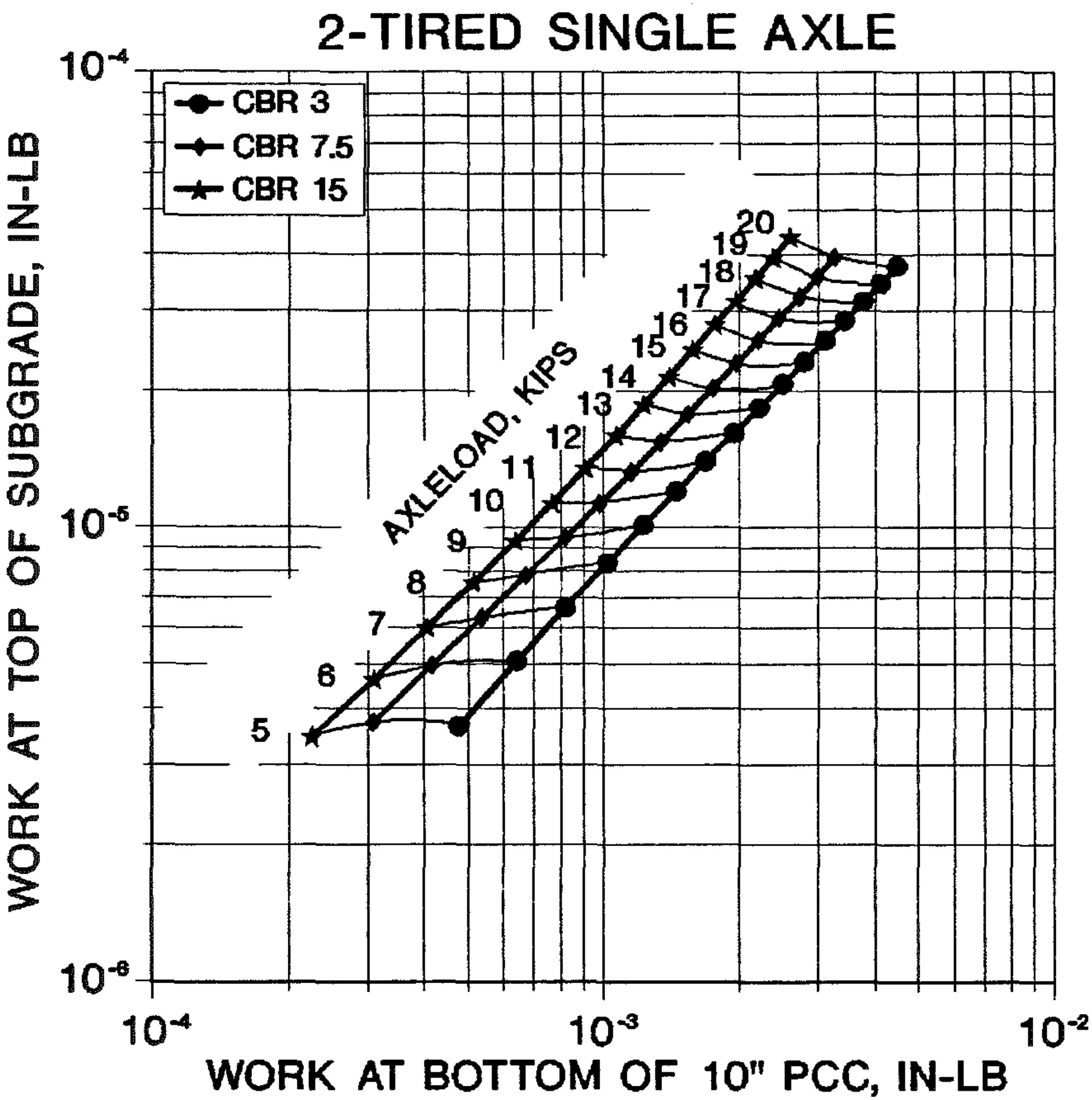

FIGURE E12. RELATIONSHIP BETWEEN WORK AT THE BOTTOM OF 10-INCH PCC PAVEMENT AND WORK AT THE TOP OF THE SUBGRADE FOR 2-TIRED SINGLE AXLE.

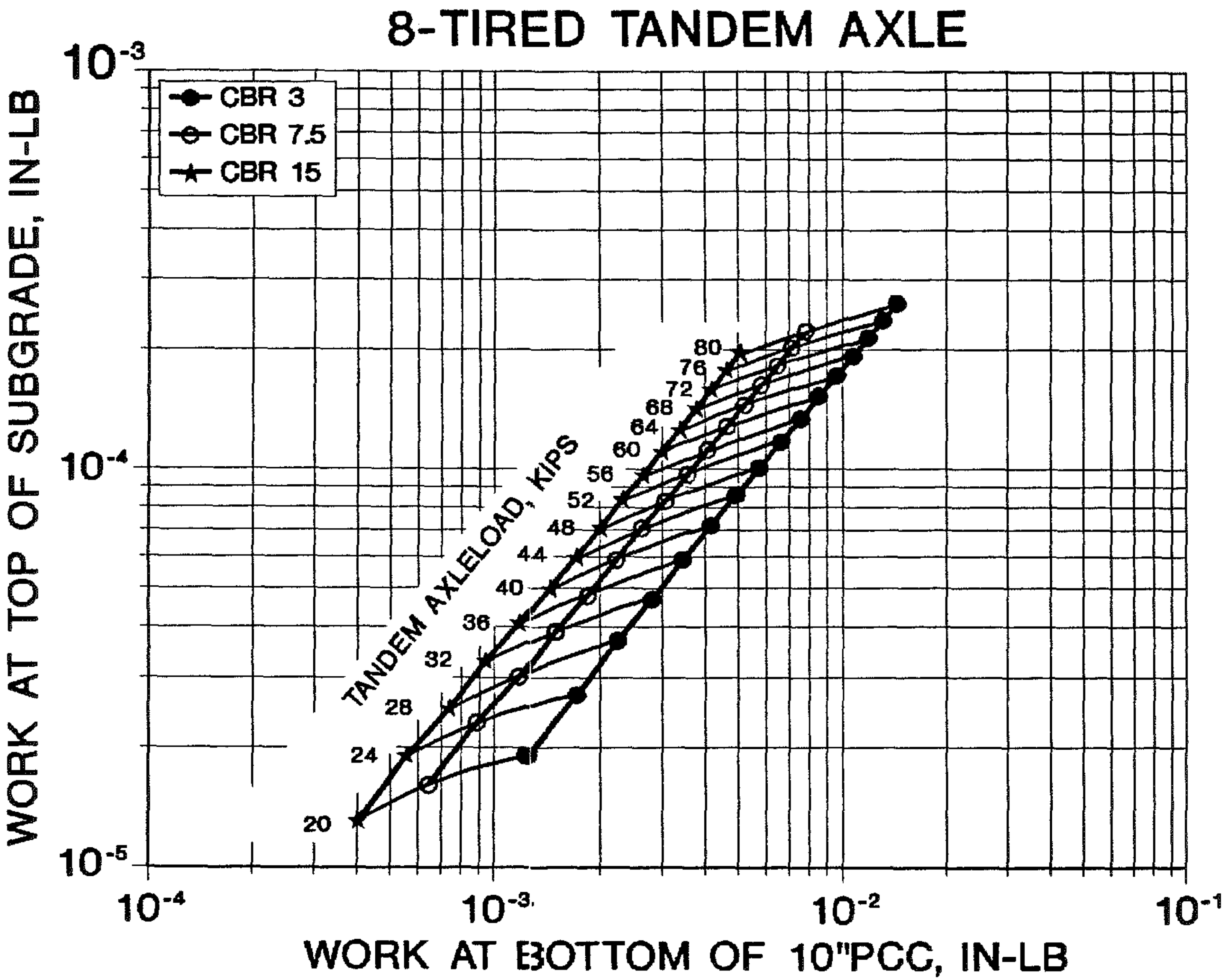

FIGURE E13. RELATIONSHIP BETWEEN WORK AT THE BOTTOM OF 10-INCH PCC PAVEMENT AND WORK AT THE TOP OF THE SUBGRADE FOR 8-TIRED TANDEM AXLE.

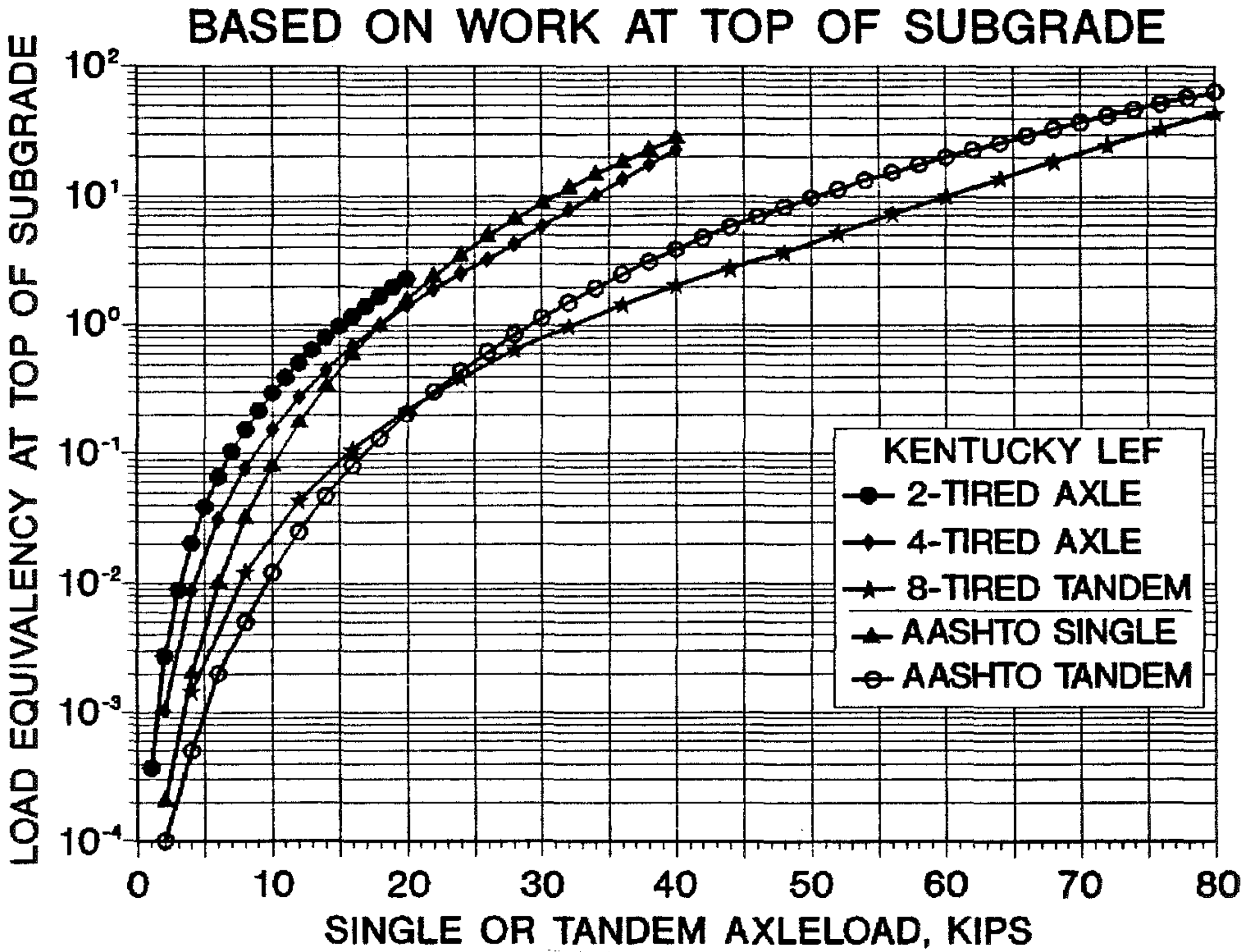

FIGURE E14. LOAD EQUIVALENCY FACTOR RELATIONSHIPS FOR 2-TIRED, 4-TIRED SINGLE AXLELOADS, AND 8-TIRED TANDEM AXLELOADS.

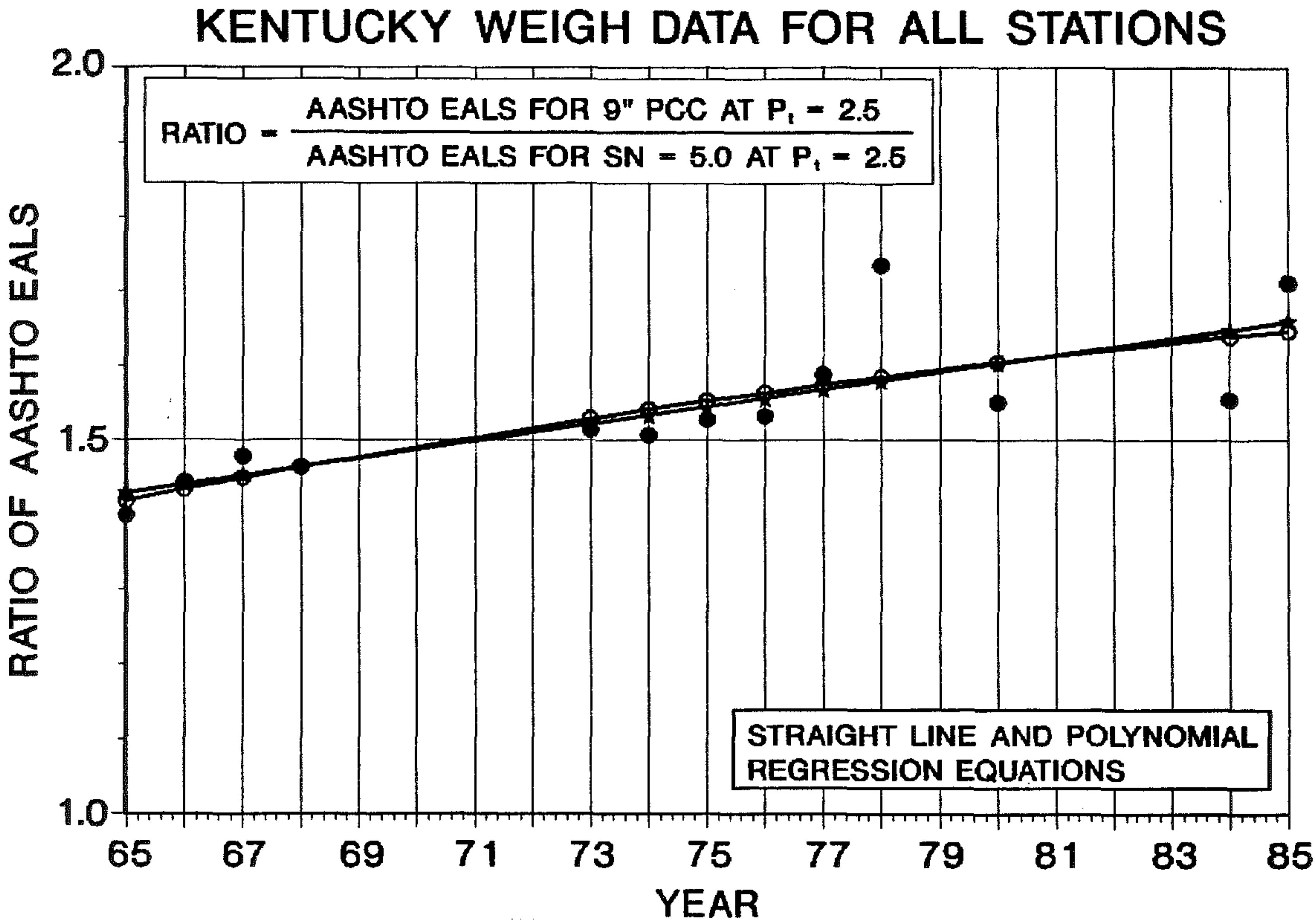

FIGURE E15. RATIO OF AASHTO EALS AS A FUNCTION CALENDAR YEAR FOR KENTUCKY WEIGH DATA FOR ALL STATIONS.

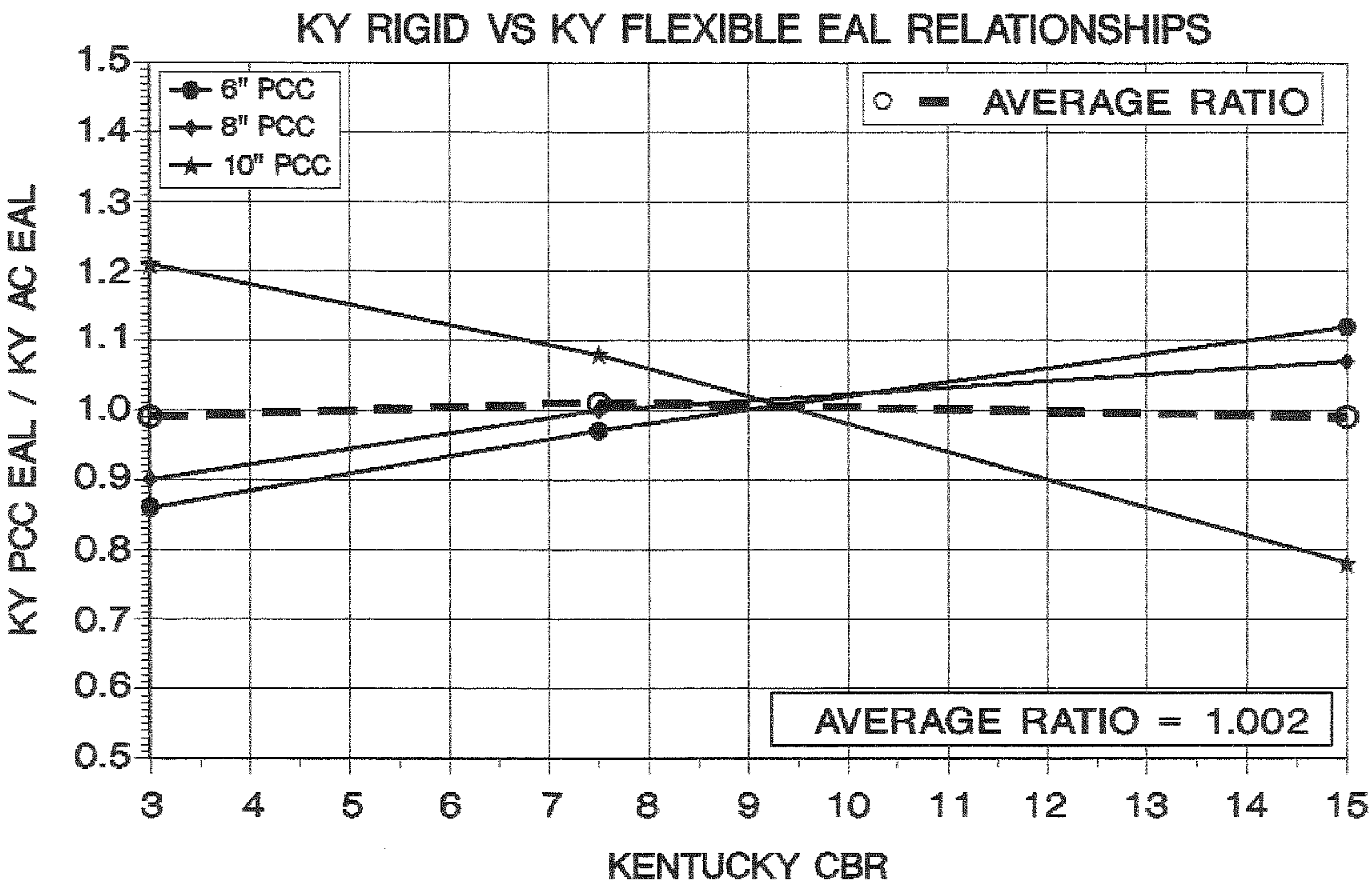

FIGURE E16. RATIO OF KENTUCKY PCC EAL TO KENTUCKY AC EAL VERSUS KENTUCKY CBR.

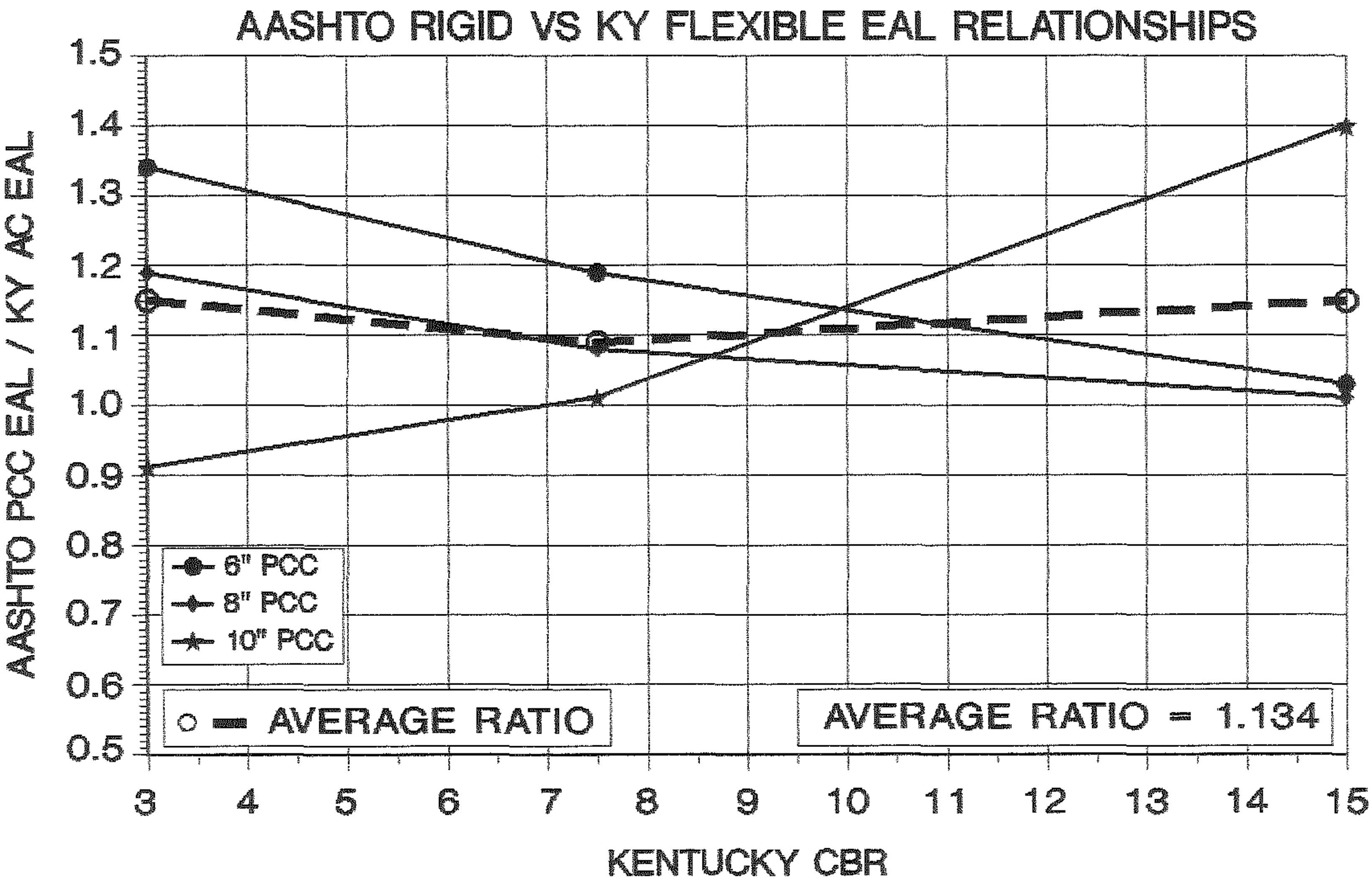

FIGURE E17. RATIO OF AASHTO PCC EAL TO KENTUCKY AC EAL VERSUS KENTUCKY CBR.

TABLE E1. EAL CALCULATION USING RECORDED AXLELOADS FOR TRUCKS IN SIX VEHICLE CLASSIFICATIONS AND LOAD EQUIVALENCY RELATIONSHIPS FOR KENTUCKY FLEXIBLE PAVEMENTS, KENTUCKY RIGID PAVEMENTS AND 1986 AASHTO AT Pt = 2.5 FOR 6-INCH CONCRETE PAVEMENT.

VEHICLE CLASS		NUMBER OF TRUCKS	18-KIP EAL	AVERAGE EAL/VEH
	CBR = 3		(KY RIGID)	
22		4,454	1,489.047	0.334317
23		1,878	1,928.417	1.026846
321		911	464.161	0.509507
322		1,713	952.114	0.555817
332		23,318	17,399.530	0.746184
5212		1,751	2,174.847	1.2420560
	TOTAL =	**34,025**	**24,408.116**	**0.717358**
	CBR = 7.5		(KY RIGID)	
22		4,454	1,724.994	0.387291
23		1,878	2,221.558	1.182938
321		911	549.881	0.603602
322		1,713	1,105.871	0.645576
332		23,318	19,376.400	0.830963
5212		1,751	2,463.006	1.406628
	TOTAL =	**34,025**	**27,441.710**	**0.806516**
	CBR = 15		(KY RIGID)	
22		4,454	2,018.487	0.453185
23		1,878	2,421.882	1.289607
321		911	629.459	0.690954
322		1,713	1,272.309	0.742737
332		23,318	22,567.810	0.967828
5212		1,751	2,807.286	1.603247
	TOTAL =	**34,025**	**31,717.230**	**0.932174**
	KENTUCKY AC		(KY FLEXIBLE)	
22		4,454	1,313.464	0.294895
23		1,878	1,342.488	0.714850
321		911	384.005	0.421520
322		1,713	819.745	0.478543
332		23,318	22,537.040	0.966508
5212		1,751	1,871.249	1.068674
	TOTAL =	**34,025**	**28,267.991**	**0.830801**
	AASHTO 6" PCC		(RIGID)	
22		4,454	916.167	0.205695
23		1,878	1,397.229	0.743998
321		911	310.499	0.340833
322		1,713	781.267	0.456081
332		23,318	27,639.430	1.185326
5212		1,751	1,848.818	1.055864
	TOTAL =	**34,025**	**32,893.410**	**0.966742**

TABLE E2. EAL CALCULATION USING RECORDED AXLELOADS FOR TRUCKS IN SIX VEHICLE CLASSIFICATIONS AND LOAD EQUIVALENCY RELATIONSHIPS FOR KENTUCKY FLEXIBLE PAVEMENTS, KENTUCKY RIGID PAVEMENTS AND 1986 AASHTO AT Pt = 2.5 FOR 8-INCH CONCRETE PAVEMENT.

VEHICLE CLASS		NUMBER OF TRUCKS	18-KIP EAL	AVERAGE EAL/VEH
	CBR = 3		(KY RIGID)	
22		4,454	1,499.474	0.336658
23		1,878	1,601.803	0.852930
321		911	480.481	0.527422
322		1,713	1,021.879	0.596543
332		23,318	21,541.070	0.923796
5212		1,751	2,438.889	1.392855
	TOTAL =	**34,025**	**25,583.596**	**0.751906**
	CBR = 7.5		(KY RIGID)	
22		4,454	1,716.790	0.385449
23		1,878	1,798.750	0.957801
321		911	548.089	0.601634
322		1,713	1,123.146	0.655660
332		23,318	20,503.460	0.879298
5212		1,751	2,649.899	1.513363
	TOTAL =	**34,025**	**28,340.134**	**0.832921**
	CBR = 15		(KY RIGID)	
22		4,454	1,928.865	0.433064
23		1,878	2,017.862	1.074474
321		911	606.202	0.665425
322		1,713	1,230.504	0.718333
332		23,318	21,691.340	0.930240
5212		1,751	2,832.376	1.617576
	TOTAL =	**34,025**	**30,307.149**	**0.890732**
	KENTUCKY AC		(KY FLEXIBLE)	
22		4,454	1,313.464	0.294895
23		1,878	1,342.488	0.714850
321		911	384.005	0.421520
322		1,713	819.745	0.478543
332		23,318	22,537.040	0.966508
5212		1,751	1,871.249	1.068674
	TOTAL =	**34,025**	**28,267.991**	**0.830801**
	AASHTO 8" PCC		(RIGID)	
22		4,454	843.169	0.189306
23		1,878	1,295.436	0.689796
321		911	285.917	0.313850
322		1,713	723.327	0.422257
332		23,318	25,766.050	1.104985
5212		1,751	1,705.412	0.973965
	TOTAL =	**34,025**	**30,619.311**	**0.899906**

TABLE E3. EAL CALCULATION USING RECORDED AXLELOADS FOR TRUCKS IN SIX VEHICLE CLASSIFICATIONS AND LOAD EQUIVALENCY RELATIONSHIPS FOR KENTUCKY FLEXIBLE PAVEMENTS, KENTUCKY RIGID PAVEMENTS AND 1986 AASHTO AT Pt = 2.5 FOR 10-INCH CONCRETE PAVEMENT.

VEHICLE CLASS		NUMBER OF TRUCKS	18-KIP EAL	AVERAGE EAL/VEH
	CBR = 3		(KY RIGID)	
22		4,454	1,732.434	0.388961
23		1,878	1,910.659	1.017390
321		911	551.981	0.605907
322		1,713	1,199.931	0.700485
332		23,318	26,181.500	1.122802
5212		1,751	2,730.040	1.559132
	TOTAL =	**34,025**	**34,306.545**	**1.008275**
	CBR = 7.5		(KY RIGID)	
22		4,454	1,746.283	0.392071
23		1,878	1,651.515	0.879401
321		911	561.904	0.616799
322		1,713	1,168.643	0.682220
332		23,318	22,754.730	0.975844
5212		1,751	2,785.584	1.590853
	TOTAL =	**34,025**	**30,668.659**	**0.901357**
	CBR = 15		(KY RIGID)	
22		4,454	1,604.506	0.360239
23		1,878	1,219.085	0.64914
321		911	526.302	0.577719
322		1,713	1,035.709	0.604617
332		23,318	15,119.560	0.648407
5212		1,751	2,705.424	1.545074
	TOTAL =	**34,025**	**22,210.586**	**0.652773**
	KENTUCKY AC		(KY FLEXIBLE)	
22		4,454	1,313.464	0.294895
23		1,878	1,342.488	0.714850
321		911	384.005	0.421520
322		1,713	819.745	0.478543
332		23,318	22,537.040	0.966508
5212		1,751	1,871.249	1.068674
	TOTAL =	**34,025**	**28,267.991**	**0.830801**
	AASHTO 10" PCC		(RIGID)	
22		4,454	858.588	0.192768
23		1,878	1,327.119	0.706666
321		911	290.217	0.318570
322		1,713	734.844	0.428981
332		23,318	26,290.740	1.127487
5212		1,751	1,730.872	0.988505
	TOTAL =	**34,025**	**31,232.380**	**0.917924**

TABLE E4. RATIOS OF EALS SHOWN IN TABLES E1 THROUGH E3.

DESCRIPTION		KENTUCKY PCC THICKNESS 6"	8"	10"
DATA SOURCE		TABLE E1	TABLE E2	TABLE E3
KENTUCKY AC EALs		**28,268**	**28,268**	**28,268**
KENTUCKY RIGID PAVEMENT EALs:				
CBR				
3.0		24,408	25,584	34,307
7.5		27,441	28,340	30,669
15.0		31,717	30,307	22,211
KY RIGID EAL / KY AC EAL:				
CBR			RATIO	
3.0		0.863	0.905	1.214
7.5		0.971	1.003	1.085
15.0		1.122	1.072	0.786
AVERAGE OF 9 RATIOS:	1.002			
AASHTO RIGID EALs FOR Pt = 2.5:		**32,893**	**30,619**	**31,232**
AASHTO RIGID EAL / KY RIGID EAL:				
CBR				
3.0		1.348	1.197	0.910
7.5		1.199	1.080	1.018
15.0		1.037	1.010	1.406
AVERAGE OF 9 RATIOS:	1.134			

CBR	Mr	K	AASHTO DESIGN EALs, MILLION		
3.0	2,063	106	0.46	2.50	7.8
7.5	3,770	194	1.10	5.80	17
15.0	5,956	307	2.20	11	32

ADJUSTED AASHTO RIGID EAL:

CBR	EXAMPLE	6"	8"	10"
3.0	0.46x1.348	0.62	2.99	7.10
7.5	1.1x1.199	1.32	6.26	17.31
15.0	2.2x1.037	2.28	11.11	45.00

TABLE E5. AASHTO DESIGN THICKNESSES FOR AASHTO EALS ADJUSTED TO EQUIVALENT KENTUCKY RIGID PAVEMENT EALS.

PERCENT RELIABILITY	KY PCC"	AASHTO DESIGN THICKNESS, INCHES		
		CBR 3	CBR 7.5	CBR 15
50	6	5.46	5.95	6.29
70		6.12	6.70	7.12
80		6.30	6.90	7.33
90		6.77	7.42	7.89
50	8	7.16	7.97	8.51
70		7.94	8.82	9.43
80		8.14	9.04	9.66
90		8.68	9.64	10.31
50	10	8.43	9.44	10.26
70		9.29	10.39	11.31
80		9.51	10.64	11.58
90		10.11	11.31	12.32

P_t = 2.5
J = 3.1
f'_c = 600 psi
STANDARD DEVIATION = 0.39
E = 4,200,000 psi
DRAINAGE COEFFICIENT = 1.0

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APPENDIX F

COMPARISON OF KENTUCKY AND AASHTO RIGID PAVEMENT THICKNESS DESIGNS

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The figures in this appendix were developed for a terminal level of serviceability of 2.50. The ACPA (1) computer program was used to calculate the AASHTO design thicknesses for 50, 80, 85, 90, 95, and 98 percent reliabilities for CBRs 3, 7.5 and 15. The "k" factors used in these calculations corresponded with the relationship given in Scale 14 in Figure D9. For Figures F1-F3 (CBR 3, 7.5, and 15 respectively), the AASHTO and Kentucky EAL scales are superimposed on the right side of each figure.

Kentucky thickness design curves for rigid pavements contain an implied, but not specific, level of serviceability built into them. However, percent reliability was not considered in the developement of the curves.

Adjusting the AASHTO and Kentucky EALs to "equivalent" values, a Kentucky thickness for a given CBR would correspond to an AASHTO EAL curve for which the thickness varies as a function of percent reliability. For equal thickness designs, the fixed Kentucky thickness would coincide with the AASHTO thickness at some percent reliability. Figures F4-F6 are the Kentucky thickness designs superimposed on the family of AASHTO thickness designs. Intersections of equal design thicknesses (labeled "Match Line") indicate that the percent reliability varies according to thickness for a given CBR and also across CBRs.

Figures similar to F1-F6 were created for "k" values corresponding to Scale 12 of Figure D9. Intersections of equal design thicknesses created match lines having quite different relationships for percent reliability. This serves to emphasize the need to resolve the subgrade stiffness problems discussed in Appendix D.

LIST OF REFERENCES

1. Simplified Guide for the Design of Concrete Pavements," American Concrete Pavement Association, Arlington Heights, IL, 1988.

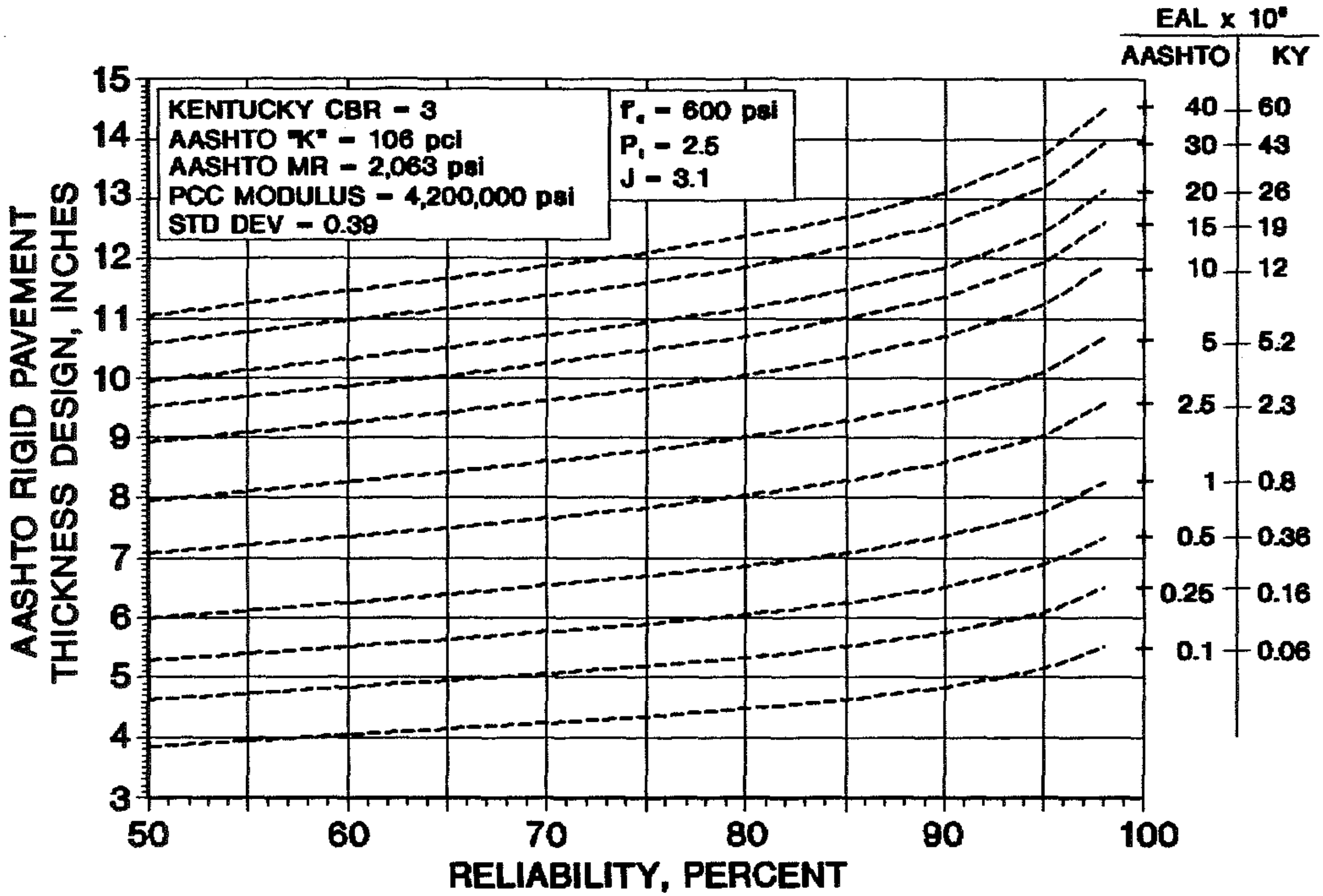

FIGURE F1. INTERRELATIONSHIPS OF AASHTO 18-KIP EALS, PERCENT RELIABILITY, AND AASHTO PAVEMENT THICKNESS DESIGNS FOR CBR 3 SUBGRADE.

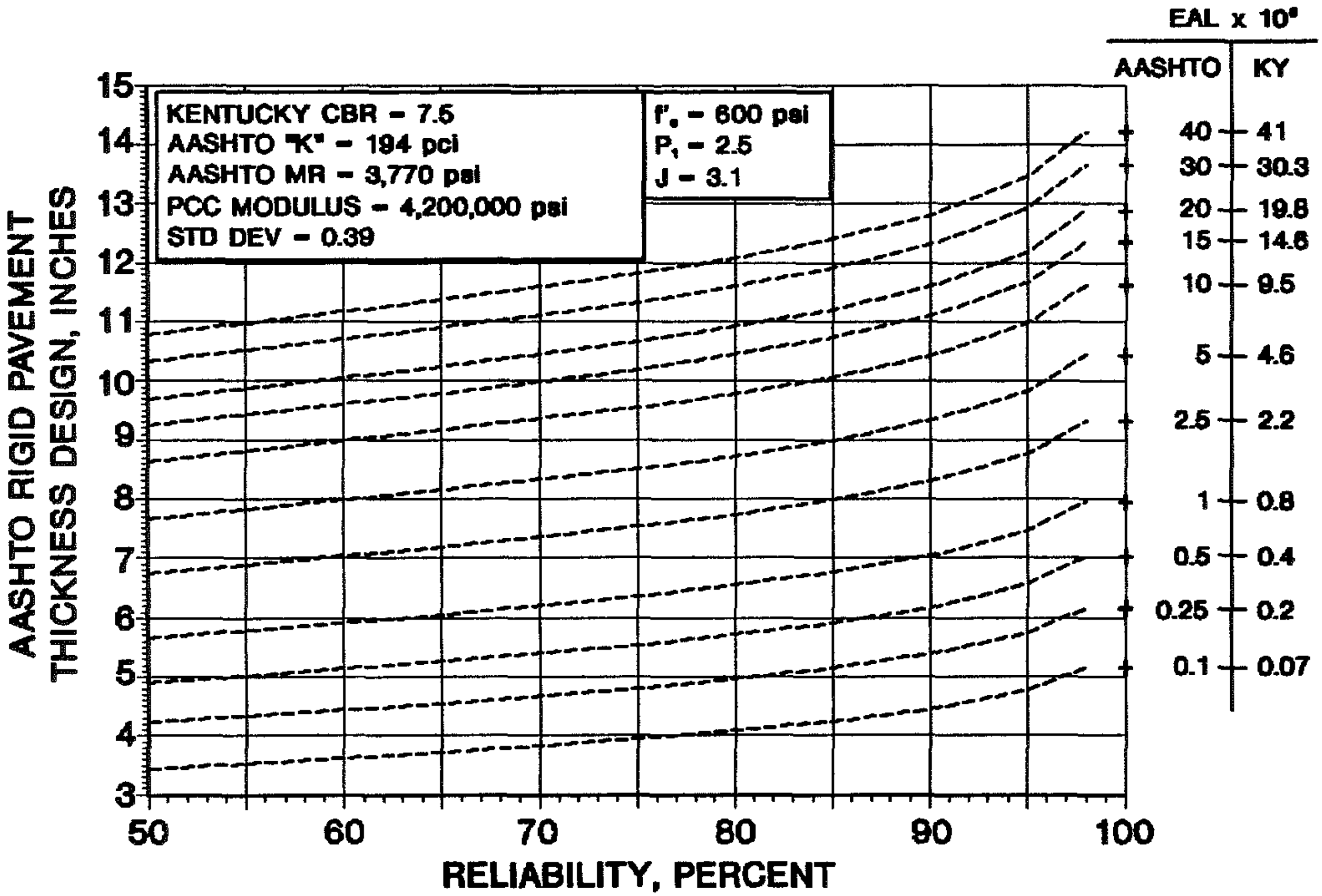

FIGURE F2. INTERRELATIONSHIPS OF AASHTO 18-KIP EALS, PERCENT RELIABILITY, AND AASHTO PAVEMENT THICKNESS DESIGNS FOR CBR 7.5 SUBGRADE.

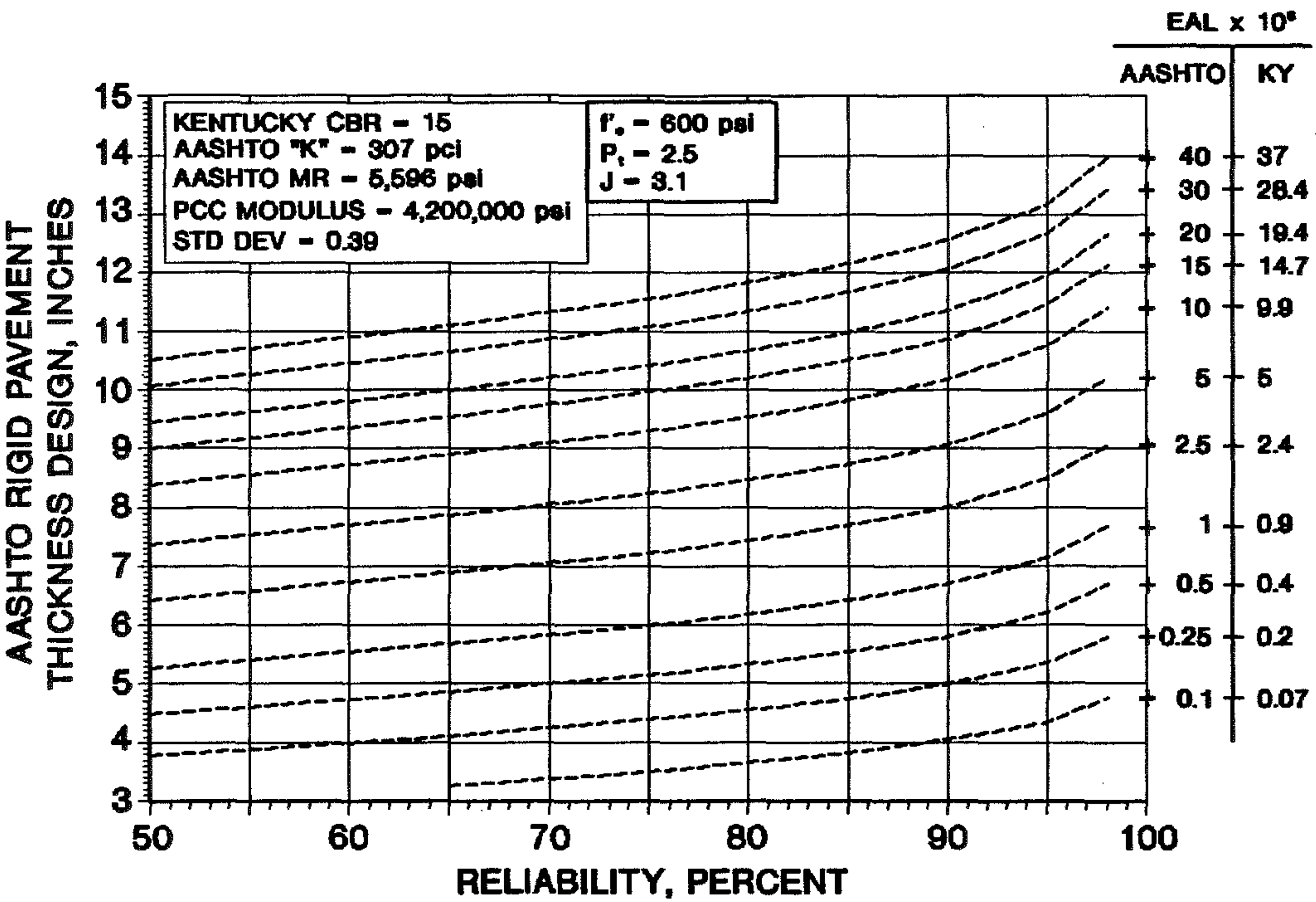

FIGURE F3. INTERRELATIONSHIPS OF AASHTO 18-KIP EALS, PERCENT RELIABILITY, AND AASHTO PAVEMENT THICKNESS DESIGNS FOR CBR 15 SUBGRADE.

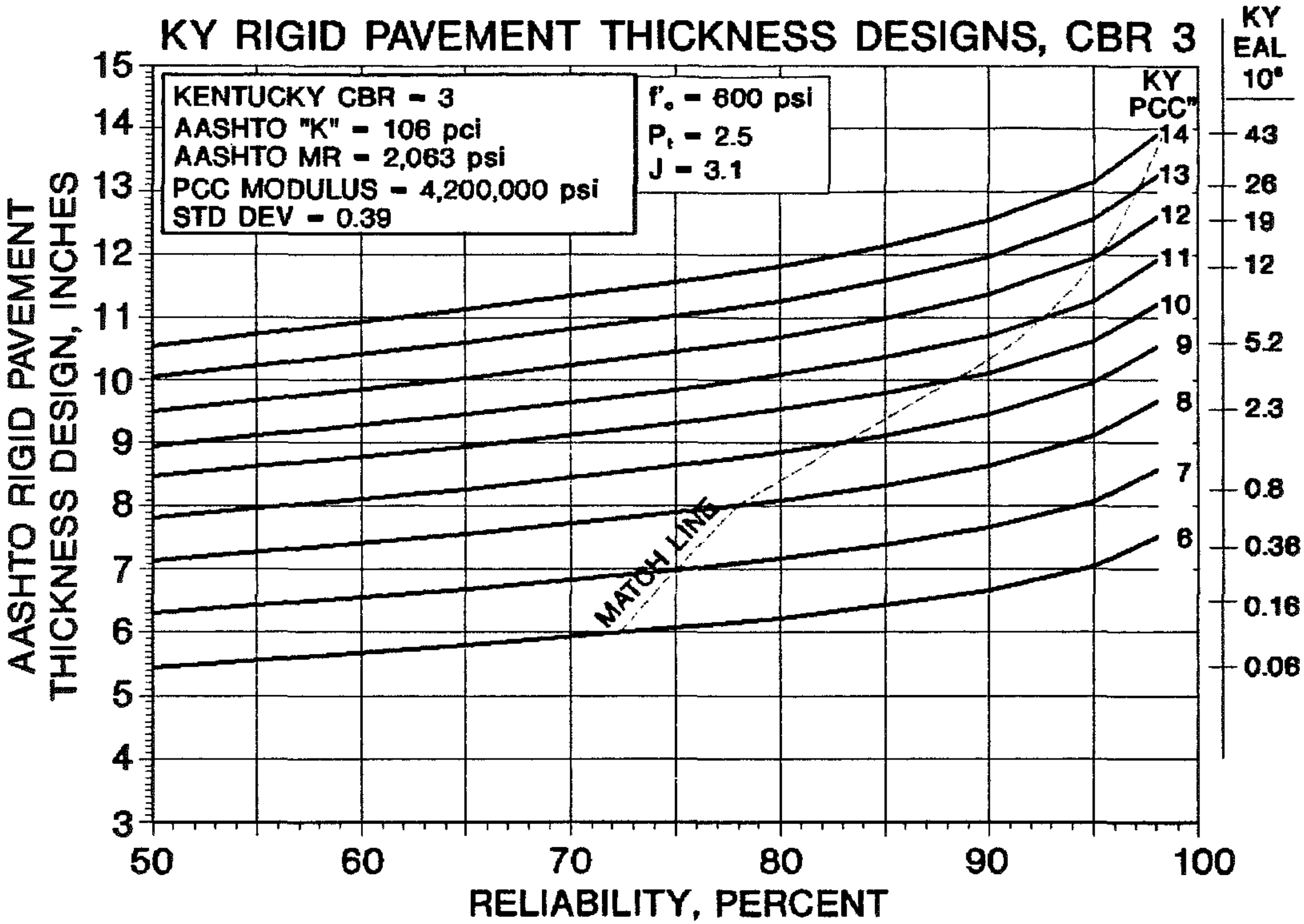

FIGURE F4. KENTUCKY THICKNESS DESIGNS FOR CBR 3 SUBGRADE SUPERIMPOSED ON FIGURE F1.

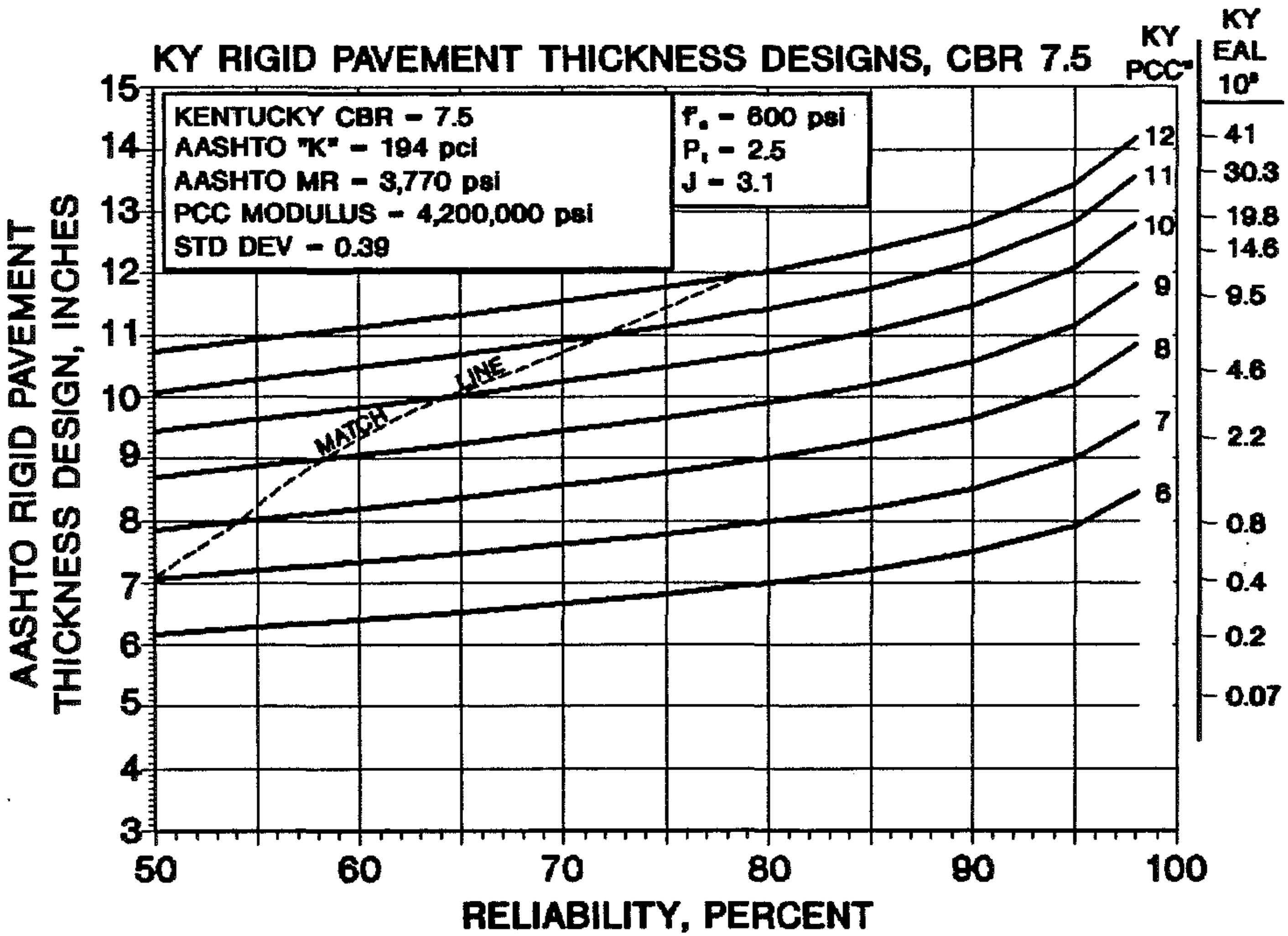

FIGURE F5. KENTUCKY THICKNESS DESIGNS FOR CBR 7.5 SUBGRADE SUPERIMPOSED ON FIGURE F2.

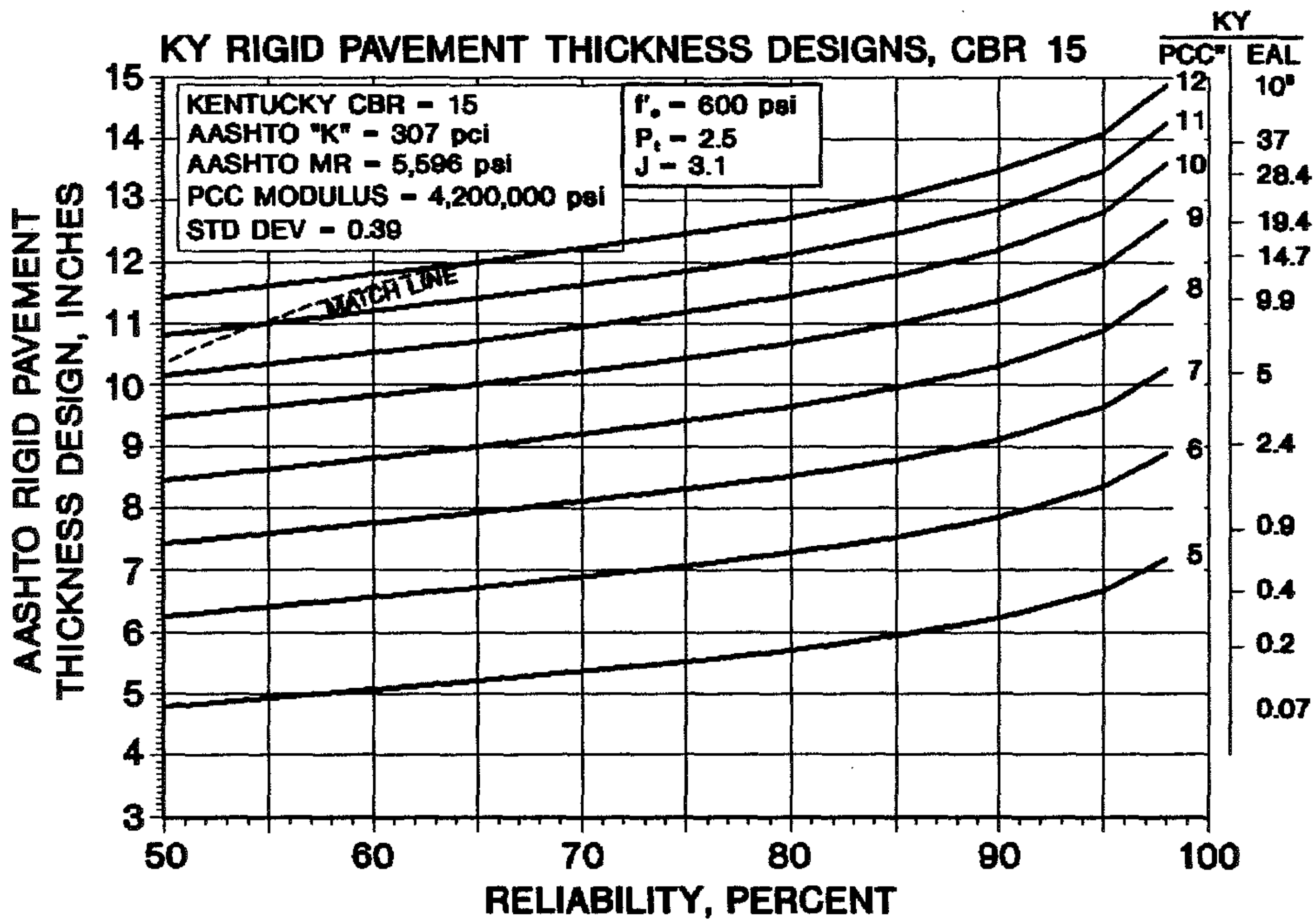

FIGURE F6. KENTUCKY THICKNESS DESIGNS FOR CBR 15 SUBGRADE SUPERIMPOSED ON FIGURE F3.

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APPENDIX G

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS

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In each table, four sets of thickness designs are given. The second column contains thickness designs that are calculated solutions using 1984 Kentucky rigid pavement thickness design method for the level of EAL contained in the left column. The next three columns are calculated rigid pavement thickness designs using the 1986 AASHTO Guide for Design of Pavement Structures. The first column of AASHTO thickness designs corresponds to the value of EAL shown in the left column. The second column corresponds to 1.5 times the EAL in the left column and the third column corresponds to 2 times the EAL in the left column. For each CBR, three tables are shown corresponding to serviceability levels of 2.0, 2.5, and 3.0 respectively. For each CBR, the corresponding modulus of subgrade reaction, k, is given.

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 2.

Pt = 2.0

CBR 2 K=80 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	7.18	7.29	7.78	8.14
2.5	8.46	8.43	8.98	9.39
5	9.72	9.39	9.99	10.43
7.5	10.57	9.99	10.62	11.09
10	11.23	10.43	11.09	11.57
15	12.23	11.09	11.78	12.3
20	12.99	11.57	12.3	12.83
25	13.61	11.97	12.71	13.26
30	14.13	12.3	13.06	13.62
35	14.59	12.58	13.36	13.94
40	15	12.83	13.62	14.21
45	15.36	13.06	13.86	14.46
50	15.7	13.26	14.07	14.68
75	17.04	14.07	14.94	15.58
100	18.04	14.68	15.58	16.24

Pt = 2.5

CBR 2 K=80 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	7.18	7.48	8.01	8.39
2.5	8.46	8.69	9.26	9.68
5	9.72	9.68	10.3	10.76
7.5	10.57	10.3	10.95	11.43
10	11.23	10.76	11.43	11.94
15	12.23	11.43	12.15	12.68
20	12.99	11.94	12.68	13.23
25	13.61	12.34	13.11	13.67
30	14.13	12.68	13.46	14.04
35	14.59	12.97	13.77	14.36
40	15	13.23	14.04	14.65
45	15.36	13.46	14.29	14.9
50	15.7	13.67	14.51	15.14
75	17.04	14.51	15.4	16.05
100	18.04	15.31	16.05	16.74

Pt = 3.0

CBR 2 K=80 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	7.18	7.78	8.34	8.73
2.5	8.46	9.05	9.65	10.09
5	9.72	10.09	10.73	11.21
7.5	10.57	10.73	11.41	11.91
10	11.23	11.21	11.91	12.43
15	12.23	11.91	12.65	13.2
20	12.99	12.43	13.2	13.77
25	13.61	12.85	13.64	14.23
30	14.13	13.2	14.01	14.62
35	14.59	13.51	14.33	14.95
40	15	13.77	14.62	15.24
45	15.36	14.01	14.87	15.51
50	15.7	14.23	15.1	15.74
75	17.04	15.1	16.02	16.7
100	18.04	15.74	16.7	17.41

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 3.

Pt = 2.0

CBR 3 k=101 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.75	7.19	7.68	8.05
2.5	7.92	8.34	8.89	9.3
5	9.08	9.3	9.9	10.34
7.5	9.86	9.9	10.53	11
10	10.46	10.34	11	11.49
15	11.38	11	11.69	12.21
20	12.07	11.49	12.21	12.74
25	12.64	11.88	12.62	13.17
30	13.12	12.21	12.97	13.53
35	13.54	12.49	13.27	13.84
40	13.91	12.74	13.53	14.12
45	14.25	12.97	13.77	14.37
50	14.55	13.17	13.99	14.59
75	15.78	13.99	14.85	15.49
100	16.7	14.59	15.49	16.15

Pt = 2.5

CBR 3 k=101 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.75	7.39	7.91	8.3
2.5	7.92	8.6	9.17	9.59
5	9.08	9.59	10.21	10.67
7.5	9.86	10.21	10.87	11.35
10	10.46	10.67	11.35	11.85
15	11.38	11.35	12.06	12.59
20	12.07	11.85	12.59	13.14
25	12.64	12.26	13.02	13.59
30	13.12	12.59	13.38	13.96
35	13.54	12.89	13.68	14.28
40	13.91	13.14	13.96	14.56
45	14.25	13.38	14.2	14.82
50	14.55	13.59	14.43	15.05
75	15.78	14.43	15.31	15.97
100	16.7	15.05	15.97	16.65

Pt = 3.0

CBR 3 k=101 EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.75	7.69	8.24	8.65
2.5	7.92	8.97	9.56	10
5	9.08	10	10.65	11.12
7.5	9.86	10.65	11.32	11.83
10	10.46	11.12	11.83	12.35
15	11.38	11.83	12.57	13.12
20	12.07	12.35	13.12	13.69
25	12.64	12.77	13.56	14.15
30	13.12	13.12	13.93	14.53
35	13.54	13.42	14.25	14.87
40	13.91	13.69	14.53	15.16
45	14.25	13.93	14.79	15.42
50	14.55	14.15	15.02	15.66
75	15.78	15.02	15.93	16.62
100	16.7	15.66	16.62	17.33

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 4.

Pt = 2.0

CBR 4, k=119 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.45	7.11	7.61	7.97
2.5	7.56	8.27	8.82	9.23
5	8.64	9.23	9.83	10.27
7.5	9.38	9.83	10.46	10.93
10	9.94	10.27	10.93	11.42
15	10.81	10.93	11.62	12.14
20	11.46	11.42	12.14	12.67
25	12	11.81	12.55	13.1
30	12.45	12.14	12.9	13.46
35	12.84	12.42	13.2	13.78
40	13.2	12.67	13.46	14.05
45	13.51	12.9	13.7	14.3
50	13.8	13.1	13.92	14.52
75	14.95	13.92	14.78	15.42
100	15.82	14.52	15.42	16.09

Pt = 2.5

CBR 4, k=119 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.45	7.31	7.84	8.23
2.5	7.56	8.53	9.11	9.53
5	8.64	9.53	10.15	10.61
7.5	9.38	10.15	10.8	11.29
10	9.94	10.61	11.29	11.79
15	10.81	11.29	12	12.53
20	11.46	11.79	12.53	13.08
25	12	12.19	12.95	13.52
30	12.45	12.53	13.31	13.89
35	12.84	12.82	13.62	14.21
40	13.2	13.08	13.89	14.5
45	13.51	13.31	14.14	14.75
50	13.8	13.52	14.36	14.98
75	14.95	14.36	15.24	15.91
100	15.82	14.98	15.91	16.59

Pt = 3.0

CBR 4, k=119 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.45	7.62	8.18	8.58
2.5	7.56	8.9	9.5	9.94
5	8.64	9.94	10.58	11.06
7.5	9.38	10.58	11.26	11.76
10	9.94	11.06	11.76	12.29
15	10.81	11.76	12.51	13.05
20	11.46	12.29	13.05	13.63
25	12	12.71	13.5	14.09
30	12.45	13.05	13.87	14.47
35	12.84	13.36	14.19	14.8
40	13.2	13.63	14.47	15.1
45	13.51	13.87	14.72	15.36
50	13.8	14.09	14.96	15.6
75	14.95	14.96	15.88	16.56
100	15.82	15.6	16.56	17.27

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 5.

Pt = 2.0

CBR 5, k=135 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.23	7.05	7.55	7.91
2.5	7.28	8.21	8.76	9.17
5	8.32	9.17	9.77	10.22
7.5	9.02	9.77	10.4	10.87
10	9.56	10.22	10.87	11.36
15	10.38	10.87	11.57	12.08
20	11.01	11.36	12.08	12.62
25	11.52	11.75	12.5	13.05
30	11.96	12.08	12.85	13.41
35	12.33	12.37	13.14	13.72
40	12.67	12.62	13.41	14
45	12.97	12.85	13.65	14.24
50	13.25	13.05	13.86	14.47
75	14.35	13.86	14.72	15.36
100	15.18	14.47	15.36	16.03

Pt = 2.5

CBR 5, k=135 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.23	7.25	7.78	8.17
2.5	7.28	8.47	9.05	9.47
5	8.32	9.47	10.09	10.55
7.5	9.02	10.09	10.75	11.23
10	9.56	10.55	11.23	11.73
15	10.38	11.23	11.95	12.48
20	11.01	11.73	12.48	13.03
25	11.52	12.14	12.9	13.47
30	11.96	12.48	13.26	13.84
35	12.33	12.77	13.57	14.16
40	12.67	13.03	13.84	14.44
45	12.97	13.26	14.09	14.7
50	13.25	13.47	14.31	14.93
75	14.35	14.31	15.19	15.85
100	15.18	14.93	15.85	16.53

Pt = 3.0

CBR 5, k=135 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC'	KY EALS	1.5 KY EALS	2 KY EALS
1	6.23	7.55	8.12	8.53
2.5	7.28	8.85	9.45	9.89
5	8.32	9.89	10.53	11.01
7.5	9.02	10.53	11.21	11.71
10	9.56	11.01	11.71	12.24
15	10.38	11.71	12.46	13.01
20	11.01	12.24	13.01	13.58
25	11.52	12.66	13.45	14.04
30	11.96	13.01	13.82	14.42
35	12.33	13.31	14.14	14.76
40	12.67	13.58	14.42	15.05
45	12.97	13.82	14.68	15.31
50	13.25	14.04	14.91	15.55
75	14.35	14.91	15.82	16.51
100	15.18	15.55	16.51	17.22

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 6.

Pt = 2.0

CBR 6
k=150

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.05	6.99	7.49	7.86
2.5	7.06	8.16	8.71	9.12
5	8.06	9.12	9.72	10.17
7.5	8.73	9.72	10.36	10.82
10	9.25	10.17	10.82	11.31
15	10.05	10.82	11.52	12.04
20	10.66	11.31	12.04	12.57
25	11.15	11.71	12.45	13
30	11.57	12.04	12.79	13.36
35	11.93	12.32	13.1	13.68
40	12.26	12.57	13.36	13.95
45	12.55	12.79	13.6	14.2
50	12.82	13	13.81	14.42
75	13.89	13.81	14.68	15.32
100	14.69	14.42	15.32	15.98

Pt = 2.5

CBR 6
k=150

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.05	7.2	7.73	8.12
2.5	7.06	8.42	9	9.43
5	8.06	9.43	10.05	10.51
7.5	8.73	10.05	10.7	11.19
10	9.25	10.51	11.19	11.69
15	10.05	11.19	11.9	12.43
20	10.66	11.69	12.43	12.98
25	11.15	12.09	12.86	13.42
30	11.57	12.43	13.21	13.8
35	11.93	12.73	13.52	14.12
40	12.26	12.98	13.8	14.4
45	12.55	13.21	14.04	14.66
50	12.82	13.42	14.26	14.89
75	13.89	14.26	15.15	15.81
100	14.69	14.89	15.81	16.49

Pt = 3.0

CBR 6
k=150

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	6.05	7.5	8.07	8.48
2.5	7.06	8.8	9.4	9.84
5	8.06	9.84	10.49	10.97
7.5	8.73	10.49	11.17	11.67
10	9.25	10.97	11.67	12.19
15	10.05	11.67	12.41	12.96
20	10.66	12.19	12.96	13.54
25	11.15	12.61	13.41	13.99
30	11.57	12.96	13.78	14.38
35	11.93	13.27	14.1	14.71
40	12.26	13.54	14.38	15
45	12.55	13.78	14.63	15.27
50	12.82	13.99	14.87	15.51
75	13.89	14.87	15.78	16.46
100	14.69	15.51	16.46	17.18

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 7.

Pt = 2.0

CBR 7
k=164

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.91	6.95	7.45	7.82
2.5	6.88	8.11	8.67	9.07
5	7.84	9.07	9.68	10.12
7.5	8.5	9.68	10.31	10.78
10	9	10.12	10.78	11.27
15	9.78	10.78	11.48	11.99
20	10.37	11.27	11.99	12.53
25	10.85	11.66	12.41	12.96
30	11.25	11.99	12.75	13.32
35	11.61	12.28	13.06	13.63
40	11.92	12.53	13.32	13.91
45	12.21	12.75	13.56	14.16
50	12.47	12.96	13.77	14.38
75	13.51	13.77	14.64	15.27
100	14.29	14.38	15.27	15.94

Pt = 2.5

CBR 7
k=164

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.91	7.15	7.69	8.07
2.5	6.88	8.38	8.96	9.38
5	7.84	9.38	10.01	10.47
7.5	8.5	10.01	10.66	11.15
10	9	10.47	11.15	11.65
15	9.78	11.15	11.86	12.39
20	10.37	11.65	12.39	12.94
25	10.85	12.05	12.82	13.39
30	11.25	12.39	13.17	13.76
35	11.61	12.68	13.48	14.08
40	11.92	12.94	13.76	14.36
45	12.21	13.17	14	14.62
50	12.47	13.39	14.22	14.85
75	13.51	14.22	15.11	15.77
100	14.29	14.85	15.77	16.45

Pt = 3.0

CBR 7
k=164

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.91	7.46	8.02	8.43
2.5	6.88	8.76	9.36	9.8
5	7.84	9.8	10.45	10.93
7.5	8.5	10.45	11.13	11.63
10	9	10.93	11.63	12.15
15	9.78	11.63	12.38	12.93
20	10.37	12.15	12.93	13.5
25	10.85	12.58	13.37	13.96
30	11.25	12.93	13.74	14.34
35	11.61	13.23	14.06	14.68
40	11.92	13.5	14.34	14.97
45	12.21	13.74	14.59	15.23
50	12.47	13.96	14.83	15.47
75	13.51	14.83	15.74	16.43
100	14.29	15.47	16.43	17.14

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 8.

Pt = 2.0

CBR 8, k=177 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.78	6.9	7.4	7.77
2.5	6.72	8.07	8.63	9.04
5	7.66	9.04	9.64	10.09
7.5	8.3	9.64	10.28	10.75
10	8.79	10.09	10.75	11.24
15	9.55	10.75	11.44	11.96
20	10.12	11.24	11.96	12.49
25	10.59	11.63	12.37	12.92
30	10.99	11.96	12.72	13.28
35	11.34	12.24	13.02	13.6
40	11.64	12.49	13.28	13.87
45	11.92	12.72	13.52	14.12
50	12.18	12.92	13.74	14.34
75	13.19	13.74	14.6	15.24
100	13.96	14.34	15.24	15.9

Pt = 2.5

CBR 8, k=177 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.78	7.11	7.64	8.03
2.5	6.72	8.34	8.92	9.35
5	7.66	9.35	9.97	10.43
7.5	8.3	9.97	10.63	11.11
10	8.79	10.43	11.11	11.61
15	9.55	11.11	11.83	12.36
20	10.12	11.61	12.36	12.91
25	10.59	12.02	12.78	13.35
30	10.99	12.36	13.14	13.72
35	11.34	12.65	13.45	14.04
40	11.64	12.91	13.72	14.33
45	11.92	13.14	13.97	14.58
50	12.18	13.35	14.19	14.81
75	13.19	14.19	15.07	15.73
100	13.96	14.87	15.73	16.42

Pt = 3.0

CBR 8, k=177 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.78	7.41	7.99	8.4
2.5	6.72	8.72	9.32	9.77
5	7.66	9.77	10.41	10.89
7.5	8.3	10.41	11.1	11.6
10	8.79	10.89	11.6	12.12
15	9.55	11.6	12.34	12.89
20	10.12	12.12	12.89	13.47
25	10.59	12.54	13.33	13.92
30	10.99	12.89	13.7	14.31
35	11.34	13.2	14.02	14.64
40	11.64	13.47	14.31	14.94
45	11.92	13.7	14.56	15.2
50	12.18	13.92	14.79	15.44
75	13.19	14.79	15.71	16.4
100	13.96	15.44	16.4	17.1

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 9.

Pt = 2.0

CBR 9, k=189 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.68	6.86	7.37	7.74
2.5	6.59	8.04	8.59	9
5	7.5	9	9.6	10.05
7.5	8.13	9.6	10.24	10.71
10	8.61	10.05	10.71	11.2
15	9.35	10.71	11.41	11.92
20	9.91	11.2	11.92	12.46
25	10.37	11.59	12.34	12.89
30	10.76	11.92	12.68	13.25
35	11.1	12.21	12.99	13.56
40	11.41	12.46	13.25	13.84
45	11.68	12.68	13.49	14.09
50	11.93	12.89	13.7	14.31
75	12.93	13.7	14.57	15.21
100	13.68	14.31	15.21	15.87

Pt = 2.5

CBR 9, k=189 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.68	7.07	7.61	8
2.5	6.59	8.31	8.89	9.31
5	7.5	9.31	9.94	10.4
7.5	8.13	9.94	10.59	11.08
10	8.61	10.4	11.08	11.58
15	9.35	11.08	11.79	12.33
20	9.91	11.58	12.33	12.88
25	10.37	11.99	12.75	13.32
30	10.76	12.33	13.11	13.69
35	11.1	12.62	13.42	14.01
40	11.41	12.88	13.69	14.29
45	11.68	13.11	13.94	14.55
50	11.93	13.32	14.16	14.78
75	12.93	14.16	15.04	15.7
100	13.68	14.78	15.7	16.39

Pt = 3.0

CBR 9, k=189 — AASHTO PCC THICKNESS, IN.

EAL 10^6	KY PCC*	KY EALS	1.5 KY EALS	2 KY EALS
1	5.68	7.37	7.95	8.36
2.5	6.59	8.69	9.29	9.73
5	7.5	9.73	10.38	10.86
7.5	8.13	10.38	11.06	11.57
10	8.61	10.86	11.57	12.09
15	9.35	11.57	12.31	12.86
20	9.91	12.09	12.86	13.43
25	10.37	12.51	13.3	13.89
30	10.76	12.86	13.67	14.28
35	11.1	13.17	14	14.61
40	11.41	13.43	14.28	14.91
45	11.68	13.67	14.53	15.17
50	11.93	13.89	14.76	15.41
75	12.93	14.76	15.68	16.37
100	13.68	15.41	16.37	17.08

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 10.

Pt = 2.0 — CBR 10, k=200

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.58	6.83	7.34	7.71
2.5	6.47	8	8.56	8.97
5	7.36	8.97	9.58	10.02
7.5	7.98	9.58	10.21	10.69
10	8.45	10.02	10.69	11.17
15	9.17	10.69	11.38	11.9
20	9.73	11.17	11.9	12.43
25	10.18	11.57	12.31	12.86
30	10.56	11.9	12.66	13.22
35	10.9	12.18	12.96	13.54
40	11.2	12.43	13.22	13.81
45	11.47	12.66	13.46	14.06
50	11.71	12.86	13.68	14.28
75	12.7	13.68	14.54	15.18
100	13.43	14.28	15.18	15.84

Pt = 2.5 — CBR 10, k=200

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.58	7.04	7.58	7.97
2.5	6.47	8.28	8.86	9.29
5	7.36	9.29	9.91	10.37
7.5	7.98	9.91	10.57	11.05
10	8.45	10.37	11.05	11.55
15	9.17	11.05	11.77	12.3
20	9.73	11.55	12.3	12.85
25	10.18	11.96	12.72	13.29
30	10.56	12.3	13.08	13.66
35	10.9	12.59	13.39	13.99
40	11.2	12.85	13.66	14.27
45	11.47	13.08	13.91	14.52
50	11.71	13.29	14.13	14.75
75	12.7	14.13	15.02	15.67
100	13.43	14.75	15.67	16.36

Pt = 3.0 — CBR 10, k=200

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.58	7.34	7.92	8.33
2.5	6.47	8.66	9.26	9.71
5	7.36	9.71	10.36	10.84
7.5	7.98	10.36	11.04	11.54
10	8.45	10.84	11.54	12.07
15	9.17	11.54	12.29	12.84
20	9.73	12.07	12.84	13.41
25	10.18	12.48	13.28	13.87
30	10.56	12.84	13.65	14.25
35	10.9	13.14	13.97	14.59
40	11.2	13.41	14.25	14.88
45	11.47	13.65	14.51	15.15
50	11.71	13.87	14.74	15.39
75	12.7	14.74	15.66	16.34
100	13.43	15.39	16.34	17.05

COMPARISON OF RIGID PAVEMENT THICKNESS DESIGNS FOR CBR 11.

Pt = 2.0 — CBR 11, k=212

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.5	6.79	7.3	7.67
2.5	6.36	7.97	8.53	8.94
5	7.24	8.94	9.54	9.99
7.5	7.84	9.54	10.18	10.65
10	8.31	9.99	10.65	11.14
15	9.02	10.65	11.35	11.86
20	9.57	11.14	11.86	12.4
25	10.01	11.53	12.28	12.83
30	10.39	11.86	12.63	13.19
35	10.72	12.15	12.93	13.51
40	11.02	12.4	13.19	13.78
45	11.28	12.63	13.43	14.03
50	11.52	12.83	13.65	14.25
75	12.49	13.65	14.51	15.15
100	13.22	14.25	15.15	15.82

Pt = 2.5 — CBR 11, k=212

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.5	7	7.54	7.93
2.5	6.36	8.25	8.83	9.25
5	7.24	9.25	9.88	10.34
7.5	7.84	9.88	10.54	11.02
10	8.31	10.34	11.02	11.53
15	9.02	11.02	11.74	12.27
20	9.57	11.53	12.27	12.82
25	10.01	11.93	12.7	13.26
30	10.39	12.27	13.05	13.63
35	10.72	12.56	13.36	13.96
40	11.02	12.82	13.63	14.24
45	11.28	13.05	13.88	14.49
50	11.52	13.26	14.1	14.73
75	12.49	14.1	14.99	15.65
100	13.22	14.73	15.65	16.33

Pt = 3.0 — CBR 11, k=212

EAL 10^6	KY PCC*	AASHTO PCC THICKNESS, IN. KY EALS	1.5 KY EALS	2 KY EALS
1	5.5	7.31	7.88	8.3
2.5	6.36	8.63	9.23	9.68
5	7.24	9.68	10.33	10.81
7.5	7.84	10.33	11.01	11.52
10	8.31	10.81	11.52	12.04
15	9.02	11.52	12.26	12.81
20	9.57	12.04	12.81	13.38
25	10.01	12.46	13.25	13.84
30	10.39	12.81	13.62	14.22
35	10.72	13.11	13.94	14.56
40	11.02	13.38	14.22	14.85
45	11.28	13.62	14.48	15.12
50	11.52	13.84	14.71	15.36
75	12.49	14.71	15.63	16.31
100	13.22	15.36	16.31	17.02