AASHO Road Test Equations Applied to the Design of Portland Cement Concrete Pavements in Illinois

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• THE ILLINOIS Division of Highways has been directing a considerable portion of its research efforts toward developing practical applications of the findings of the AASHO Road Test Project to design and evaluation of highway pavements in Illinois. The work has been done in cooperation with the U. S. Bureau of Public Roads.

This paper presents and describes the development of a procedure for applying the results of the Road Test rigid pavement research to the structural design of portland cement concrete pavements in Illinois.

The developed procedure provides a means for determining the types and thicknesses of concrete slab so that, on the average, the pavement will be capable of carrying a specific volume and composition of mixed truck and passenger car traffic for a designated period of time and, at the same time, retain a level of serviceability at or above a designated minimum.

The AASHO Road Test rigid pavement performance equation (2) 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 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 the development of 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 to which the public considers itself to be served. This has come to be known as the Pavement Serviceability-Performance Concept (1).

Under this concept, the term "present serviceability" was chosen to represent how well a highway 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.

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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 rigid pavement sections on the AASHO Road Test:

$$p = 5.41 - 1.80 \log (1 + \overline{SV}) - 0.09 \sqrt{C + P}$$

in which

p = present serviceability index;

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

C = lineal feet of cracking per 1,000 sq ft of pavement area; and

P = square feet of bituminous patching per 1,000 sq ft of pavement area.

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

$$p = 12.0 - 4.27 \log \overline{RI} - 0.09 \sqrt{C + P}$$

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 number of axle-load applications.

Performance Equations from AASHO Road Test

Present Serviceability Index values were determined every two weeks for each Road Test Section. Serviceability trends were developed for the sections by plotting the Present Serviceability Index values against the corresponding numbers of 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 axle-load applications.

The performance equation developed for the rigid pavement sections (2) is

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

where

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 4.5 on test road),

p_t = serviceability at the 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 at time t, 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.

Expressions for β and ρ are as follows:

$$\log (\beta - 1.0) = \log 3.63 + 5.20 \log (L_1 + L_2) - 8.46 \log (D_2 + 1) - 3.52 \log L_2$$

and

$$\log \rho = 5.85 + 7.35 \log (D_2 + 1) - 4.62 \log (L_1 + L_2) + 3.28 \log L_2$$

in which

L_1 = log on one single-load axle or in one tandem-axle set, kips;
L_2 = axle code = 1 for single axle = 2 for tandem axle; and
D_2 = thickness of concrete slab, 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 lb 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 portland cement concrete 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 (7).

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 upon 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 area where the pavement is to be built; and
5. Relative ability of the pavement slabs to support loads.

All of these factors have been taken into consideration in the development of this design procedure. The influences of climate on freeze-thaw cycles, frost penetration, subgrade moisture content and on other factors that affect pavement design have an appreciable effect on pavement life. These influences undoubtedly vary from one part of the state to another, and particularly between the extreme northern and extreme southern portions. 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 design, and climatic effects can now be considered only on a statewide basis. A comparison of the 14-day flexural strength of the concrete used in the AASHO Road Test pavements to that of the concrete used in Illinois highway construction has indicated that, for design purposes, the two concretes are of the same strength. The remaining factors are included in the design charts and equations.

The charts and equations included in this design procedure were developed from the AASHO Road Test rigid pavement performance equation with necessary modifications to reflect in the structural design the effects on pavement performance of:

1. Mixed-traffic axle loadings when compared to the controlled traffic axle loadings on the Road Test;
2. Pavement subjected to traffic over a long period of time when compared to the two years of traffic on the Road Test; and
3. Variations in the support strengths of the roadbed soils.

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, "equivalency factors" were used 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-axle 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 units and multiple units operating on highways ranging from high-volume major highways with heavy commercial hauling to low-volume secondary roads with farm-to-market type hauling. To accomplish this the highway system was divided into three general classifications:

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

The above classifications of roads and streets were selected so that, in general, Class I roads and streets represent the Interstate and expressway systems, Class II the remainder of the primary system, and Class III the secondary system of highways.

The results of the AASHO Road Test provided a means of developing equivalency factors for converting any given single- or tandem-axle loading into an equivalent number of 18-kip (18,000-lb) single-axle load applications relative to its effect on pavement performance. The equivalency factor may be expressed as follows:

$$\begin{pmatrix}\text{18-kip single-axle} \\ \text{equivalency} \\ \text{factor}\end{pmatrix} = \frac{\begin{pmatrix}\text{No. of 18-kip single-axle load applications to a given} \\ \text{present serviceability index}\end{pmatrix}}{\begin{pmatrix}\text{No. of x-kip applications to the same given} \\ \text{present serviceability index}\end{pmatrix}}$$

This factor was developed by the following mathematical analysis:

$$\log W_t = \log \rho + \frac{G_t}{\beta} \tag{1}$$

or

$$\log W_t = 5.85 + 7.35 \log (D_2 + 1) - 4.62 \log (L_1 + L_2) + 3.28 \log L_2 + \frac{G_t}{\beta} \tag{2}$$

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

$$\log W_{t_{18}} = 5.85 + 7.35 \log (D_2 + 1) - 4.62 \log (18 + 1) + \frac{G_t}{\beta_{18}} \tag{3}$$

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

$$\log W_{t_x} = 5.85 + 7.35 \log (D_2 + 1) - 4.62 \log (x + 1) + \frac{G_t}{\beta_x} \tag{4}$$

Subtracting Eq. 4 from Eq. 3, the equivalency factor for single-axle loadings becomes

$$\log = \frac{W_{t_{18}}}{W_{t_x}} = 4.62 \log (x + 1) - 4.62 \log (18 + 1) + \frac{G_t}{\beta_{18}} - \frac{G_t}{\beta_x} \tag{5}$$

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

$$\log W_{t_x} = 5.85 + 7.35 \log (D_2 + 1) - 4.62 \log (x + 2) + 3.28 \log (2) + \frac{G_t}{\beta_x} \tag{6}$$

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

$$\log \frac{W_{t_{18}}}{W_{t_x}} = 4.62 \log (x + 2) - 4.62 \log (18 + 1) - 3.28 \log (2) + \frac{G_t}{\beta_{18}} - \frac{G_t}{\beta_x} \tag{7}$$

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

The ratio between $W_{t_{18}}$ and W_{t_x} in Eqs. 5 and 7 expresses the relationship between 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 vary from 6 to 11 in., and for present serviceability, levels of 2.0 and 2.5 have been 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 Class I and Class II roads and streets. No loadometer data were available for Class III. Traffic classification-count data were available for Class I, Class II and Class III.

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 18-kip equivalent single-axle load applications per passenger

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	0.0002	4	0.0005	-
4	0.002	0.002	8	0.005	-
6	0.010	0.010	12	0.030	0.030
8	0.030	0.030	16	0.082	0.085
10	0.082	0.085	20	0.207	0.212
12	0.178	0.183	24	0.443	0.452
14	0.343	0.352	28	0.850	0.850
16	0.603	0.610	32	1.49	1.473
18	1.000	1.000	36	2.467	2.388
20	1.572	1.552	40	3.858	3.673
22	2.363	2.302	44	5.797	5.430
24	3.437	3.300	48	8.412	7.760

TABLE 2

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

Road and Street Classification	18-Kip Equivalent Single-Axle Load Applications Per Vehicle		
	Passenger Cars	Single Units	Multiple Units
Class I	0.0004	0.123	1.155
Class II	0.0004	0.123	1.134
Class III	0.0004	0.123	1.134

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 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 roads and streets, it was assumed that the distribution of axle loadings for each individual classification of vehicle within the single unit and 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 (Table 1) 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 single unit, and per multiple unit for Class III. The resultant values did not vary appreciably from those determined for Class II, and they did not affect a material difference in pavement design. Therefore, to further simplify the design procedure, the values used for Class III are the same as those determined for Class II.

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

Road and Street Classification	Traffic Factor Equation
Class I	$TF = DP\left[\frac{(0.146\ PC \times P) + (44.995\ SU \times S) + (421.575\ MU \times M)}{1,000,000}\right]$
Class II and III	$TF = DP\left[\frac{(0.146\ PC \times P) + (44.995\ SU \times S) + (413.910\ MU \times M)}{1,000,000}\right]$

The 18-kip equivalent single-axle load applications per vehicle classification factors determined for Class I, Class II, and Class III are given in Table 2. The results of this analysis yielded two important facts regarding the reduction of mixed-traffic axle loadings to 18-kip equivalent single-axle load applications:

1. The effect of passenger cars is small in proportion to the effect of single and multiple units; and
2. The effect of multiple units is nine to ten 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 listed in Table 2 were used in developing equations to convert mixed-traffic axle loadings into a traffic factor for use in structural design. In developing the equations, 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 vehicles per day in the design lane may be estimated by multiplying the structural design traffic by the percentage distributions presented in Table 3.

Traffic factor equations were developed for all three classifications of roads and streets. The equations are given in Table 4. They were developed from the following model:

$$TF = DP \left[\frac{(k_1 \times PC \times P) + (k_2 \times SU \times S) + (k_3 \times MU \times M)}{1,000,000} \right]$$

in which

TF = traffic factor;
DP = design period, years;
k_1 = constant for passenger cars = value in Table 2 × 365;
k_2 = constant for single units = value in Table 2 × 365;
k_3 = constant for multiple units = value in Table 2 × 365;
PC = passage car ADT (two directions);
SU = single-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 Road Test 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 study were selected on the basis of subgrade soil, pavement materials, and climatic conditions being similar to those that existed on the Road Test.

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 predicted, 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 agreed closely with the performance of pavements on the Road Test of lesser thickness, as shown in Figure 1. 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 slab thickness in the equations for ρ and β. This factor has been termed a time-traffic exposure factor, T.

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

Performance data were obtained on 48 pavement sections representing PCC slab thicknesses of 10-in. uniform, and 9-6-9 and 9-7-9 in. thickened edge designs.

Some of the pavements included in this study were constructed on a granular subbase; others were built directly on the earth subgrade. The AASHO Road Test demonstrated that thickness of granular subbase does not affect performance of PCC pavement; however, the performance of Road Test pavements with subbase was increased by one-third over those of the same slab thickness but without subbase. Therefore, the performance of each pavement section included in this study that did not have a granular subbase was adjusted to compensate for the lack of subbase by increasing the total number of equivalent 18-kip single-axle load applications by one-third.

The time-traffic exposure factor was determined for each of the 48 pavement sections by dividing the thickness, D, of the Illinois slab by the thickness, D_2, of the Road Test slab that is capable of carrying the same number of equivalent 18-kip single-axle load applications to the same level of serviceability. This relationship is expressed as follows:

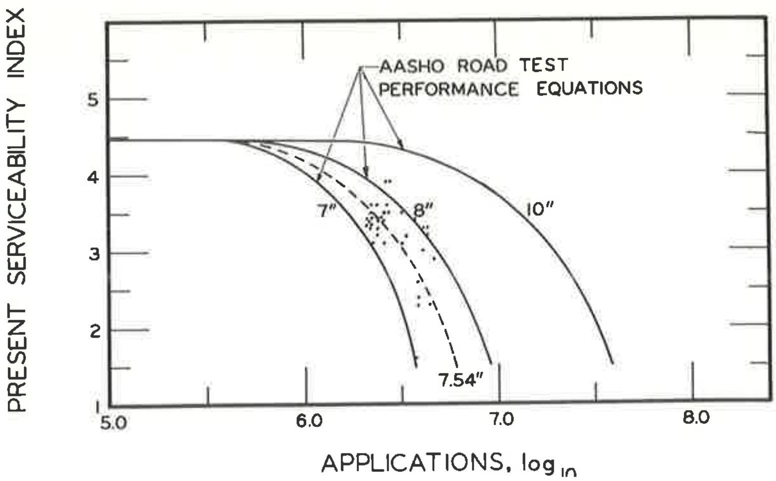

Figure 1. Performance of Illinois 10-in. P.C.C. pavement vs performance predicted by Road Test performance equation.

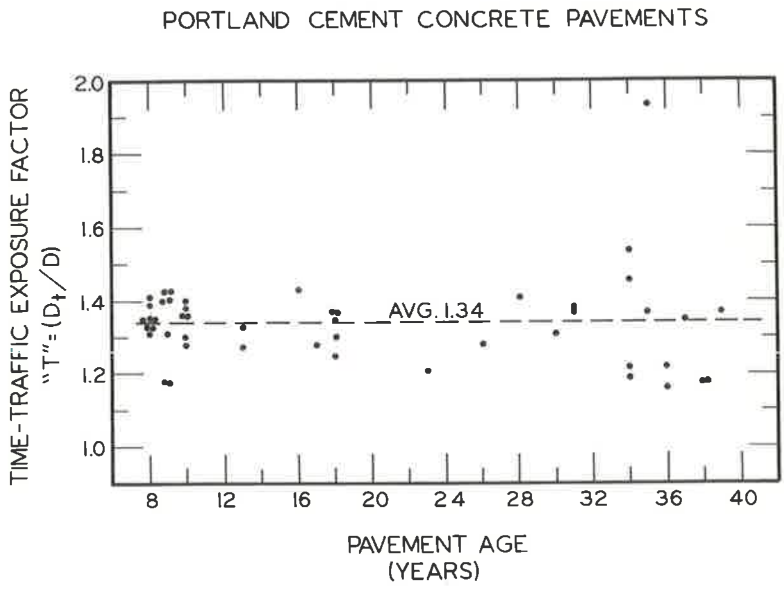

Figure 2. Time-traffic exposure factor vs pavement age.

$$TF = \frac{D}{D_2}$$

in which

T = time-traffic exposure factor;
D = Illinois slab thickness in inches; and
D_2 = Road Test slab thickness in inches.

The value of D for the thickened edge designs was taken as the effective uniform thickness by a procedure which makes use of Westergaard's equation for corner loading (8) as follows:

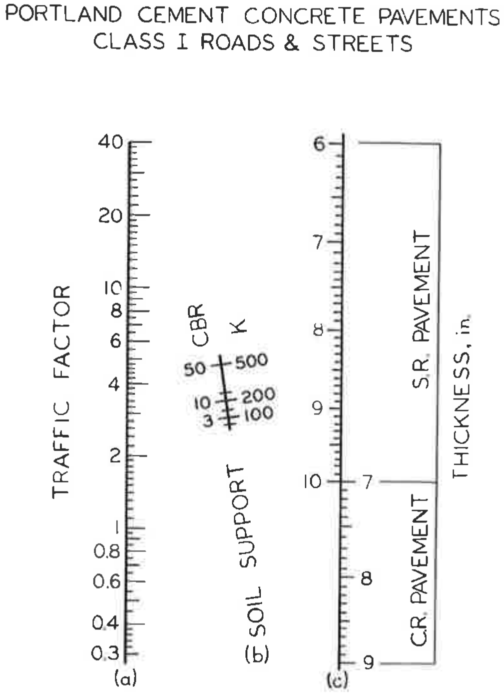

Figure 3. Design nomograph.

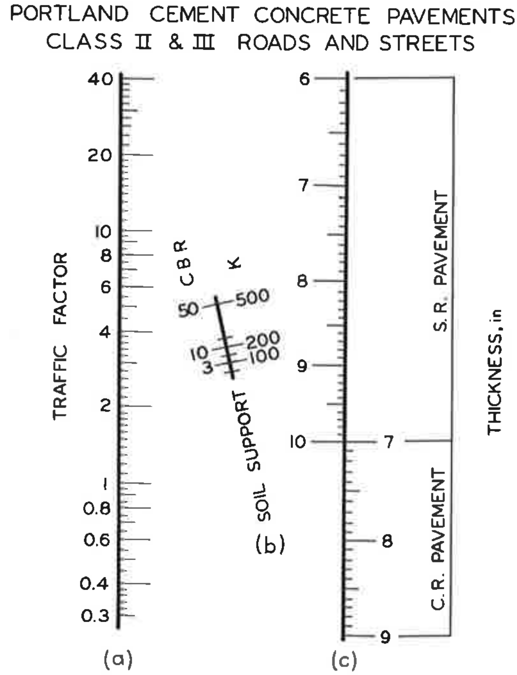

Figure 4. Design nomograph.

Slab Thickness (in.)	Effective Thickness (in.)
9-6-9	7.06
9-7-9	7.71

The results of the analyses for the 48 pavement sections are shown in Figure 2, where the time-traffic exposure factor, T, has been plotted against pavement age in years. The mean value of T was determined to be 1.34; a value of 1.30 has been used in developing the design nomographs in Figures 3 and 4.

Roadbed Soils

Soil of one type and strength was used in the AASHO Road Test. The upper 3-ft portion of embankment under all the pavement test sections was an A-6 (9-13) soil. This fact made it necessary to develop a means of modifying the results obtained from the AASHO Road Test performance equation to permit the development of slab thickness designs for other soil types.

The soil support scales in Figures 3 and 4 represent the modification that has been made to include the support strength of roadbed soils as a variable. This modification was made on a theoretical basis, following the precedures suggested by the AASHO Committee on Design (7). It involved a comparison of stresses computed from actual strains measured on the Road Test slabs to the stresses calculated by Spangler's equation for corner loading. The resultant modification of the Road Test equation provides a theoretical measure of the effects on performance of changes in the modulus of subgrade reaction, the flexural strength of the concrete slab, and the modulus of elasticity of the concrete. The revised performance equation for $p = 2.0$ is:

$$\log W_p = 7.35 \log (D_2 + 1) - 0.06 + \frac{G_t}{\beta} + 3.58 \log \left[\frac{S'_c \left(D_2^{0.75} - \dfrac{18.416\, k^{0.25}}{E_c^{0.25}} \right)}{S_c \left(D_2^{0.75} - \dfrac{18.416\, k'^{0.25}}{E'^{0.25}_c} \right)} \right]$$

and for p = 2.5:

$$\log W_p = 7.35 \log (D_2 + 1) - 0.06 + \frac{G_t}{\beta} + 3.42 \log \left[\frac{S'_c \left(D_2^{0.75} - \dfrac{18.416\, k^{0.25}}{E_c^{0.25}} \right)}{S_c \left(D_2^{0.75} - \dfrac{18.416\, k'^{0.25}}{E'^{0.25}_c} \right)} \right]$$

in which

W_p = number of axle load applications at a given present serviceability index, p;
S_c and S'_c = modulus of rupture for Road Test slabs and slabs to be designed, respectively;
E_c and E'_c = Young's modulus of elasticity of concrete for Road Test slabs and slabs to be designed, respectively;
k and k′ = modulus of subgrade reaction for Road Test soils and other soils, respectively.

The remaining terms are as previously defined.

A comparison of the average flexural strength of the concrete used in Illinois pavements to that of the Road Test concrete demonstrated that the two were of the same strength, hence $S'_c = S_c$. E_c was determined on the Road Test to be 4.2×10^6 (static at 28 days) psi. E'_c was assumed to be equal to E_c. The value of k for the Road Test soils was taken at 100 psi/in. Then, by substitution, the modified equation for p = 2.0 becomes:

$$\log W_p = 7.35 \log (D_2 + 1) - 0.06 + \frac{G_t}{\beta} + 3.58 \log \left[\frac{D_2^{0.75} - 1.286}{D_2^{0.75} - 0.4068\, k'^{0.25}} \right]$$

and for p = 2.5 becomes

$$\log W_p = 7.35 \log (D_2 + 1) - 0.06 + \frac{G_t}{\beta} + 3.42 \log \left[\frac{D_2^{0.75} - 1.286}{D_2^{0.75} - 0.4068\, k'^{0.25}} \right]$$

Points to construct a soil support scale in Figures 3 and 4 were developed by solving the revised equations for W_p, using various values of modulus of subgrade reaction with various slab thicknesses. Since the Illinois Division of Highways does not use k values to determine soil strengths, it was necessary to convert k values to CBR values. This was done by using a correlation previously developed by T. A. Middlebrooks and G. E. Bertram (9). Both the k-scale and the CBR-scale are shown as a part of the soil support scale (Figs. 3 and 4).

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.

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 Proceedings, Vol. 22, 1942, pp. 124-129.)

Design Charts

The design charts (Figs. 3 and 4) include a traffic factor scale, a soil support scale, and a slab thickness scale. They represent graphic presentations of the AASHO Road Test rigid pavement performance equation as modified for Illinois use. Figure 3 is for use in determining the pavement structural design for Class I roads and streets (Interstate highways and expressways); Figure 4 is for use in determining the pavement structural design for Class II and Class III.

The basic difference between the two is the terminal serviceability level assumed in the development. Figure 3 is based on a terminal serviceability level of 2.5; Figure 4, on 2.0. The 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). A study of the terminal serviceability level of highway pavements in Illinois has fairly well substantiated this value as an average value for Illinois. However, pavements of 4-lane divided expressways in Illinois are being retired at serviceability levels above 2.0, and generally averaging 2.4. Further, the AASHO Committee on Design (7) has recommended that the design period for major highways be considered ended 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 roads and streets (expressways and Interstate highways), and 2.0 for all others.

The slab thickness scales (Figs. 3 and 4) include both standard reinforced and continuously-reinforced pavement. The continuously-reinforced pavement is taken to have longitudinal reinforcement amounting to not less than 0.6 percent of the cross-sectional area of the slab. A correlation of the thicknesses of the two types of pavement was developed from a study of the performance of Illinois pavements.

In 1948, the Illinois Division of Highways constructed a 6-mile section of continuously-reinforced concrete pavement on US 40 near Vandalia. To date, this pavement is giving outstandingly good service. The maintenance cost has been low and its surface smoothness, when compared to that of regular pavements of equal age and traffic load, is superior to any that has been measured elsewhere in the State.

The experimental pavement in Vandalia has been under close observation since it was constructed. Pavements of 7- and 8-in. thicknesses were included in this project. Longitudinal steel in amounts of 0.3, 0.5, 0.7, and 1.0 percent of the cross-sectional area of the pavement are included for both thicknesses.

All sections of the 8-in. thickness and the section of 7-in. thickness with 1.0 percent longitudinal reinforcement have not dropped in serviceability sufficiently to distinguish any differences in pavement performance. However, the results of an analysis of the data for the 7-in. sections (Fig. 5) have indicated that a 7-in. slab with 0.6 percent longitudinal steel and no subbase could be expected to perform in a manner equivalent to the 10-in. standard reinforced control section which has a 6-in. granular subbase. Based on this, the ratio of the thickness of continuously-reinforced pavement to the thickness of standard reinforced pavement that can be expected to give the same performance has been taken to be 0.7.

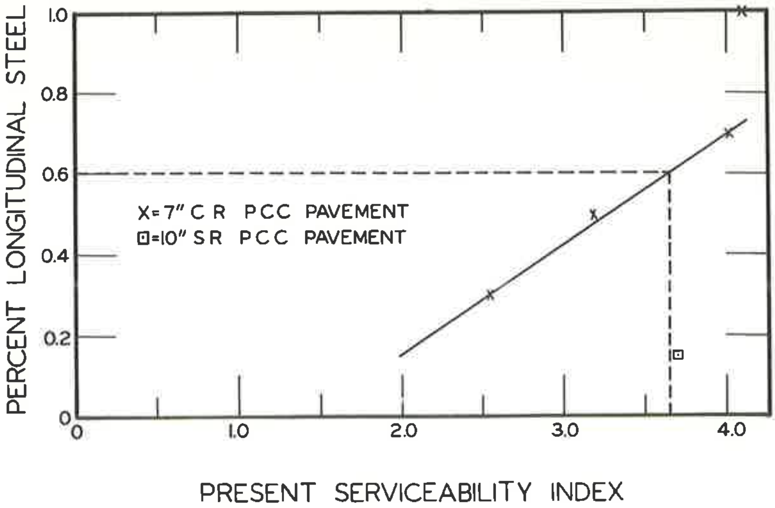

Figure 5. Comparison of 7-in. continuously-reinforced pavement with 10-in. standard reinforced pavement.

SPECIAL CONSIDERATIONS

The structural design procedure presented in this paper establishes a means of determining the type and thickness of slab required for a portland cement concrete pavement to give satisfactory performance while carrying a given volume and composition of mixedtraffic 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, and the length of time the pavement is being designed to serve traffic (design period).

The procedure has been developed specifically for application in the structural design of pavements in Illinois, and the general considerations described in the preceding paper for flexible pavements are also applicable in the case of portland cement concrete pavements. This is also true of the preceding discussion on "Traffic and Loads." (See p. 20.)

Roadbed Soils

The performance of a portland cement concrete pavement is directly related to the physical properties and supporting power of the roadbed soils. Some soils have a detrimental effect on performance that cannot always be overcome by increasing slab thickness. The problems that can arise as a result of the various properties of roadbed soils, such as permanent deformations, excessive deflection and rebound, excessive volume changes, and frost susceptibility need to be recognized in the design stage. Provisions for the solution of these problems should be included in the plans and specifications.

Slab Type and Thickness

The type and thickness of slab is determined directly from Figure 3 or 4. To insure against impractical designs the following basic rules are suggested to serve as guides:

1. Minimum requirements should be established (suggested minimum requirements are given in Table 6);

2. When the analysis indicates a slab thickness less than the minimum requirements, the pavement design should be based on the minimum requirements.

3. When the analysis indicates a slab thickness 0.3 in. or less over an even inch, the design thickness may be taken as the even inch; when the analysis indicates a slab

TABLE 6

MINIMUM STRUCTURAL DESIGN REQUIREMENTS

Road and Street Classification	Portland Cement Concrete Pavement[a]		Subbase[b]	
	Type	Thickness (in.)	Type	Thickness (in.)
Class I[c]	Standard reinforced	10	Granular material	6
			Stabilized granular material	4
	Continuously-reinforced	7	Granular material	6
			Stabilized granular material	4
Class II	Standard reinforced	8	Granular material	6
			Stabilized granular material	4
Class III	Standard reinforced	8	Granular material	6
			Stabilized granular material	4

[a] For streets with curbs and gutters and storm sewer systems that are to be used by only residential traffic, the pavement design shall be not less than a 6-in. nonreinforced slab without subbase.
[b] Subbase not required: (a) for Class II and Class III roads and streets at locations having roadbed soils of a quality equal to the standard granular subbase requirements, (b) for those streets in Class II and Class III with curbs and gutters and storm sewer systems that are to be constructed on an existing roadbed that is not to be appreciably disturbed during the new construction, and (c) for streets with curbs and gutters and storm sewer systems that are to serve only residential traffic.
[c] When the number of 18-kip equivalent single-axle load applications exceeds 130,000 per year, the minimum subbase requirement shall be 4 in. of stabilized granular material.

thickness 0.4 in. or more over an even inch, the next higher full inch should be used (e.g., for 8.3 in. use 8.0 in., and for 8.4 in. use 9.0 in.).

Subbase Type and Thickness

Subbase thickness was excluded as a variable in the AASHO Road Test performance equation. An analysis of the results of the test demonstrated that variations of between 3 and 9 inches in subbase thickness had no significant effect on the performance of the test pavements. The performance of sections of pavement having a subbase, however, was superior to that of sections having the same slab thickness without a subbase.

Subbase thickness has not been included as a design variable in this work. The design procedure was developed on the basis that a subbase is to be used beneath all portland cement concrete slabs. It is considered, however, that a subbase could be omitted on (a) streets with curbs and gutters and storm sewer systems that are to serve only residential traffic, (b) streets with curbs and gutters and storm sewer systems that are to be constructed on existing roadbeds which are not to be appreciably disturbed during the new construction, and (c) Class II or Class III roads and streets at locations that have roadbed soils of a quality equal to the standard granular subbase requirements.

Suggested minimum requirements for type and thickness of subbase are given in Table 6. To provide a subbase that will be less susceptible to scouring, it is recommended that only stabilized granular materials be used when the number of 18-kip equivalent single-axle load applications exceeds 130,000 per year. This number of applications resulted from an analysis of performance data from the AASHO Road Test and condition survey data obtained on US 66. These data suggest that when the magnitude and frequency of heavy axle-load applications increase to the point that the number of 18-kip equivalent single-axle load applications reached approximately 130,000 per year, pumping could become a serious problem with a granular subbase that is not stabilized.

Design Period

The design period is left to the option of the designer. However, it is recommended that the design period should be not less than 20 years except for unusual conditions.

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 upon the conditions under which the pavement actually serves, and conditions given for the design. Highly significant are the differences between the structural design traffic and the actual traffic carried by the pavement, and the difference between the design terminal serviceability level and the actual serviceability level at which the pavement is retired from service.

APPLICATION OF DESIGN PROCEDURE

The design procedure described herein enables the designer to determine the type of thickness of 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.

The application of the method involves three principal determinations: (a) the conditions under which the pavement is to serve, namely, the length of time it is to serve, the traffic it is to carry, and the support that will be provided by the roadbed soils; (b) the type and thickness, D, of pavement slab that will be required; and (c) the type and thickness of the subbase.

The design period is left to the discretion of the designer. It is recommended, however, that the design period should be not less than 20 years except for unusual conditions.

The structural design traffic is an estimate of the average daily traffic (numbers 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 equations in Table 4 and the data in Table 3 are used to convert structural design traffic into a traffic factor representing total 18-kip equivalent single-axle load applications to be generated by the traffic during the design period. Any special case will require a special analysis.

The soil support CBR value should be determined from a soil survey and laboratory CBR tests on samples of the soil to be used in the roadbed. In the absence of laboratory CBR tests, the minimum CBR values shown in Table 5 may be used.

The type and thickness of a pavement are determined from Figures 3 or 4. When a straight line connecting the point on the traffic-factor scale (scale a), corresponding to the value of the traffic factor obtained by solving the pertinent equation shown in Table 4, with the point on the soil-support scale (scale b), representing the CBR or K value obtained from soil strength tests or Table 5, is extended to the right in Figure 3 or 4 to an intersection with the pavement type and thickness scale (scale c), the point of intersection will reveal the type and thickness of the portland cement concrete pavement which will satisfy the prescribed design requirements.

The necessary steps in utilizing the figures and equations in structural design are illustrated in the following example problem.

<u>The Problem</u>:

Determine the type and thickness of pavement that will satisfy the following prescribed conditions:

1. Class II roads and streets (two-lane facility);
2. Design period = 30 years;
3. Structural design traffic:

   4,000 total average daily traffic representing traffic predicted for the year 1980 (construction scheduled for completion in 1965)—

   (a) 3,000 passenger cars,
   (b) 250 single units,
   (c) 750 multiple units; and
4. Soil Support CBR value = 3.0.

The Solution:

1. The first step is to convert the structural design traffic into a traffic factor. Referring to Table 4, the equation for use with Class II roads and streets is

$$TF = DP \frac{(0.146\ PC \times P) + (44.995\ SU \times S) + (413.910\ MU \times M)}{1,000,000}$$

From Table 3, values of P, S, and M are 0.50 for two-lane facilities. Substituting in the equation the information given in the problem:

$$TF = 30 \frac{(0.146 \times 3,000 \times 0.50) + (44.995 \times 250 \times 0.50) + (413.910 \times 750 \times 0.50)}{1,000,000}$$

$$TF = 4.83$$

2. It is now possible to determine the slab type and thickness from Figure 4. Enter the Chart at 4.83 on the traffic factor scale and project a straight line through CBR = 3.0 on the soil support scale to intersect the thickness scale. The point of intersection shows that a 9.7-in. standard reinforced pavement is required. For design purposes, use a 10-in. slab thickness, or a 7-in. continuously-reinforced pavement may be used.

3. Minimum requirements for type and thickness of subbase are given in Table 6. Since the number of 18-kip equivalent single-axle load applications per year exceeds 130,000, the subbase shall be 4 in. of stabilized granular material: $(4.83 \times 1,000,000) \div 30 = 161,000$ equivalent 18-kip single-axle load applications per year.

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