AASHO Road Test Equations Applied to the Design of Bituminous Pavements in Illinois

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•SINCE the completion of the AASHO Road Test Project, the Illinois Division of Highways has been studying the results and doing research directed towards developing practical applications of the findings. The research work has been done in cooperation with the U. S. Bureau of Public Roads. This work has culminated in the development of two interim structural design procedures, one for bituminous pavements and the other for portland cement concrete pavements.

This paper is concerned with the work done in applying the findings of the Road Test flexible pavement research to the structural design of bituminous pavements in Illinois. It presents background information and concepts used in developing the design procedure, describes the development of the procedure, and demonstrates its application.

The procedure provides for establishing the types and thicknesses of materials to be used in the various layers of the pavement structure consistent with the volume and composition of traffic, the length of time the pavement is to serve this traffic, the strength characteristics of the subgrade soils and pavement materials, and the minimum level of service to be provided by the pavement during its lifetime.

The AASHO Road Test flexible pavement performance equation serves as the basis of this design procedure. The equation explains performance of the test sections as related to pavement design, the magnitude and configuration of the axle load, and the number of axle-load applications. This equation necessarily is limited to the physical environment of the project; to the materials used in the test pavements; to the range in pavement thicknesses included in the experiment; to the axle loads, number of axle-load applications, and the specific times and rates of application of the test traffic; to the construction techniques employed; and to the climatic cycles experienced during construction and testing of the experimental facility. To apply the equation in the design of regular highway pavements, it was necessary to make certain assumptions and extrapolations based on experience and engineering judgment. As additional knowledge is gained through further research and experience, the precision of these assumptions and extrapolations should become sharpened. Therefore, the design procedure presented herein is provisional in nature and is subject to modification based on additional experience and research.

RESEARCH BACKGROUND INFORMATION

Pavement Serviceability-Performance Concept

Essential to the development of the Road Test equations was the establishment of a definition of pavement performance and a system for its measurement. The definition was founded on the basic principle that the prime function of a pavement is to serve the traveling public. The system of measurement that was developed establishes the degree

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to which the public considers itself to be served. This system has come to be known as the Pavement Serviceability-Performance System (1).

Under this concept, the term "present serviceability" was chosen to represent how well a highway pavement is serving high-volume, high-speed mixed truck and passenger vehicle traffic at a specific time. Performance was then said to be related to the ability of the pavement to serve traffic over a period of time.

The system of measuring present serviceability was derived through the use of the subjective serviceability ratings of a great number of typical pavements. The pavements were rated on a scale of zero to five by a panel of men selected to represent many important groups of highway users. A mathematical index (Present Serviceability Index) was then developed for estimating the subjective ratings from objective measurements taken on the pavement.

The following equation was developed to determine the level of serviceability of flexible pavement sections on the AASHO Road Test:

$$p = 5.03 - 1.91 \log (1 + \overline{SV}) - 0.01 \sqrt{C + P} - 1.38 \overline{RD}^2$$

in which

p = the present serviceability index;

$\overline{SV}$ = the mean of the slope variance in the two wheelpaths as measured by the AASHO longitudinal profilometer, $\times 10^6$;

$C + P$ = a measure of cracking and patching in the pavement surface; and

$\overline{RD}$ = a measure of rutting in the wheelpaths.

By relating the results of the AASHO profilometer and Illinois roadometer, the present serviceability index equation becomes:

$$p = 10.91 - 3.90 \log \overline{RI} - 0.01 \sqrt{C + P} - 1.38 \overline{RD}^2$$

in which

$\overline{RI}$ = Roughness index in inches per mile, as obtained by the Illinois roadometer.

Performance of a pavement is then determined by relating its serviceability records to the corresponding numbers of axle-load applications.

Performance Equation from AASHO Road Test

Present Serviceability Index (PSI) values were determined every two weeks for each Road Test section. Serviceability trends were developed for the sections by plotting the PSI values against the corresponding axle-load applications. These trends represent the performance of the pavement sections. An equation was then derived to express the shape of the serviceability trend curves in terms of design thickness, axle load and its configuration, and number of axle-load applications. The performance equation (2) developed for the flexible pavement sections is

$$G_t = \log \frac{c_o - P_t}{c_o - 1.5} = \beta (\log W_t - \log \rho)$$

in which

G_t = a function (the logarithm) of the ratio of loss in serviceability at time t to the total potential loss taken to the point where p = 1.5, the point at which pavement sections were removed from test in the AASHO Road Test;

c_o = initial serviceability of pavement (equal to 4.2 on test road);

p_t = serviceability at end of time t;

β = a function of design and load variables that influences the shape of the p versus W_t performance curve;

W_t = number of axle-load applications; and

ρ = a function of design and load variables that denotes the expected number of axle-load applications to a serviceability index of 1.5.

For weighted axle-load applications, expressions for β and ρ are as follows:

$$\log(\beta - 0.4) = \log 0.081 + 3.23 \log(L_1 + L_2) - 5.19 \log(D + 1) - 3.23 \log L_2$$

and

$$\log \rho = 5.93 + 9.36 \log(D + 1) - 4.79 \log(L_1 + L_2) + 4.33 \log L_2$$

in which

L_1 = load on one single-load axle or on one tandem-axle set, kips;

L_2 = axle code (1 for single axle; 2 for tandem axle); and

D = thickness index.

The thickness index is a function of the various thicknesses of the layers that constitute the pavement structure expressed as a single number. This thickness index, D, is as follows:

$$D = a_1D_1 + a_2D_2 + a_3D_3$$

in which

a_1, a_2, a_3 = coefficients of relative strength of surface, base, and subbase as related to performance (for the Road Test $a_1 = 0.44$, $a_2 = 0.14$, and $a_3 = 0.11$);

D_1 = thickness of bituminous surface course in inches;

D_2 = thickness of base course in inches; and

D_3 = thickness of subbase in inches.

Equivalent Axle Load Concept

As previously stated, the Road Test equations express the performance of the test sections in terms of pavement design, axle load and configuration, and number of axle-load applications. The term W_t in the performance equations denotes the number of axle-load applications of a given magnitude and configuration. This was possible on the Road Test because the traffic on any one test section had identical axle loads and arrangements.

Before any attempt could be made to apply the equations for design purposes, it was necessary to reduce normal mixed-traffic axle loadings to some common denominator, or basic loading. The system developed reduces mixed-traffic axle-load applications to an equivalent number of 18-kip (18,000-lb) single-axle load applications. The selection of 18-kip single-axle load applications as the common denominator has no particular significance except that 18,000 pounds is the legal single-axle load limit in Illinois.

This system makes use of "equivalency factors" that were derived from the Road Test performance equations. The equivalency factor for any given axle load expresses the number of applications of an 18-kip single-axle load equivalent to one application of the given axle load.

Mixed-traffic axle loadings can be reduced to the common denominator, or basic loading, by grouping the individual axles in the traffic stream into various weight and configuration categories. The sum of the products of the equivalency factors times the corresponding numbers of axles in the various categories gives the total number of equivalent 18-kip single-axle load applications in the traffic stream.

DEVELOPMENT OF DESIGN PROCEDURE

This procedure for the structural design of bituminous pavements in Illinois has been prepared on the basis of the findings of the AASHO Road Test supplemented with the results of research studies (3) conducted by the Illinois Division of Highways. The procedure reflects engineering experience and judgment of the Division, and recommendations of the AASHO Committee on Design (4).

The design of a pavement structure requires the compilation and correlation of the following factors:

1. Volume and axle-load distribution of the traffic that the pavement will be expected to carry;
2. Type and strength of the roadbed soil on which the pavement will be built;
3. Length of time and quality of service expected from the pavement;
4. Environmental and climatic conditions of the region where the pavement is to be built; and
5. Relative ability of the available pavement materials to support loads.

All of these factors have been taken into consideration in the development of this design procedure. Variations in climatic conditions as they exist from one part of the State to another, and particularly between the extreme northern and extreme southern portions, undoubtedly affect pavement performance. However, the relative effects of these variations on pavement performance are not sufficiently distinguishable at the present state of knowledge to be taken into account in pavement structural design. Therefore, climatic effects have been considered only on a statewide basis. This involved the assumption that, based on present knowledge, climatic conditions throughout Illinois do not differ sufficiently from those of the Ottawa area (site of the AASHO Road Test) to cause a significant difference in the required structural design. The remaining factors are included in the design charts and equations.

The charts and equations included in the design procedure were developed from the AASHO Road Test flexible pavement performance equation. Modifications to the performance equations were made to reflect in the structural design the effects on pavement performance of the following:

1. Mixed-traffic (truck and passenger car) axle loadings when compared to the controlled traffic axle loadings on the Road Test;
2. Pavements subjected to traffic over long periods of time when compared to the two years of traffic on the Road Test;
3. Variations in the support strengths of the roadbed soils; and
4. Variations in the strength characteristics of the pavement structure materials.

Mixed-Traffic Axle Loadings

To evaluate the effects of mixed-traffic axle loadings on pavement performance, a system was developed to convert these loadings into a "traffic factor." The traffic factor is the total number of equivalent 18-kip single-axle load applications (in millions) estimated to be generated by the traffic a pavement may be expected to carry throughout its entire service life.

In developing the system, use was made of "equivalency factors" for various groupings of single- and tandem-axle loadings determined from the Road Test equation, and statewide loadometer survey data and classification counts at loadometer stations dating back to 1936 and as recent as 1962. The equivalency factor for any given single- or tandem-axle load expresses the number of 18-kip single-axle load applications that will have the same effect on pavement performance as one application of the given axle load. The loadometer and traffic count data were used to determine the distribution of single- and tandem-axle weights for the various classifications of vehicles in the mixed-traffic stream.

Preliminary analyses demonstrated the need to give special consideration to average axle loadings as they exist for the various individual classifications of commercial vehicles. Variations in the distribution of vehicle classifications in the commercial traffic stream from one highway to another are too great to permit the use of a statewide average commercial vehicle in evaluating the effects of mixed-traffic axle loadings on pavement performance. In the final analysis, consideration was given to the differences in average axle loadings as they exist for passenger cars, single units (all 2- and 3-axle single-unit trucks and all buses), and multiple units (3-axle, 4-axle, and 5-axle truck-tractor semitrailers and all full-trailer combinations).

The preliminary analyses also indicated the need for considering the differences in average axle weights of both single and multiple units operating on highways ranging from high-volume major highways with heavy commercial hauling to low-volume local roads with farm-to-market type hauling. To accomplish this the entire highway system was divided into four general classifications:

1. Class I Roads and Streets–roads and streets being designed as four-lane or more facilities, or as part of future four-lane or more facilities;
2. Class II Roads and Streets–roads and streets with structural design traffic greater than 1,000 ADT and being designed as two-lane or three-lane facilities;
3. Class III Roads and Streets–roads and streets with structural design traffic between 400 and 1,000 ADT; and
4. Class IV Roads and Streets–roads and streets with structural design traffic less than 400 ADT.

These classifications were selected so that, in general, Class I represents the Interstate and expressway system, Class II the primary system, Class III the secondary system, and Class IV the local roads.

The results of the AASHO Road Test provided a means of developing equivalency factors for converting any given single- or tandem-axle load into an equivalent number of 18-kip single-axle load applications relative to its effect on pavement performance. The equivalency factor may be expressed as follows:

$$\left(\begin{array}{c}\text{18-kip single-axle}\\ \text{equivalency}\\ \text{factor}\end{array}\right) = \left(\frac{\text{No. of 18-kip single-axle load applications to a given present serviceability index}}{\text{No. of x-kip application to the same given present serviceability index}}\right)$$

This factor was developed by the following mathematical analysis:

$$\log W_t = \log \rho + \frac{G_t}{\beta} \qquad (1)$$

or

$$\log W_t = 5.93 + 9.36 \log (D + 1) - 4.79 \log (L_1 + L_2) + 4.33 \log L_2 + \frac{G_t}{\beta} \qquad (2)$$

When L_1 = 18-kip and L_2 = 1 (single axles), then

$$\log W_{t_{18}} = 5.93 + 9.36 \log (D + 1) - 4.79 \log (18 + 1) + \frac{G_t}{\beta_{18}} \tag{3}$$

When L_1 = x kips and L_2 = 1 (single axles), then

$$\log W_{t_x} = 5.93 + 9.36 \log (D + 1) - 4.79 \log (x + 1) + \frac{G_t}{\beta_x} \tag{4}$$

Subtracting Eq. 4 from Eq. 3 the equivalency factor for single loads becomes

$$\log \frac{W_{t_{18}}}{W_{t_x}} = 4.79 \log (x + 1) - 4.79 \log (18 + 1) + \frac{G_t}{\beta_{18}} - \frac{G_t}{\beta_x} \tag{5}$$

Similarly, when L_1 = x and L_2 = 2 (tandem axles)

$$\log W_{t_x} = 5.93 + 9.36 \log (D + 1) - 4.79 \log (x + 2) + 4.33 \log (2) + \frac{G_t}{\beta_x} \tag{6}$$

Then, subtracting Eq. 6 from Eq. 3, the equivalency factor for tandem-axle loads becomes

$$\log \frac{W_{t_{18}}}{W_{t_x}} = 4.79 \log (x + 2) - 4.79 \log (18 + 1) - 4.33 \log (2) + \frac{G_t}{\beta_{18}} - \frac{G_t}{\beta_x} \tag{7}$$

The terms W_t, β, G_t, and ρ are as previously defined.

The ratios between $W_{t_{18}}$ and W_{t_x} in Eqs. 5 and 7 express the relationship between an 18-kip single-axle load and any other axle load (x). As shown, the equivalency factors vary with pavement design and serviceability level as well as with axle load and axle configuration. Therefore, averages of the values obtained for designs varying from D = 1.0 to D = 6.0, and for present serviceability, levels of 2.0 and 2. 5 have been

TABLE 1

18-KIP SINGLE-AXLE EQUIVALENCY FACTORS

Single-Axle Load (kips)	18-Kip Single-Axle Equivalency Factor		Tandem-Axle Load (kips)	18-Kip Single-Axle Equivalency Factor	
	p = 2.0	p = 2.5		p = 2.0	p = 2.5
2	0.0002	0.0003	4	0.0003	0.0005
4	0.0022	0.0032	8	0.0030	0.0054
6	0.0100	0.0133	12	0.0133	0.0167
8	0.0333	0.0383	16	0.0450	0.0550
10	0.0817	0.0950	20	0.1117	0.1300
12	0.1750	0.1967	24	0.2417	0.2667
14	0.3383	0.3633	28	0.4683	0.4983
16	0.6017	0.6217	32	0.8267	0.8533
18	1.0000	1.0000	36	1.3800	1.3800
20	1.5800	1.5333	40	2.1717	2.1133
22	2.3917	2.2667	44	3.2900	3.1183
24	3.5000	3.2433	48	4.8150	4.4617

used. The 18-kip equivalency factor was determined for each 2, 000-lb increment of load for single axles and for each 4, 000-lb increment of load for tandem axles (Table 1).

These factors were used in combination with loadometer survey data and traffic classification-count data to reduce mixed traffic to a fixed number of 18-kip equivalent single-axle load applications. Loadometer data, dating from 1945 to 1962, were available from 19 loadometer stations located on the primary system of highways in Illinois (Class I and Class II roads and streets). Traffic classification-count data were available for highways carrying traffic volumes corresponding to Class I, Class II, and Class III. Neither loadometer data nor adequate traffic classification-count data were available for highways carrying traffic volumes corresponding to Class IV.

The loadometer data were adjusted in accordance with the traffic classification-count data to provide more representative samples since only a small percentage of vehicles were weighed. The adjusted data provided the distribution of single and tandem axles in each weight group for each classification of vehicle type on Class I and Class II. The axle-load equivalency factors (Table 1) were then applied to these distributions to determine the number of 18-kip equivalent single-axle load applications per passenger car, per average single unit, and per average multiple unit for each of the two classifications. The factors corresponding to a terminal serviceability level of 2. 5 were used in connection with the determinations for Class I. Those factors corresponding to a terminal serviceability level of 2. 0 were used for Class II.

A study of the traffic classification-count data for Class III roads and streets disclosed that the total percent of single and multiple units was not significantly different from that on Class I and Class II, but a larger portion consisted of single units classified as smaller types of vehicles. Since loadometer data were not available for Class III, it was assumed that the distribution of axle loadings for each individual classification of vehicle within the single-unit and the multiple-unit groupings was the same as that for Class I and Class II. The loadometer data for Class I and Class II were then adjusted in accordance with the traffic classification count data for Class III. The axle-load equivalency factors for a terminal serviceability of 2. 0 were applied to the adjusted data to determine 18-kip equivalent single-axle load applications per passenger car, per average single unit, and per average multiple unit for Class III.

As previously stated, neither loadometer data nor adequate traffic classification-count data were available for highways carrying traffic volumes corresponding to Class IV roads and streets. This made it impossible to determine the performance of these highways and to correlate this performance to the Road Test equation. Thus, to extend the design procedure to include Class IV, it was necessary to assume a basic structural design and traffic loading consistent with previous experience.

The 18-kip equivalent single-axle load application factors per vehicle classification for Class I, Class II, Class III, and Class IV are given in Table 2. The results of the analysis yielded two important facts regarding reducing mixed traffic to a number of equivalent 18-kip single-axle load applications: (a) the effect of passenger cars is small in proportion to the effect of single and multiple units; and (b) the effect of multiple units is eight to nine times greater than the effect of single units. Thus, the total number of equivalent 18-kip single-axle load applications to be generated by mixed traffic can depend more on the distribution of the various classifications of vehicles in the traffic stream than on the total volume of traffic.

The values in Table 2 were used in developing equations to convert mixed traffic into a traffic factor for use in structural design. Special attention was given to the structural design traffic and to the number of single units and multiple units per day in the design lane. While the structural design traffic represents an estimate of the average daily traffic in both directions that will be carried by the highway facility, the pavement structural design will be based on the lane (design lane) carrying the greatest number of single and multiple units. Based on traffic placement studies, the number of vehicle per day in the design lane may be estimated by multiplying the structural design traffic by the percentage distributions given in Table 3.

Traffic factor equations (Table 4) were developed for the four classifications of roads and streets. They were developed from the following model:

$$TF = DP\left[\frac{(c_1 \times PC \times P) + (c_2 \times SU \times S) + (c_3 \times MU \times M)}{1,000,000}\right]$$

in which

- TF = traffic factor;
- DP = design period, years;
- c_1 = constant for passenger cars = value in Table 2 × 365;
- c_2 = constant for single units = value in Table 2 × 365;
- c_3 = constant for multiple units = value in Table 2 × 365;
- PC = total passenger car ADT, two directions;
- SU = total single-unit ADT, two directions;
- MU = total multiple-unit ADT, two directions;
- P = percent of passenger car ADT in design lane;
- S = percent of single-unit ADT in design lane; and
- M = percent of multiple-unit ADT in design lane.

Performance of Existing Pavements vs Predicted Performance

After developing a system for handling mixed-traffic axle loadings, the performance equation was tested for applicability to Illinois pavements in regular service. This was done by comparing the actual performance of selected pavements with performance as predicted by the equation. The pavements included in the studies were selected on the basis of the subgrade soil, pavement materials, and climatic conditions being similar to those that existed on the Road Test.

TABLE 2

EQUIVALENT 18-KIP SINGLE-AXLE LOAD APPLICATIONS PER VEHICLE CLASSIFICATION

Road and Street Classification	18-Kip Equivalent Single-Axle Load per Vehicle		
	Passenger Cars	Single Units	Multiple Units
Class I	0.0004	0.117	0.947
Class II	0.0004	0.109	0.924
Class III	0.0004	0.098	0.794
Class IV	0.0004	0.027	0.216

TABLE 3

AVERAGE LANE DISTRIBUTION OF STRUCTURAL DESIGN TRAFFIC

No. Lanes in Pavement Facility	Structural Design Traffic	
	% of Single and Multiple Units in Design Lane	% of Passenger Cars in Design Lane
2 or 3	50	50
4	45	32
6 or more	40	20

TABLE 4

TRAFFIC FACTOR (TF) EQUATIONS[a]

Road and Street Classification	Equation
Class I	$TF = DP\left[\frac{(0.146\ PC \times P) + (42.705\ SU \times S) + (345.655\ MU \times M)}{1,000,000}\right]$
Class II	$TF = DP\left[\frac{(0.146\ PC \times P) + (39.785\ SU \times S) + (337.260\ MU \times M)}{1,000,000}\right]$
Class III	$TF = DP\left[\frac{(0.146\ PC \times P) + (35.770\ SU \times S) + (289.810\ MU \times M)}{1,000,000}\right]$
Class IV	$TF = DP\left[\frac{(0.146\ PC \times P) + (\ 9.855\ SU \times S) + (\ 78.840\ MU \times M)}{1,000,000}\right]$

[a]Definition of terms is as given for model equation on p. 10.

The actual performance of each selected pavement was established by determining the present serviceability index at the time of the study and the total number of equivalent 18-kip single-axle load applications representing the traffic carried by the pavement to this point in time. The present serviceability index was determined from roadometer measurements and a patching and cracking survey. The total number of equivalent 18-kip single-axle load applications was determined from the recorded numbers of passenger cars, single units, and multiple units, and the developed 18-kip equivalency factors for these three vehicle classifications (Table 2).

The analyses of the data from the selected pavements showed that the Road Test performance equation cannot be applied directly, as it predicts, on the average, higher levels of performance than were actually obtained. However, there was evidence of definite trends which indicated that performance of the selected pavements agrees closely with the performance of pavements on the Road Test of lesser thickness. This suggested the hypothesis that the general form of the performance equation is applicable, and that the equation could be suitably modified for practical application in structural design by developing a factor for adjusting the design thickness term in the equation for ρ and β. This factor has been termed a time-traffic exposure factor, T.

The relationship between Road Test pavement thickness design and Illinois pavement thickness design that can be expected to give the same performance is

$$D = \frac{D_t}{T}$$

where

D = Road Test thickness index;
D_t = Illinois structural number; and
T = Time-traffic exposure factor.

It should be noted that the time-traffic exposure factor is considered to modify the Road Test equation only so as to be more representative of the behavior of pavements serving under similar conditions but over periods of time more typical of regular service life.

A total of 63 pavement sections were included in this study. They included both bituminous concrete on granular base (flexible base pavement) and bituminous-concrete-resurfaced portland cement concrete pavement (composite pavement). The flexible base pavement designs included 4.5 in. of bituminous-concrete surface course on 14 in. of crushed stone base and 5 in. of gravel subbase; and 4.5 in. of bituminous concrete on 9 in. of crushed stone base and 11 in. of gravel subbase. The composite pavement designs represented 2, 2.5, 3, and 4½ in. of bituminous-concrete resurfacing over uniform thicknesses of existing portland cement concrete pavement of 7, 8, 9, and 10 in., and over existing concrete pavement having thickened edge designs of 7-8-7, 9-6-9, 9-7-9, 9-9-7-9-9, and 10-10-8-10-10 in.

For each of the pavements, the time-traffic exposure factor was determined by dividing the Illinois structural number, D_t, by the thickness index, D, of the Road Test pavement capable of carrying the same number of equivalent 18-kip single-axle load applications to the same level of serviceability. In determining the structural number, the following equations were used.

For flexible base pavement:

$$D_t = a_1D_1 + a_2D_2 + a_3D_3$$

For composite pavement:

$$D_t = a_1D_1 + a_2D_2$$

in which

D_t = Illinois structural number;
a_1, a_2, a_3 = coefficients of relative strength of the surface, base and subbase, respectively; and

D_1, D_2, D_3 = thicknesses in inches of the surface, base, and subbase, respectively; for composite pavement, D_2 = thickness of the existing slab.

Values of the coefficients a_1, a_2, and a_3 for the pavements included in this study are as follows:

Surface course	
Bituminous-concrete subclass I-11	a_1 = 0.40
Base course	
Crushed stone, Grade 8	a_2 = 0.13
Existing PCC slab	a_2 = 0.40*
Subbase course	
Gravel, Grade 7	a_3 = 0.12

The thickened edge slabs were converted to effective uniform thicknesses for use with the structural number equation by a procedure which makes use of Westergaard's equation for corner loading, as follows:

Slab Thickness (in.)	Effective Thickness (in.)
7-8-7	7.00
9-6-9	7.06
9-7-9	7.71
9-9-7-9-9	8.75
10-10-8-10-10	9.75

The results of the analyses for the 63 pavement sections are shown in Figure 1, where the time-traffic exposure factor has been plotted against pavement age in years. The data represented by circles include composite pavements that had previously been retired by resurfacing a second time, and the terminal serviceability index of these pavements was assumed to be 2.0. The present serviceability index values of all other pavements were computed from roadometer measurements and patching and cracking surveys.

A linear regression line was fitted to the data which indicated an increase in the T-factor with pavement age. The results of the regression analysis indicated that only about 10 percent of the variation in the data is explained by pavement age. Therefore, the mean value of the data was used. The mean value is 1.11; a value of T = 1.10 was used in developing the design nomographs (Figs. 2 and 3).

Roadbed Soils

Only one soil type was used on the AASHO Road Test. The upper three feet of embankment under all pavement test sections was constructed with a clay A-6 soil having a Group Index between 9 and 13. This made it necessary to develop a means of modifying the results obtained from the AASHO Road Test flexible pavement performance equation to permit the establishment of pavement designs for other types of soil.

The soil support CBR scales (Figs. 2 and 3) represent the modification that has been made to take into consideration changes in support strength of roadbed soils. The

*Mean value determined from analysis of data from Illinois composite pavements. Also, it is the value suggested for use in the Manual of Instructions for Pavement Evaluation Survey (Aug. 1962) by the AASHO Committee on Highway Transport.

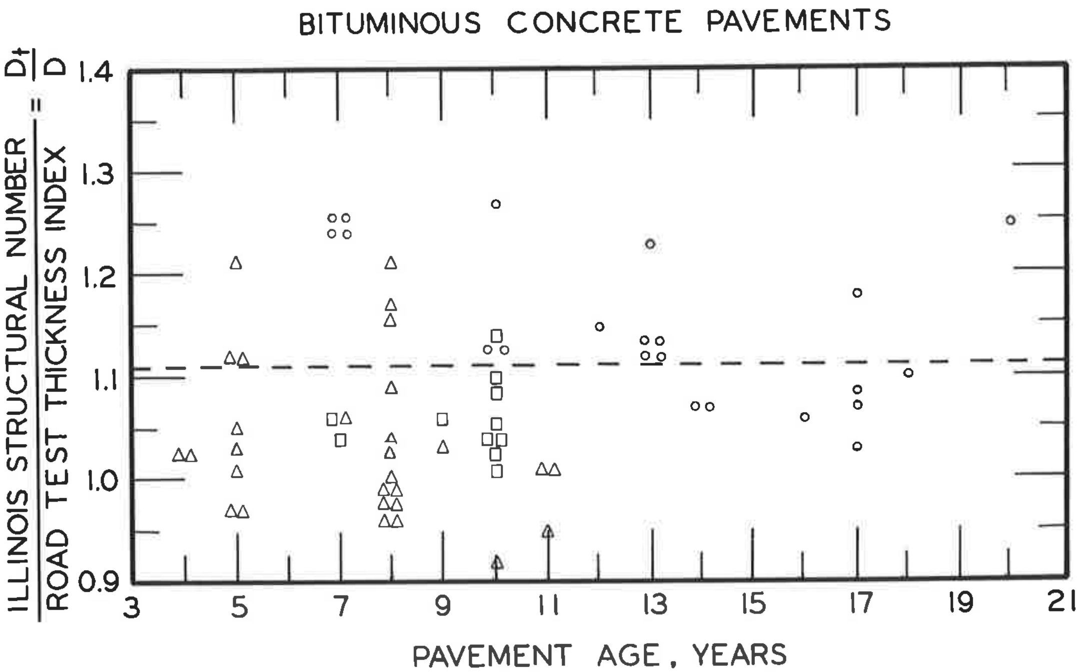

Figure 1. Time-traffic exposure factor vs pavement age.

scales were developed on the basis of the recommendations of the AASHO Committee on Design (4) and on the results of laboratory CBR tests conducted by the Illinois Division of Highways on the AASHO Road Test materials. The results of the laboratory tests indicated CBR values of 3.0 for the A-6 soil and 110 for the Road Test crushed-stone base course material.

In plotting the soil support CBR scale, only one point (CBR = 3.0) was obtained directly from the performance equation. A second point in Figure 3 was obtained by the procedures recommended by the AASHO Committee on Design. The Committee studied the performance of several sections having the greatest thickness of crushed-stone base that were on the loop carrying the 18-kip single-axle loads. The study indicated that approximately 4.5 in. of bituminous concrete on a sufficient thickness of crushed stone to minimize the effects of the roadbed soils should carry approximately 1,000 18-kip single-axle load applications per day for a 20-yr period, and at the same time, retain a present serviceability level at or above 2.0 for the entire period. The Road Test thickness index, D, for this section is 1.98 (4.5 x 0.44) which is equivalent to a value of 2.18 on the structural number scale included in Figure 3 (1.98 x 1.1). The 1,000 18-kip single-axle load applications per day for 20 years is equivalent to a value of 7.3 on the traffic factor scale of Figure 3.

Thus, a second or maximum point on the soil support CBR scale of Figure 3 was established by projecting a line through 2.18 on the structural number scale and 7.3 on the traffic factor scale. The intersection of this line with the soil support CBR scale was assigned a value of 110, corresponding to the results of Illinois CBR tests on the Road Test crushed-stone material. This point represents the supporting value of soils having the support characteristics of the crushed-stone base material used on the AASHO Road Test. A logarithmic scale between CBR values of 3.0 and 110 was assumed and extended to 1.

Similar procedures were used to establish a second or maximum point on the soil support CBR scale of Figure 2. A Road Test thickness index of 1.98 on a crushed stone embankment is equivalent in performance to a Road Test thickness index of 4.9 on the A-6 soil embankment. The total 18-kip single-axle load applications that can be

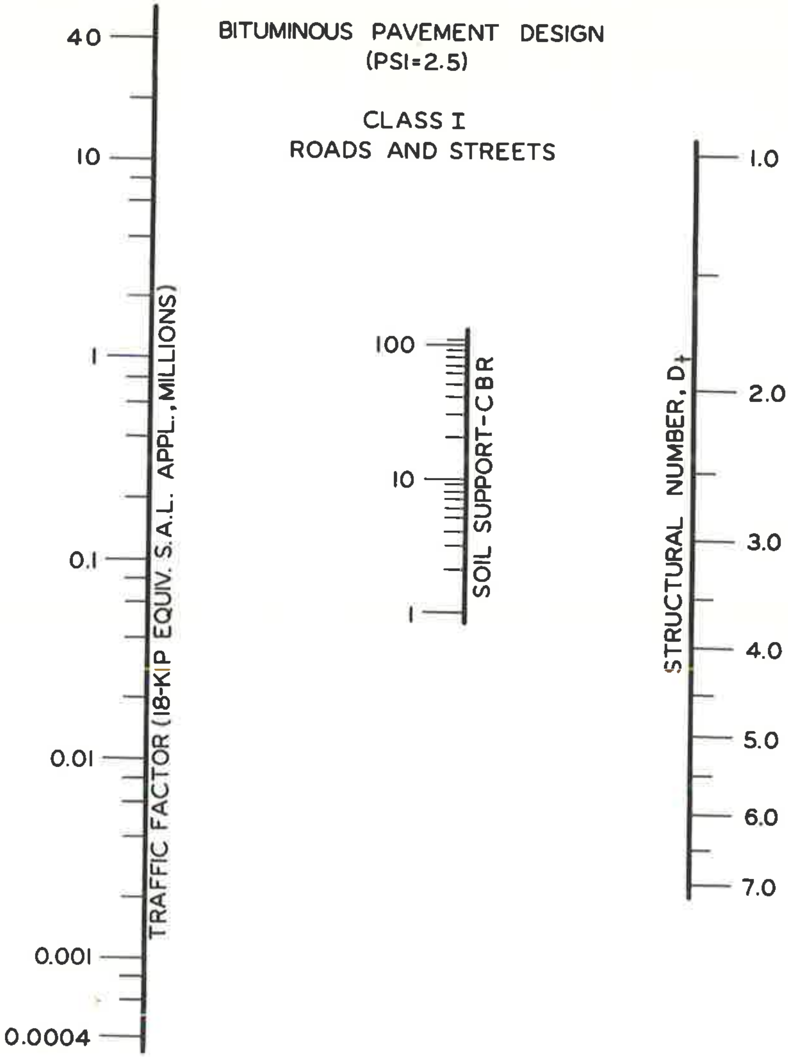

Figure 2. Design nomograph.

carried by these designs to PSI = 2.5 is 4,500,000, which is equivalent to a traffic factor of 4.5. A line passing through 2.18 on the structural number scale and 4.5 on the traffic factor scale of Figure 2 also intersects the soil support CBR scale at CBR = 110.

The soil support CBR value selected for use by the designer should represent a minimum value for the soil to be used. Preferably, laboratory tests should be made on 4-day soaked samples of the soils to be used in construction. In the event that actual test data cannot be obtained the minimum values given in Table 5 are recommended for use.

Design Charts

The design charts (Figs. 2 and 3) include a traffic factor scale, a soil support scale, and a structural number scale. They represent graphic presentations of the AASHO Road Test flexible pavement performance equation as modified for Illinois use. Figure

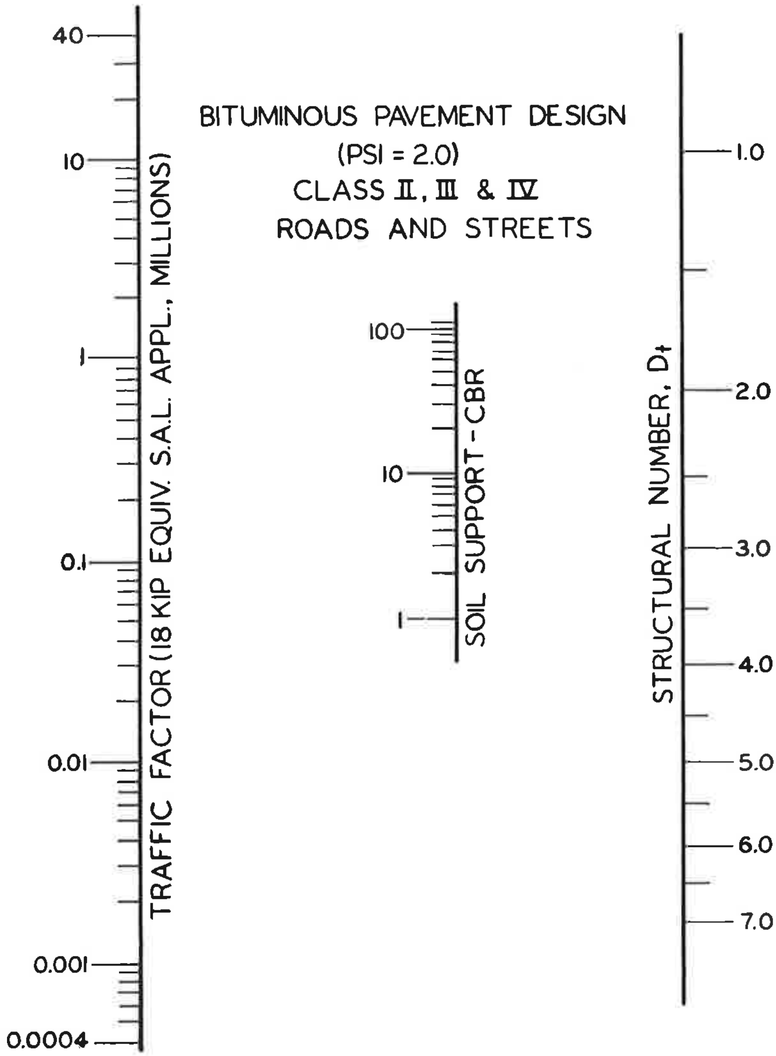

Figure 3. Design nomograph.

2 is for use in determining the pavement structural design for Class I roads and streets (Interstate highways and expressways); Figure 3, for Classes II, III, and IV.

The basic difference between the two charts is the terminal serviceability level assumed in the development. Figure 2 is based on a terminal serviceability level of 2. 5; Figure 3, on 2. 0. Selection of these levels was based on the average level of retirement throughout the nation, the level at which pavements are being retired in Illinois, and recommendations of the AASHO Committee on Design.

The terminal serviceability level of 2. 0 is representative of the average level at which pavements are being retired throughout the nation. This level was determined by a survey conducted in 1961 by the Bureau of Public Roads in cooperation with the state highway departments at the request of the AASHO Committee on Highway Transport (5). An Illinois study of the terminal serviceability level of highway pavements has fairly well substantiated this value as an average value for Illinois. However, pavements of four-lane divided expressways are being retired at serviceability levels above 2. 0, and generally averaging 2. 4. Further, the AASHO Committee on Design (4) has recommended that the design period for major highways be considered ended

TABLE 5

SUGGESTED MINIMUM SOIL SUPPORT CBR VALUES

Soil Classification	CBR Value*
A-1	20
A-2-4, A-2-5	15
A-2-6, A-2-7	12
A-3	10
A-4, A-5, A-6	3
A-7-5, A-7-6	2

*Values obtained by the CBR test procedure used by the Illinois Division of Highways; test specimens prepared by the static method of compaction using 2,000-psi pressure, and soaked for four days before testing (HRB Proc., Vol. 22, 1942, pp. 124-129).

at a present serviceability index of 2.5. For these reasons the design requirements have been based on a terminal serviceability level of 2.5 for Class I and 2.0 for all others.

Pavement Structure Materials

The developed design procedure reflects the pavement structure thickness in terms of a structural number. The structural number is related to the thickness of the various layers of the pavement structure as follows:

For pavements with granular and stabilized granular base courses:

$$D_t = a_1D_1 + a_2D_2 + a_3D_3$$

TABLE 6

MINIMUM COEFFICIENTS FOR PAVEMENT STRUCTURE MATERIALS

Materials	Minimum Strength Requirements			Coefficients[b]		
	MS[a]	CBR	PSI	a_1	a_2	a_3
Bituminous surface, subclass:						
B-1, B-2, B-3, and B-4	300			0.20		
B-5 and J-1	900			0.30		
I-11	1,700			0.40		
Base course:						
Granular						
Gravel, Grade 7		50			0.10	
Gravel, Grade 9		70			0.12	
Crushed stone, Grade 8		90			0.13	
Waterbound macadam		110			0.14	
Selected soil stabilized w/PC			300[c]		0.15	
Granular material stabilized w/PC,			450[c]		0.20	
plant mix			650[c]		0.23	
Granular material stabilized w/lime-			450[c]		0.20	
fly ash			650[d]		0.23	
Granular material stabilized w/bit. materials:						
Emulsified asphalts	300				0.16	
Liquid asphalts	400				0.18	
Paving asphalts	600				0.20	
	900				0.24	
	1,700				0.33	
PC concrete (new)			2,500+[c]		0.50	
Subbase:						
Gravel						
Grade 11		30				0.11
Grade 7		50				0.12
Grade 9		70				0.13
Crushed stone						
Grade 8		90				0.14

[a]Marshall stability or equivalent. [b]These coefficients may be considered as minimums for the materials listed. For use of materials with minimum strengths in excess of those given above, the coefficients may be determined from Figures 4 through 9. Other approved materials of similar strengths may be substituted. [c]7-day compressive strength (value that can be reasonably expected under field conditions). [d]21-day compressive strength (value that can be reasonably expected under field conditions).

For pavements with portland cement concrete base course:

$$D_t = a_1D_1 + a_2D_2$$

where

D_t = the structural number;
a_1, a_2, and a_3 = coefficients of relative strength of the surface course, base course, and subbase, respectively;
D_1 = thickness of surface course in inches;
D_2 = thickness of base course in inches; and
D_3 = thickness of subbase in inches.

The modifications that were developed to account for differences in strength characteristics of the pavement structure materials were based on the hypothesis that the value of a coefficient for a particular layer of the pavement structure is not constant, but will vary in accordance with the strength of the material selected for use in that layer. Relationships between coefficient values and material strengths determined by test procedures used by the Illinois Division of Highways were established for surface course, base course, and subbase materials. Experience with the materials, coefficients developed on the Road Test, and the results of tests were used in establishing these relationships.

Minimum coefficient values selected for the materials normally used in Illinois are given in Table 6. Values for materials other than those listed can be estimated from the relationships that follow. Additional research is being planned to validate these relationships.

Surface Course. — The coefficient for the surface course (a_1) was correlated with Marshall stability values (Fig. 4). The upper value represents the bituminous concrete on the Road Test. The value of a_1 for this material is 0.44, and the results of tests by the Division indicated a Marshall stability of 2,100 lb. The lower point represents a low stability road mix where values of a_1 and Marshall stability were assumed to be 0.20 and 300, respectively. The intermediate point represents the Division's bituminous concrete subclass I-11. A value of 0.40 was assumed for a_1 and a minimum Marshall stability value was taken as 1,700.

Base Course. —The relationship between the coefficient a_2 and material strengths has been developed for four general categories of base course (a_2): granular materials, granular materials stabilized with bituminous materials, granular materials stabilized with portland cement, and granular materials stabilized with lime-fly ash.

Figure 5 shows the relationship developed between coefficients for granular base materials and laboratory CBR values. The upper limit represents the Road Test

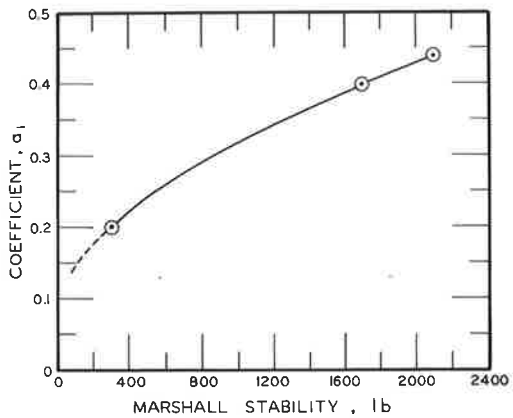

Figure 4. Coefficient for bituminous concrete surface course.

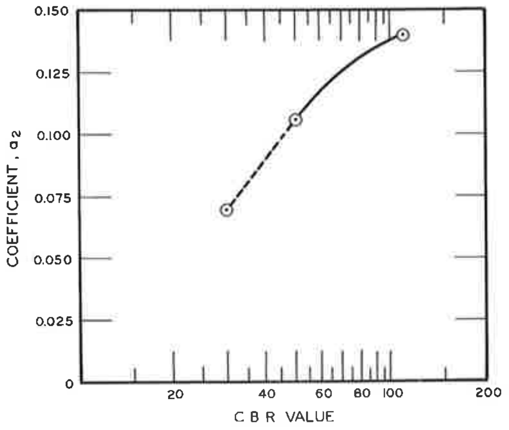

Figure 5. Coefficient for granular base course.

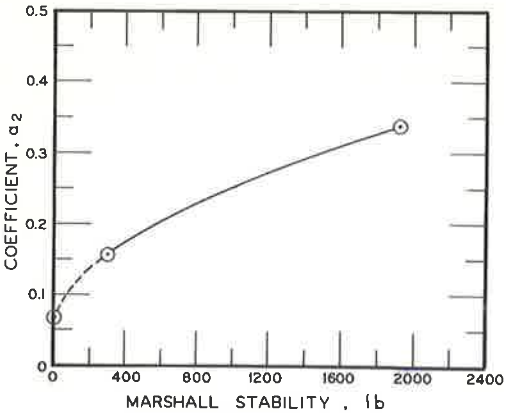

Figure 6. Coefficient for bituminous-treated granular base course.

crushed-stone base material. The value of a_2 for this material is 0.14 and the CBR value determined by Illinois is 110. The lower point represents the Road Test sand-gravel subbase material when used as a base course. The coefficient as a base course was estimated from the Road Test data as 0.07, and the CBR value of the material is 30.

The coefficient for bituminous-stabilized granular base course materials was considered to vary with Marshall stability (Fig. 6). The upper point on the curve represents the bituminous-treated base on the Road Test. The sand-gravel subbase material was mixed with 5.2 percent of 85-100 penetration grade paving asphalt. A value of 0.34 was estimated from the Road Test data for this material, and the Marshall stability tests indicated a value of 1,900. The intermediate point represents Grade 11 gravel stabilized with either emulsified or liquid asphalts. The coefficient was taken as equal to 0.16 and the equivalent Marshall stability as 300. The lower point represents the Road Test sand-gravel material without treatment (a_2 = 0.07).

It was assumed that the coefficient for portland cement-stabilized granular base course material varies with the 7-day compressive strength of the material, determined from field and related laboratory tests (Fig. 7). The curve was developed from three

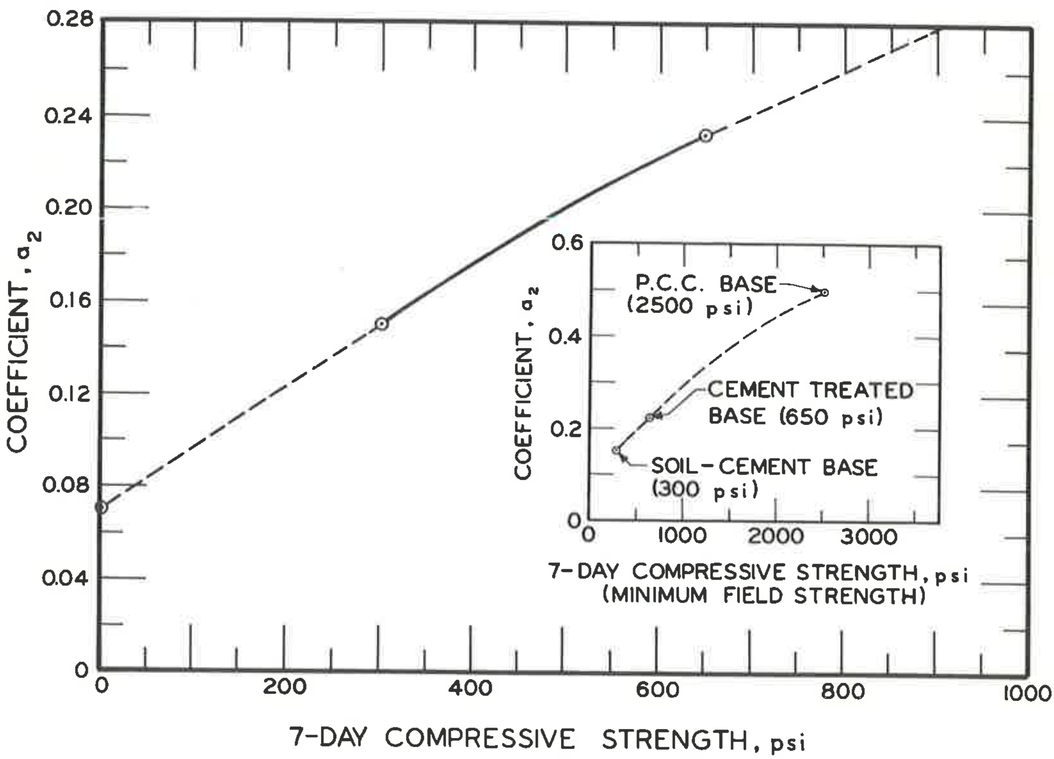

Figure 7. Coefficient for cement-treated granular base course.

points. The upper represents the Road Test cement-treated base material (sand-gravel subbase material) with 4 percent cement. The value of a_2 was estimated from the Road Test data to be 0.23. The lower point represents the same sand-gravel material without cement stabilization ($a_2 = 0.07$). The intermediate point represents the minimum compressive strength for adequate durability of soil cement base; it was assigned a value of $a_2 = 0.15$.

It was assumed that the coefficient for lime-fly ash-stabilized granular base course material (pozzolanic base) varies with the 21-day compressive strength of the material, as determined from field and related laboratory tests (Fig. 8). Since performance data on pozzolanic base course were not available for determining the relationship between the coefficient a_2 and compressive strength, it was necessary to compare the ratios between compressive strengths at various ages and 7-day strengths for pozzolanic bases to those for cement stabilized bases. Compressive strength tests on field-cured specimens representing fall construction indicate that 40 to 50 percent of the ultimate strength can be expected to be obtained in 7 days for cement-stabilized granular material and in 21 days for pozzolanic base material. The 7-day strength of a pozzolanic base can be expected to represent only 15 to 20 percent of the ultimate strength. Thus, the relationship between a_2 and 7-day compressive strength of cement-stabilized granular base course has been assumed to be the same as the relationship between a_2 and 21-day compressive strength of lime-fly ash-stabilized granular base course.

The coefficient for new portland cement concrete base course was estimated to be 0.5 at a 7-day compressive strength of 2,500 psi. An indicated relationship between the coefficient for portland cement concrete base course and the coefficients for cement-stabilized base course is shown in Figure 7.

Subbase. — The coefficient for subbase material (a_3) was correlated with laboratory CBR values obtained by the procedures used by the Division (Fig. 9). The point at a_3 = 0.11 and CBR = 30 represents the sand-gravel subbase material used on the Road Test. The lower point was established at a_3 = 0.05 and CBR = 5. This is considered to represent a sandy-clay material. The upper point was established at a_3 = 0.14 and CBR = 110 for 100 percent crushed material with rough textured surfaces.

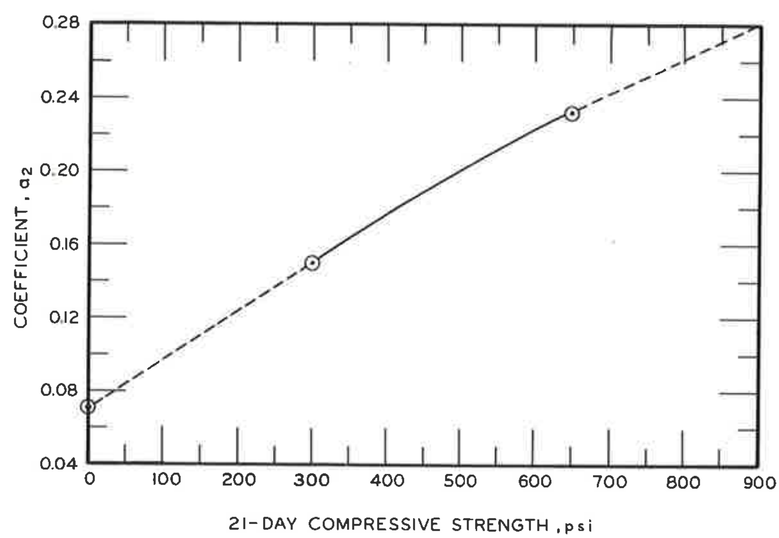

Figure 8. Coefficient for lime-fly ash-treated granular base course.

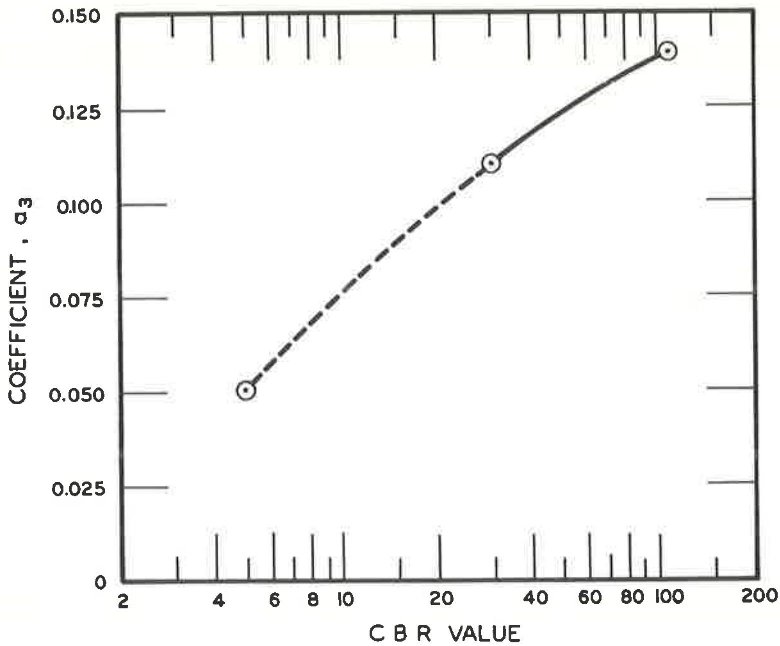

Figure 9. Coefficient for granular subbase course.

SPECIAL CONSIDERATIONS

The structural design procedure presented in this paper establishes a means of determining the structural number (D_t) and, subsequently, the thicknesses of subbase, base, and surface courses required for a bituminous pavement to give satisfactory performance while carrying a given volume of mixed traffic for a definite period of time. The factors affecting pavement design that are considered in this procedure include the volume and composition of mixed traffic, the support strength of the roadbed soils, the strength characteristics of the materials used in the pavement structure, and the length of time the pavement is being designed to serve traffic (design period).

This procedure has been developed specifically for application in the structural design of bituminous pavements in Illinois. Applying the procedure in the design of pavements in regions where climatic and environmental conditions vary widely from those in Illinois must be done with extreme caution. It is expected that modifications to reflect variations in climatic and environmental conditions will be necessary to permit direct application of this procedure in pavement design for other regions. Further, the differences in axle loadings as they exist on Illinois highways and on highways in other regions should be considered.

The design procedure has been developed primarily from a study of the performance of existing pavements in Illinois. Thus, the effects of the various factors on design are considered to represent statewide average conditions. Situations can be expected to arise in which special consideration of one or more of the factors will be necessary so that the determined design will be both practical and adequate for the traffic the pavement is intended to carry.

Traffic and Loads

The equivalency factors and equations used for converting structural design traffic into a traffic factor representing the total number of equivalent 18-kip single-axle load applications are based on statewide average distributions of vehicle types and axle loadings, and are directly applicable to most roads and streets. However, cases will arise in which these factors and equations cannot be used, and a special analysis will be necessary. One such case would be a highway adjacent to an industrial site where heavy commercial vehicles entering and leaving the site generally travel empty in one direction and fully loaded in the other. The information needed for a special analysis in such a case includes loadometer and traffic classification-count data in sufficient detail to permit a determination of the distribution of commercial vehicle types and of single- and tandem-axle loadings within each type.

Roadbed Soils

The performance of a bituminous pavement is directly related to the physical properties and supporting power of the materials used in the pavement structure and of the roadbed soils. The effect of less satisfactory soils can be reduced by increasing the thickness of the pavement structure, but it may be necessary to take other steps to assure adequate pavement performance. The problems that can be encountered because of the roadbed soils being subject to permanent deformation, excessive volume changes, excessive deflection and rebound, frost susceptibility, and nonuniform support from wide variations in soil type or state should be recognized at the design stage, and corrective measures should be included in the design. These corrective measures are in addition to the design thicknesses determined by the procedure.

Pavement Structure

A bituminous pavement consists of a two-layer or a three-layer structure, including a surface course and base course, or a surface course, base course and subbase course. Each layer must have sufficient strength and thickness to sustain the load imposed on it and to distribute it over an area sufficient to prevent the structural strength of the next succeeding layer from being exceeded. Thus, the composition of the pavement structure must be such that the strength characteristics of the surface course material are higher than those of the base course or subbase, and that the strength characteristics of the base course material are higher than those of the subbase. This must be borne in mind in selecting the materials to be used in the pavement structure. In other words, if two granular materials having different strength characteristics are selected for use, the higher strength material must be used as the base course and the lower strength material as the subbase. If only one material is to be used for both the subbase and the base course, then the pavement structure must be considered as a two-layer system consisting only of a surface course and a base course.

It is necessary to consider construction and maintenance problems in the early stages of design to avoid an impractical design. Such considerations usually result in the establishment of minimum thickness and material requirements for each layer of the pavement structure.

The minimum thickness and material requirements given in Table 7 serve only as guides in determining the structural design. The thicknesses and strength characteristics of the materials to be used as the surface course, base course, and subbase for any required structural number should not be less than those given in the table; however, the actual thicknesses are to be determined from the pavement thickness equation and from minimum strengths consistently developed by materials normally used in the locality involved.

Design Period

The analysis period for the design (design period) has been left to the option of the designer. It is recommended, however, that the design period generally should not be less than 20 years for Class I and Class II roads and streets, and should not exceed 20 years for Class III and Class IV. The recommendation that the design period for Class III and Class IV not exceed 20 years is based on the fact that the required structural design generally will permit the use of a granular base under a bituminous mat, and the nationwide average life of this type of pavement is about 17 years. Data from the AASHO Road Test has demonstrated that the level of performance of a bituminous pavement is increased considerably when the granular base material is

TABLE 7

MINIMUM REQUIREMENTS

Structural Number (D_t) From	To	Minimum Thickness Requirements (in.) Surface Course	Base Course	Subbase Course	Minimum Material Requirements[a] Surface Course (Subclass)	Base Course (Type)	Subbase Course (Type and Grade)
1.00	1.99	2[b]	8[c]	4 (if used)	B-1, B-2, B-3, or B-4	Gravel or cr. stone, Type B	Gravel, 11
2.00	2.49	2	8[c]	4 (if used)	B-5	Gravel or cr. stone, Type B	Gravel, 11
2.50	2.99	3	8[c]	4 (if used)	B-5	Gravel or cr. stone, Type A	Gravel, 11
3.00	3.99	3[d]	8	4 (if used)	I-11	Stab. Gran. Mtl's.: Min. CS = 450 psi[e] or Min. MS - 400[f]	Gravel, 11
4.00	4.99	4[d]	8	4 (if used)	I-11	Stab. Gran. Mtl's.: Min. CS = 650 psi[e] or Min. MS = 1,000[f]	Gravel, 11
5.00	5.99	4[d]	10	4	I-11	Stab. Gran. Mtl's.: Min. CS = 650 psi[e] or Min. MS = 1,000[f]	Gravel, 7
		3[d]	8	4	I-11	P/C concrete	Gravel, 7
6.00 or greater		4[d]	12	6	I-11	Stab. Gran. Mtl's.: Min. CS = 1,000 psi[e] Min. MS = 1,500[f]	Gravel, 7
		4[d]	9	4	I-11	P/C concrete	Gravel, 7

[a] Other approved materials having equal or greater strengths may be substituted.
[b] If the surface course is subclass B-2, the minimum surface course thickness shall be $2\frac{1}{2}$ in.
[c] The minimum base thickness may be reduced to 6 in. if a granular subbase is used, or if the base course is a stabilized select soil or stabilized granular material.
[d] For excellent riding surface, three courses should be laid over the base course; Minimum course thickness = $1\frac{1}{4}$ in.
[e] CS = 7-day compressive strength of cement-aggregate mixture or 21-day compressive strength of lime-fly ash-aggregate mixture. (Value that can be reasonably expected under field conditions. To be considered as minimum strength to be obtained before any traffic, including contractor's, is permitted on the surface course.)
[f] MS = Marshall stability or equivalent.

stabilized with bituminous materials or portland cement. Thus, it is recommended that the longer design period be used for Class I and Class II since the structural design generally will require a stabilized granular base course.

The design period may or may not be the actual service life of the pavement. The actual service life may be longer or shorter than the design period, depending on the differences between conditions under which the pavement actually serves and the conditions assumed in design. Highly significant are the differences between the structural design traffic and the actual traffic carried by the pavement, and between the structural design terminal serviceability level and the actual serviceability level at which the pavement is retired from service.

Stage Construction

Planned stage construction is the construction of roads and streets in two stages according to design and a predetermined time schedule. The first stage includes the complete construction of the required thickness of subbase and/or base course along with the application of a bituminous surface treatment to serve as a temporary surface course. The second stage includes the construction of the required type and thickness of bituminous mat.

It is recommended that planned stage construction be used only on roads and streets requiring structural numbers not in excess of 2.49. The required structural design should be determined for the full selected design period. The pavement structure may then be scheduled for construction in two stages. It is important, however, that the second stage of construction be scheduled and performed before any major distress develops in the base course. Otherwise, satisfactory performance of the completed pavement cannot be expected. In the event that major distress should occur in a base course before the second stage construction is accomplished, a complete re-evaluation of the pavement design will be necessary.

APPLICATION OF DESIGN PROCEDURE

This design procedure establishes a means of determining the thickness of a bituminous pavement required to carry a specific volume and composition of mixed traffic for a designated period of time and retain a serviceability level at or above a designated minimum value at the end of this period of time.

The application of this method involves three principal determinations:

1. Conditions under which the pavement is to serve; namely, the length of time it is to be designed to serve, the traffic it is to carry, and the support that will be provided by the roadbed soils;
2. Structural number, D_t, that will be required; and
3. Types and thicknesses of the individual layers of material that are to constitute the pavement structure.

The design period is left to the discretion of the designer. It is again recommended, however, that the design period should be at least 20 years for Class I and Class II and not more than 20 years for Class III and IV.

The structural design traffic is an estimate of the average daily traffic (number of passenger cars, single units, and multiple units) for the year representing one-half of the design period; e.g., when the design period is 20 years and the anticipated construction date is 1965, the structural design traffic will be an estimate of the average daily traffic projected to 1975.

The traffic equations (Table 4) and the percent of vehicles in the design lane (Table 3) are used to convert structural design traffic into a traffic factor representing the total number of equivalent 18-kip single-axle load applications to be carried by the pavement during the entire design period. Any special case, such as previously described, will require a special analysis.

The soil support CBR value should be determined from the soil survey and from laboratory CBR tests on the soil samples. In the absence of laboratory CBR tests, or other approved test procedures, the CBR value may be estimated as shown in Table 5. It is necessary that the soil support CBR value be taken as a minimum value. In addition, corrective measures must be provided for any and all isolated areas where the support of the roadbed soils falls below the minimum so that the minimum requirement will be met throughout.

The structural number, D_t, required for the conditions under which the pavement is being designed to serve is determined from Figure 2 for Class I roads and streets, and from Figure 3 for all other classifications. A line passing through the determined point on the traffic factor scale and on the soil support CBR scale will intersect the structural number scale at the required D_t value.

The thicknesses of the various layers of the pavement are then determined from the structural number equations and from the data in Table 6. By setting the thicknesses of two of the layers, the thickness of the third layer can be determined. Trial designs with variations in thicknesses and with various types of pavement materials will enable the designer to arrive at the most practical and economical design. To assist in this, the minimum thickness and material requirement given in Table 7 should be followed.

Minimum material requirements are suggested to insure a better level of performance throughout the pavement life. It should be remembered that increasing the quality of material will reduce the thickness that is required and will tend to increase the level of performance of the pavement provided the established minimum thicknesses are not violated.

Application of the design procedure is best demonstrated in the following example problem.

The Problem:

Determine the structural thickness needed for the following conditions:

1. Class II roads or street;
2. Two-lane pavement;
3. Design period—20 years;

4. Structural design traffic:
   4,000 total average daily traffic—
   (a) 3,000 passenger cars,
   (b) 250 single units,
   (c) 750 multiple units;
5. Soil support CBR value = 3.0; and
6. Both gravel and crushed stone are readily available for use in the pavement structure.

The Solution:

1. The first step is to determine the traffic factor. Referring to Table 4, the TF equation for Class II roads and streets is

$$TF = DP \frac{(0.146\ PC \times P) + (39.785\ SU \times S) + (337.260\ MU \times M)}{1,000,000}$$

Values of P, S, and M, obtained from Table 3, for a two-lane facility are 0.50. Substituting in the equation, the information given in the problem:

$$TF = 20 \frac{(0.146 \times 3000 \times 0.50) + (39.785 \times 250 \times 0.50) + (337.260 \times 750 \times 0.50)}{1,000,000}$$

$$TF = 2.63.$$

2. It is now possible to determine the structural number, D_t, from Figure 3. Enter the chart at 2.63 on the traffic factor scale and project a line through 3.0 on the soil support CBR scale to intersect the structural number scale. The structural number, D_t, at this intersection is 4.75.

3. The final step is to determine the types and thicknesses of materials for the surface course, base course, and subbase that are required for structural number of 4.75. First, it is necessary to refer to the minimum thickness and material requirements which serve as guides in selecting the types of materials and determining the actual thicknesses to be used. Referring to Table 7, the minimum requirements for D_t = 4.0 to 4.99 are as follows.

Surface Course—not less than 4 in. thick and not less than I-11 bituminous concrete.
Base Course—not less than 8 in. of stabilized granular material having a minimum compressive strength of 650 psi or a minimum Marshall stability of 900.
Subbase—not less than 4 in. thick and not less than Grade 11 gravel, if used.

Using these minimum requirements as guides, it is now possible to select the materials to be used in the surface, base, and subbase courses, and to calculate the corresponding thicknesses using the equation:

$$D_t = a_1D_1 + a_2D_2 + a_3D_3$$

The values of the coefficients for the materials selected for trail designs in this sample problem are obtained from Table 6 as follows:

Surface Course	a_1
I-11 bituminous concrete (minimum Marshall stability = 1,700)	0.40
Base Course	a_2
Bituminous-stabilized granular material (900 minimum Marshall stability)	0.24
Portland cement-stabilized granular material (7-day minimum compressive strength = 650 psi)	0.23
Lime-fly ash-stabilized granular material (21-day minimum compressive strength = 650 psi)	0.23
Subbase	a_3
Grade 11 gravel (30 minimum CBR)	0.11

The thicknesses of the various layers are determined by assuming thicknesses for two of the layers within the minimum requirements and calculating the required thickness of the third layer.

(a) Assume for the example problem that an 8-in. bituminous-stabilized granular base ($a_2 = 0.24$) is to be used, that the surface will be 4 in. thick, and that the subbase will be Grade 11 gravel. The required thickness of subbase is determined as follows:

$$D_t = a_1D_1 + a_2D_2 + a_3D_3$$

$$4.75 = (0.40 \times 4) + (0.24 \times 8) + (0.11\, D_3)$$

$$D_3 = \frac{1.23}{0.11} = 11.2 \text{ in. (Use 11 inches.)}$$

(b) A second solution to this problem is obtained by assuming that an 8-in. portland cement stabilized granular base ($a_2 = 0.23$) will be used, that the surface course will be 4 in. thick, and that the subbase will be Grade 11 gravel.

$$4.75 = (0.40 \times 4) + (0.23 \times 8) + (0.11\, D_3)$$

$$D_3 = \frac{1.31}{0.11} = 11.91 \text{ in. (Use 12 inches.)}$$

(c) A third solution is obtained by assuming a 10-in. bituminous stabilized base ($a_2 = 0.24$) and the subbase will be 4 in. of Grade 11 gravel. The thickness of surface course is determined as follows:

$$4.75 = (0.40\, D_1) + (0.24 \times 10) + (0.11 \times 4)$$

$$D_1 = \frac{1.91}{0.40} = 4.78 \text{ in. (Use 4.75 inches.)}$$

Thus, for this sample problem three combinations of thicknesses of specific types of materials (three trail designs) have been determined, and all equally satisfy the requirement that the structural number, D_t, equals 4.75.

Other trail designs could be determined by assuming different types of materials and different thicknesses of two of the layers and computing a new thickness for the third, providing the types and thicknesses meet the minimum requirements given in Table 7. The selection of the combination of thicknesses and materials to be used for the pavement structure from those determined by the trail designs is basically a problem of economics. The one selected generally should be the one that can be built and maintained for the least amount of money. This can be determined by applying current unit prices to the various combinations of materials and thicknesses and to maintenance operations.