Structural Design for Heavily Trafficked, Plain-Jointed Concrete Pavement Based on Serviceability Performance

Michael I. Darter, Department of Civil Engineering, University of Illinois, Urbana-Champaign

A new structural design procedure has been developed for heavily trafficked plain-jointed concrete pavement. The objective of this design procedure is to provide a structure that gives zero-maintenance (i.e., very low structural maintenance) performance over the designated design period. The overall design procedure includes both a serviceability-performance analysis and a fatigue-damage analysis; however, only the serviceability-performance analysis procedure is described here. The fatigue-damage analysis provides a direct evaluation of slab cracking, whereas the serviceability-performance analysis provides an overall evaluation of several distress types. Long-term performance data from 25 sections of the original AASHO test road that have been under heavy mixed traffic on I-80 since 1962 were used to derive the new design equations. The new equations fit the long-term performance data much better than did the original AASHO equation that was based on only 2 years' data. Westergaard's edge-stress equation and a climatic factor were incorporated into the design equation to extend it to a large variety of conditions. Performance data from 12 additional heavily trafficked projects located in various climates were used to determine the climatic effects on performance. The new design procedure results in a pavement structure that is expected to provide zero-maintenance performance over the designated design period.

A new structural design procedure for plain-jointed concrete pavement has been developed from long-term performance data of many heavily trafficked pavements. The procedure is based on the serviceability-performance concept and also on a fatigue-damage analysis and was developed specifically to provide zero-maintenance (i.e., very low structural maintenance) performance over the design period (1, 2) [only the serviceability-performance analysis is described here; the fatigue-damage analysis, which gives a direct evaluation of slab cracking, is described elsewhere (2)].

The 2-year American Association of State Highway and Transportation Officials (AASHO) Road Test provided data that were used to derive an empirical equation that has been used extensively in concrete pavement design since 1962 (3, 4). One of the major deficiencies of the AASHO performance equation is that it is based on data for only 2 years, and most of the sections that failed did so because of severe pumping (5).

Since the end of the AASHO Road Test in 1960, the Illinois Department of Transportation has monitored the performance of 25 of the original sections of the test road (which was opened to traffic as part of I-80 in 1962). These data represent a time period of 16 years and the application of almost 20 million 80-kN (18 000-lbf = 18-kip) equivalent single-axle loads (ESALs). The new design equation was developed by using multiple regression techniques to provide a more accurate representation of the relationships among the serviceability index, the accumulated 80-kN ESALs, and the slab thicknesses. Data were also obtained from 12 other plain-jointed concrete projects located in other regions of the United States to provide information about the effects of climate on performance. A summary of the data used in the analysis is given in Tables 1, 2, and 3.

DEVELOPMENT OF NEW DESIGN EQUATION

Original AASHO Equation

The AASHO Road Test provided data from which empirical relationships among portland cement concrete (PCC) slab thickness, load magnitude, type of axle, number of load applications, and serviceability index of the pavement for road-test conditions (i.e., the specific environment and materials) were derived by using multiple regression analyses (5):

$$\log W = \log \rho + G/B \tag{1}$$

where

- W = number of axle-load applications (load magnitude L1 and axle type load L2, to a serviceability index of P2),
- $\log\rho = 5.85 + 7.35\log(H + 1) + 4.62\log(L1 + L2) + 3.28\log(L2)$,
- $B = 1.00 + 3.63(L1 + L2)^{5.20}/(H + 1)^{8.46}L2^{3.52}$,
- $G = \log[(P1 - P2)/(P1 - 1.5)]$,
- H = PCC slab thickness (in),
- L1 = load on a single or a tandem axle (kips),
- L2 = axle code (1 for single axles or 2 for tandem axles),
- P1 = initial serviceability index, and
- P2 = terminal serviceability index.

(These equations and Figure 9 were developed for U.S. customary units only; therefore, values are not given in SI units.)

By using the Spangler corner equation, the empirical model given by Equation 1 was modified and extended to include the material properties. The following basic assumptions were made in this extension:

1. The variation in pavement life (W) for different magnitudes of load at the same level of the ratio of tensile stress: strength of the PCC slab can be calculated by using Equation 1 and is covered in the design procedure by the traffic equivalence factors, and
2. Any change in the ratio of tensile stress: strength that results from changes in the values of the material properties will have the same effect on W as an equivalent change in slab thickness (calculated by Spangler's equation) will have on W as calculated by Equation 1.

The resulting final structural design model is given below:

$$\begin{aligned}\log W_{18} = {} & 7.35\log(H + 1) - 0.06 \\ & + (G/\{1 + [1.624 \times 10^7/(H + 1)^{8.46}]\}) \\ & + (4.22 - 0.32\,P2)\log\{(FF/215.63J) \\ & \times (H^{0.75} - 1.133)/[H^{0.75} - (18.42/Z^{0.25})]\}\end{aligned} \tag{2}$$

where

W_{18} = number of 18-kip single-axle loads required to reduce the serviceability index from P1 to P2,
FF = mean modulus of rupture of PCC slab (28-d cure and 3rd point loading) (lbf/in^2),
J = load transfer coefficient,
Z = E/k,
E = modulus of elasticity of PCC slab (lbf/in^2), and
k = modulus of foundation support (lbf/in^3).

It has also been suggested (4, 6) that other terms should be added to this equation to include the effects of variation of subbase quality and of climate.

The data given in Table 4, which were determined by the Illinois Department of Transportation, for the 25 sections were analyzed to determine the ability of Equation 2 to predict long-term performance. The number of 80-kN ESALs accumulated to each year (i.e., 1962, 1968, 1969, 1971, 1972, and 1974) for each project was calculated on the basis of the mixed traffic data measured at the site and the computed load-equivalency factors. These values were plotted against the number of 80-kN loads predicted by using Equation 2 to cause a loss of serviceability index from 4.5 to the values shown in Table 3. The result is shown in Figure 1.

The standard error of the estimate (standard deviation of residuals) is 0.31 for log W. Figure 1 clearly shows that Equation 2 predicts poorly for thick slabs [H > 241 mm (9.5 in)]. For example, the actual number of 80-kN ESALs for a 318-mm (12.5-in) slab is about 18 million, and the value by using computed Equation 2 is more than 50 million. This standard error is considerably larger than the error based on only the results of the 2-year road test, which was 0.22.

New Design Equation

An approach similar to that used to develop Equation 2 was used to develop a modified equation. Data from the 25 AASHO sections were used in the regression analyses

Table 1. General data for 25 sections of plain-jointed concrete pavements constructed for AASHO Road Test (1958-60) and then subjected to regular mixed traffic on I-80 (1962-74).

					Pavement Section[b]		
Project No.	Loop	Section	Allowable Axle Load (kN)	No. of 80-kN ESALs[a] (millions)	Thickness of PCC Slab (cm)	Thickness of Granular Base (cm)	Diameter of Joint Dowels[c] (cm)
JCP-1	4	672	143	11.16	20.3	7.6	2.54
JCP-2	4	658	143	11.16	20.3	15.2	2.54
JCP-3	4	652	143	11.16	20.3	22.9	2.54
JCP-4	4	676	143	11.32	24.1	7.6	2.85
JCP-5	4	702	143	11.32	24.1	15.2	2.85
JCP-6	4	690	143	11.32	24.1	22.9	2.85
JCP-7	5	552	178	13.75	24.1	0	2.85
JCP-8	5	512	178	13.75	24.1	7.6	2.85
JCP-9	5	542	178	13.75	24.1	7.6	2.85
JCP-10	5	528	178	13.75	24.1	15.2	2.85
JCP-11	6	352	214	17.82	24.1	7.6	2.85
JCP-12	6	368	214	17.82	24.1	15.2	2.85
JCP-13	6	390	214	17.82	24.1	15.2	2.85
JCP-14	6	376	214	17.82	24.1	22.9	2.85
JCP-15	5	530	178	14.09	27.9	7.6	3.13
JCP-16	5	498	178	14.09	27.9	15.2	3.13
JCP-17	5	510	178	14.09	27.9	22.9	3.13
JCP-18	6	364	214	18.94	27.9	7.6	3.13
JCP-19	6	378	214	18.94	27.9	7.6	3.13
JCP-20	6	388	214	18.94	27.9	15.2	3.13
JCP-21	6	398	214	18.94	27.9	15.2	3.13
JCP-22	6	366	214	18.94	27.9	22.9	3.13
JCP-23	6	396	214	19.47	31.7	7.6	3.70
JCP-24	6	350	214	19.47	31.7	15.2	3.70
JCP-25	6	380	214	19.47	31.7	22.9	3.70

Notes: 1 kN = 225 lbf and 1 cm = 2.54 in.
Road opened to traffic 1958; two-directional ADT = 11 100 (mean over 1962-1974) and 18 900 (1974); number of lanes = 4; and percentage of traffic that is trucks = 32.

[a]Total one-directional in heaviest traveled lane. [b]All subgrades clay. [c]Contraction joint spacing = 4.6 m (15 ft).

Table 2. Traffic data for 12 plain-jointed concrete pavements.

				Two-Directional ADT					
				Avg. Over Life	In 1 Year				
Project No.	General Location	Route No.	Date Opened		Value	Year	No. of Lanes	Percentage Trucks	No. of 80-kN (18-kip) ESALs[a] (millions)
JPC-26	Tacoma, Washington	I-5	1962	36 750	57 250	1974	6 in 1962; 8 in 1971	12	5.42
JCP-27	San Francisco	I-80	1955	35 750	55 000	1974	6	13	14.57
JCP-28	Los Angeles	I-5	1954-1958	67 500	-		4	-	-
			1959-1974	113 500	129 000	1974	6	12	39.65
JCP-29	Dallas	I-35	December 1960	12 000	19 300	1974	4	6	3.60
JCP-30	Salt Lake City	I-15	1965	76 500	102 000	1974	3	8	5.12
JCP-31	Phoenix	I-17	1960	41 500	75 000	1974	6	25	30.23
JCP-32	New Brunswick, NJ	US-130	1949	12 850	17 100	1974	4	44	35.93
JCP-33	Atlanta	I-75 and I-85	1950	100 000	170 000	1974	6	9	21.74
JCP-34	Denver	I-70	1964	37 250	50 000	1974	6	25	6.45
JCP-35	Denver	I-25	1964	56 300	75 000	1974	6	25	8.83
JCP-36	Detroit	Davison Expressway	1942	44 300	61 600	1976	6	8	18.73
JCP-37	Toronto	Highway 27	1970	65 000	80 000	1976	6	12	6.53

[a]Total one-directional in heaviest traveled lane.

Table 3. Pavement data for 12 plain-jointed concrete pavements.

Project No.	Thickness of PCC Slab (cm)	Subbase 1 Material	Subbase 1 Thickness (cm)	Subbase 2 Material	Subbase 2 Thickness (cm)	Subbase 3 Material	Subbase 3 Thickness (cm)	Subgrade Material	Transverse Joints Type	Transverse Joints Spacing (m)
JCP-26	22.9	Granular base	5.1	Granular base	17.8	-	-	Glacial till	Contraction	4.6
JCP-27	20.3	Cement-treated base	10.2	Granular base	30.1	-	-	Sand, clay, and silt	Contraction	4.6
JCP-28	20.3	Cement-treated base	10.2	Granular base	10.2	Sand	20.3	Sand and silt	Contraction	4.6
JCP-29	25.4	Granular base	15.2	Asphalt concrete[a]	10.2	-	-	Austin chalk	Contraction	4.6
JCP-30	22.9	Cement-treated base	10.2	Granular base	5.1	-	-	Imported borrow fill	Contraction (skewed)	3.7-5.8
JCP-31	22.9	Granular base	10.2	Select material	15.2	-	-	Clay	Contraction (skewed)	4.6
JCP-32	25.4	Granular base	30.1	Select material	15.2	-	-	Silt and clayey silt	Contraction	4.6
JCP-33	20.3	Granular base	15.2	-	-	-	-	Silty clay	Contraction Expansion	9.1 151
JCP-34	20.3	Granular base	15.2	Granular base	35.6	-	-	Sand and gravel	Contraction (skewed)	3.7-5.8
JCP-35	20.3	Granular base	15.2	-	-	-	-	Sand and gravel	Contraction (skewed)	4.6
JCP-36	25.4	Granular base	12.7	-	-	-	-	Clay	Contraction	7.6
JCP-37	22.9	Cement-treated base	15.2	-	-	-	-	Clay	Contraction (skewed, dowels)	3.7-5.8

Note: 1 m = 3.28 ft.
[a]Old pavement.

Figure 1. Equivalent 80-kN single-axle load applications: predicted by using Equation 2 versus actual data calculated from 16 years at AASHO test-road site.

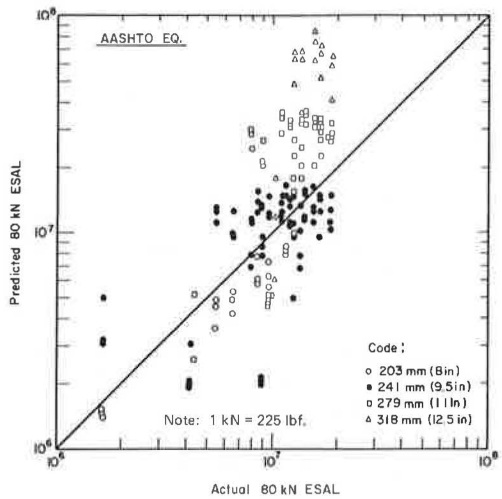
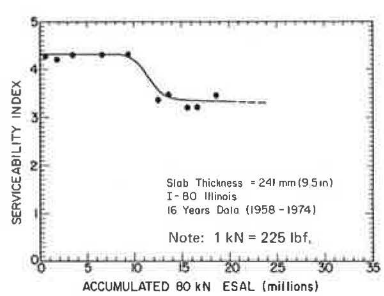


Figure 2. Typical performance curve for original AASHO test-road sections on I-80 (16-year data).



to determine the functions β and ρ. Analysis of the resulting equation and the performance data showed that this equation did not fit the general trend of the data for thicker slabs. A typical plot for a single 241-mm (9.5-in) PCC slab is shown in Figure 2. Equation 2 cannot fit this type of performance curve because of its mathematical form. A new approach based on a different

Table 4. Performance data for 25 sections of AASHO test road that are in service on I-80 (1958-74).

Section No.	Serviceability Index 1962	1968	1969	1971	1972	1974
672	4.10	3.44	3.21	2.61	2.76	1.69
658	4.10	3.14	2.83	2.61	2.11	1.69
652	4.10	3.01	3.02	2.98	2.57	1.69
552	4.30	3.36	3.52	3.14	3.14	2.87
676	4.00	3.21	3.21	2.88	3.28	2.75
512	4.30	3.70	3.70	3.36	3.36	3.32
542	4.20	3.36	3.12	3.21	3.01	2.88
352	3.10	2.98	3.11	2.80	2.95	2.96
702	4.20	3.36	3.52	3.07	3.28	2.94
528	4.30	3.79	3.61	3.28	3.36	3.27
368	4.30	3.52	3.70	3.36	3.52	3.16
390	4.30	4.00	3.79	3.14	3.01	3.36
690	4.20	3.21	3.52	3.21	2.85	2.95
376	4.30	3.36	3.44	3.21	3.21	3.44
530	4.30	3.36	3.70	3.36	3.28	3.11
364	4.30	3.79	3.78	3.21	3.28	3.44
378	4.30	3.61	3.21	3.21	3.21	3.23
498	4.50	3.52	3.70	3.14	3.28	3.08
388	4.30	3.44	3.52	3.28	3.61	3.44
398	4.30	4.11	3.70	3.70	3.44	3.44
510	4.40	3.36	3.44	3.07	3.21	3.23
366	4.30	3.89	3.21	3.29	3.34	3.39
396	4.30	3.44	3.44	3.21	3.36	3.52
350	4.20	3.70	3.89	3.45	3.63	3.81
380	4.20	3.36	3.35	3.06	3.28	3.39

mathematical function is needed to develop an equation that fits the form of the performance curve.

After several trial analyses, the following mathematical form was selected because it fits the actual measured performance data for the 25 AASHO sections.

$$W'_{18} = \{\rho \ln[(3/y) - 1] + \beta\} 10^6 \tag{3}$$

where

W'_{18} = total equivalent single-axle loads to reduce the serviceability index from P1 to P2,
$\beta = -50.08826 - 3.77485H + 30.64386(H)^{1/2}$
$\rho = -6.69703 + 0.13879H^2$, and
$y = P2 + \{3.0/[\exp(-\beta/\rho) + 1]\} - P1$.

The standard error of the estimate of log W'_{18} is 0.22. The adequacy of Equation 3 to predict the performance of the 25 sections over a 16-year period is shown in Figure 3, which can be directly compared with Figure 1. The new equation has a much smaller standard error (0.22 versus 0.31); the difference can be seen in the figures. Individual plots for the 203-, 241-, 274-, and 318-mm (8-, 9.5-, 11-, and 12.5-in) thick slabs are shown in Figure 4. Equation 3 fits the long-term performance data much better than did Equation 1.

This expression permits only an evaluation of the effect of PCC slab thickness and terminal and initial serviceability index on equivalent 80-kN load applications. Therefore, it was extended so that other variables could be included, such as the k-value and the PCC modulus of rupture, by using the Westergaard edge-stress equation in a way similar to that in which the Spangler corner equation was incorporated into the original AASHO equation (4). The following equation was obtained:

$$\log W_{18} = \log W'_{18} + (3.893 - 0.706\,P2)\log \times (F28/690) \times 4\log[(8.789^{0.75}/M) + 0.359] \div \{4\log[(Z^{0.25} \times 0.540H^{0.75})/M] + 0.589\} \qquad (4)$$

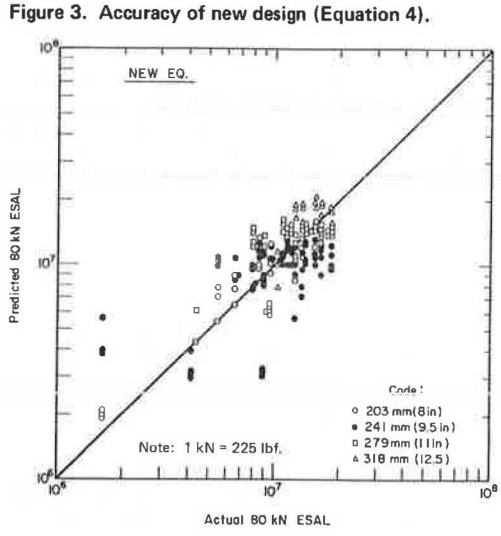
where

$M = (1.6a^2 + H^2)^{1/2} - 0.675H$,
a = radius of applied edge load (in),
$F28$ = modulus of rupture used in design (28-d, 3rd point load adjusted for variability) = $FF - C(Fcv/100)\,FF$,
FF = mean modulus of rupture at 28 d, 3rd point load (lbf/in^2),
Fcv = coefficient of variation of modulus of rupture (percent), and
C = 1.03, constant that represents a confidence level of 85 percent.

The Westergaard edge-load equation was used because the fatigue analysis and field observations of the distress, as discussed by Darter (2), show that the edge of the slab is the critical stress point.

Based on these results, Equation 4 is the best performance equation derived and is believed to be adequate for design purposes. Its standard error is 0.22 based on 150 data points from the 25 AASHO sections (which is the same as the standard error of Equation 1).

Figure 3. Accuracy of new design (Equation 4).



CLIMATIC EFFECT

Performance data were collected from 12 plain-jointed concrete projects located in eight states. These projects ranged in age from 6 to 34 years and the total number of 80-kN (18-kip) ESALs in their heaviest traveled lanes ranged up to 39 million.

Analyses were conducted to determine whether the pavements in regions different from Illinois showed different performances. Four general climatic regions—wet and freezing (WF); wet, but not freezing (W); dry and freezing (DF); and dry, but not freezing (D)—were chosen based on precipitation (PPN), potential evapotranspiration (EVT), frost heave, and freeze-thaw damage as defined below (1 m = 39.4 in).

Climatic Region	PPN and EVT (m)	Frost Heave and Freeze-Thaw Damage
WF	PPN ≥ EVT or PPN ≥ 0.76	Occurs in pavements in region
W	PPN ≥ EVT or PPN ≥ 0.76	Does not occur in pavements in region
DF	PPN < EVT	Occurs in pavements in region
D	PPN < EVT	Does not occur in pavements in region

[Generally, frost heave and freeze-thaw damage occur in areas that have a mean freezing index >0 (7)]. A climatic factor (CF) was defined as follows:

$$CF = W_{18(\text{computed})}/W_{18(\text{actual})} \qquad (5)$$

where

W_{18} (computed) = total number of 80-kN ESALs to reduce serviceability index from initial value to terminal value computed by using Equation 4.
W_{18} (actual) = total accumulated number of 80-kN ESAL applications to pass over pavement, determined from traffic data.

CF can theoretically range from less than 0.1 to more than 1.0; when it equals 1.0, there is no significant climatic effect for a given pavement. A computation of CF for each of the pavements is given in Table 5. There is considerable scatter in each climatic region, but the results indicate that pavements perform somewhat differently in the different climatic regions.

Region	CF Range	Mean CF
WF	0.64-1.45	1.02
DF	0.79-1.26	1.02
W	0.56-1.12	0.84
D	0.29-0.71	0.47

These data indicate, for example, that similar plain-jointed concrete pavements constructed in D-climates would last about twice as long as those in W-climates. However, additional data are needed to further verify these results.

The following values are recommended for use in design.

Region	Design Value
WF	1.0
W	0.9
DF	1.0
D	0.6

The relation between the number of 80-kN ESALs predicted (by using Equations 4 and 5) and the number

Table 5. Data used to compute climatic factors.

Project No.	Climatic Region	H (cm)	FF (MPa)	K[a] (MN/m^3)	Serviceability Index	W_{18} (millions) Calculated[b]	Actual[c]	CF
1-3[d]	WF	20.3	4.77	31.3	1.7	15.44	11.16	1.38
4-6	WF	24.1	4.77	31.3	2.8	15.07	11.32	1.33
7-10	WF	24.1	4.77	31.3	3.1	12.25	13.75	0.89
11-14	WF	24.1	4.77	31.3	3.2	11.40	17.82	0.64
15-17	WF	27.9	4.77	31.3	3.1	20.42	14.09	1.45
18-22	WF	27.9	4.77	31.3	3.4	15.70	18.94	0.83
23-25	WF	31.7	4.77	31.3	3.6	18.62	19.47	0.96
32	WF	25.4	5.18	53.6	3.0	22.83	35.93	0.64
36	WF	25.4	4.15	31.3	2.5	20.87	18.73	1.11
37	WF	22.9	4.48	109.0	3.9	6.11	6.53	0.94
26	W	22.9	4.83	52.2	3.9	6.08	5.42	1.12
33	W	20.3	4.88	54.3	3.4	12.16	21.74	0.56
30	DF	22.9	4.73	72.7	4.0	5.12	5.12	1.00
34	DF	20.3	4.88	96.0	3.4	8.10	6.45	1.25
35	DF	20.3	4.88	99.8	3.6	6.97	8.83	0.79
27	D	20.3	4.73	122.7	3.0	10.29	14.57	0.71
28	D	20.3	4.73	136.2	3.0	11.58	39.65	0.29
31	D	22.9	5.08	54.3	3.2	12.57	30.23	0.42

Note: 1 cm = 0.4 in, 1 MPa = 145 lbf/in^2, and 1 MN/m^3 = 3.68 lbf/in^3.
[a] Mean of lowest 9 months on top of subbase.
[b] Computed from Equation 4.
[c] Computed from traffic data.
[d] Pavements from road-test site that had similar slab thickness and traffic were combined.

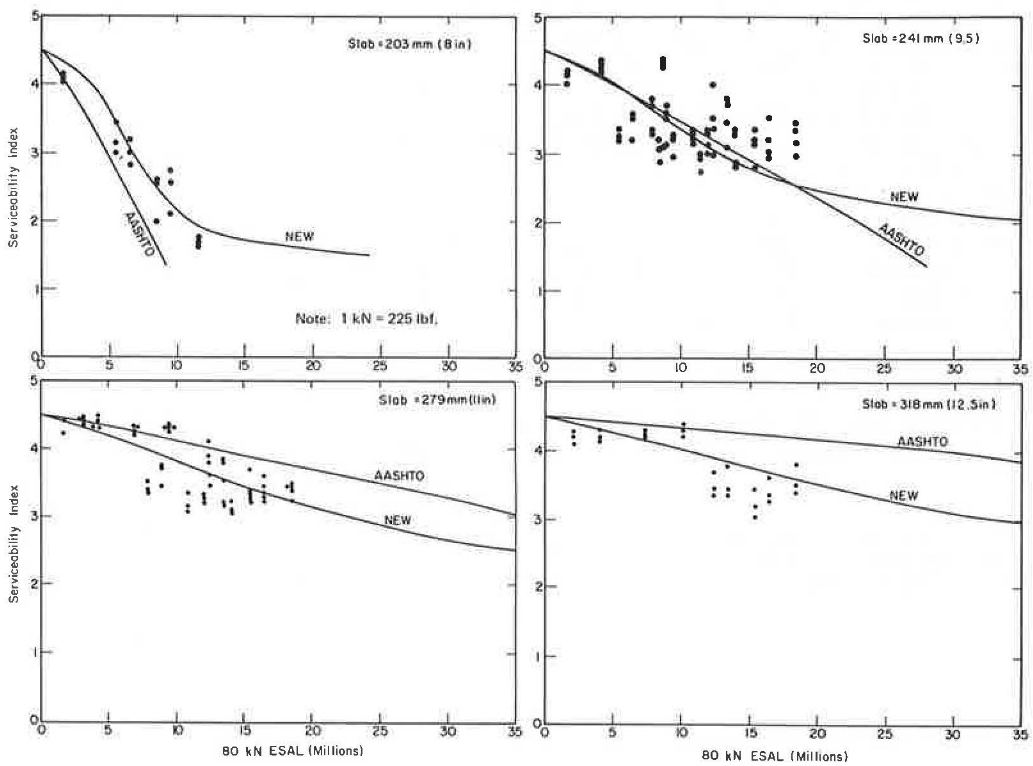


Figure 4. Comparisons of original AASHO performance (Equation 2) with new design (Equation 4) for slabs of varying thickness.

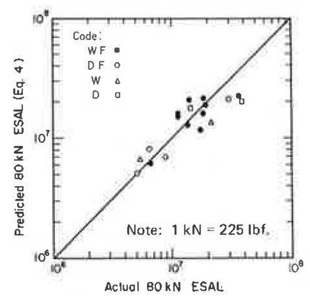


Figure 5. Comparison of actual (computed from traffic data) with predicted (computed by using Equations 4 and 5) ESALs over life of each project.

of 80-kN ESALs computed (or actual) for all pavements is shown in Figure 5. The total 80-kN ESAL data were computed based on the heaviest traveled lane from the time the pavement was opened to traffic until the date of the survey at which the serviceability index was determined. The computed ESALs were adjusted by the recommended climatic factors. The standard error of prediction of Equations 4 and 5 is 0.175.

TERMINAL SERVICEABILITY INDEX FOR LOW-MAINTENANCE DESIGN

A terminal serviceability index must be selected for design. The value selected affects (a) the initial construction cost of the pavement, (b) the probability that structural distress will occur that requires maintenance, and (c) the costs to users caused by rough pavements. The higher the terminal serviceability index, the more substantial the structure must be and, therefore, the higher the construction cost of the pavement. However, the higher the terminal serviceability index, the lower the maintenance cost and the costs to users from rough pavements and maintenance operations.

Figure 6 (8) shows that the total user costs at three different average daily traffic (ADT) rates over a 20-year life span increase dramatically as the terminal serviceability at the 20th year is decreased from 3.5 to 2.0. The rate of increase is very high for a terminal serviceability of less than 3.0.

The loss of serviceability of a jointed concrete pavement is caused by several types of distress. A previous analysis (8) indicated that linear cracking, faulting at joints and cracks, spalling of joints and cracks, and differential settlement of the slabs (causing roughness) are the major causes of loss of serviceability. As the serviceability index decreases from its initial value after construction, the probability that the pavement will require maintenance increases. A plot of serviceability index versus the percentage of projects showing relatively low maintenance performance is shown in Figure 7 for all 37 projects (pavements having less than 0.3 percent of their areas patched were considered to be low maintenance). All the pavements that had serviceability indexes of 3.6 or more showed low maintenance performance, but those that had serviceability indexes of less than 2.7 had all received maintenance, some of which was extensive. The 50th percentile for receiving maintenance is a serviceability index of 3.0.

Based on these results and the desirability of keeping construction costs as low as possible, a design terminal serviceability index in the range of 3.0 to 3.5 is the minimum acceptable. A minimum value of 3.0 is selected because it provides a reasonable ensurance that little or no maintenance will be required before the pavement reaches this value and because costs to users do not increase significantly at this value.

DESIGN APPLICATION

A sensitivity analysis of Equations 4 and 5 is given to illustrate the effects of five design parameters—the slab thickness, the modulus of rupture of PCC, the modulus of foundation reaction, the climatic factor, and the total number of 80-kN ESALs applied in the design lane. The relative effects of these variables are shown in Figure 8 in which all of the curves are developed for a terminal serviceability index of 3.0. These curves show that each of the parameters has a significant effect on pavement life as measured by the number of 80-kN ESALs. Examples of the effects of each parameter are shown below in terms of the change in number of 80-kN ESALs caused by changes in each of the other parameters (1 MPa = 145 lbf/in², 1 MN/m³ = 3.68 lbf/in³, and 1 cm = 0.39 in).

Figure 6. Total extra costs to users versus terminal serviceability for different traffic conditions.

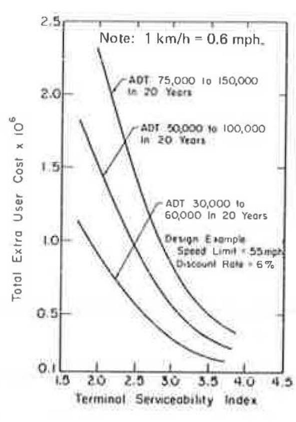


Figure 7. Effect of serviceability on maintenance requirements.

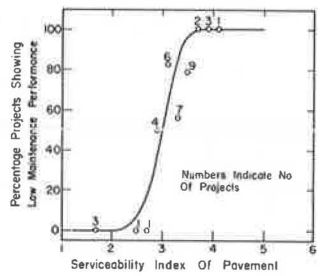
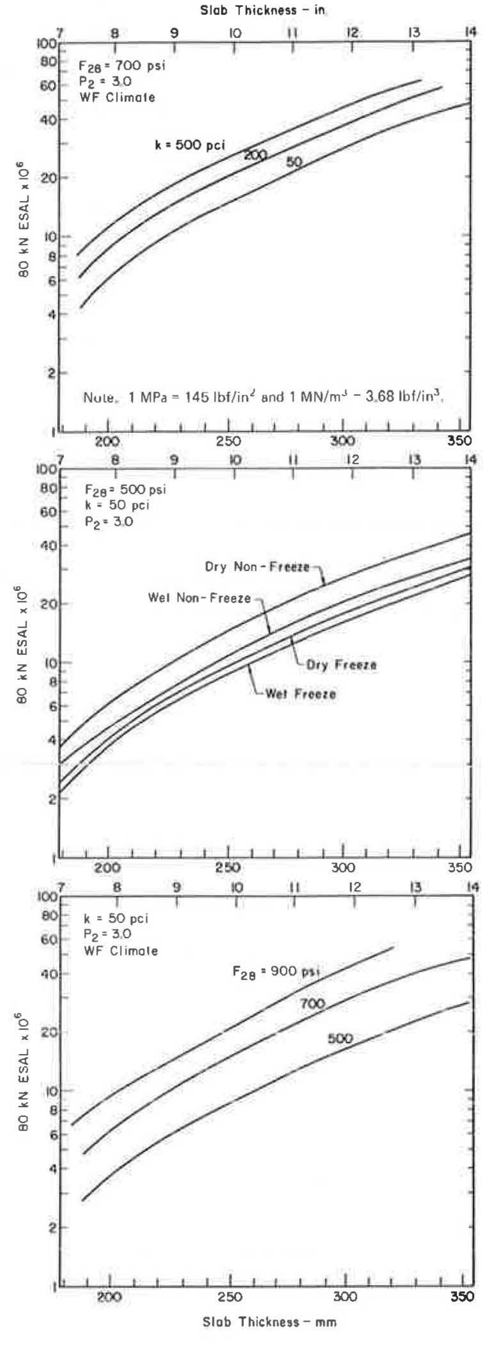


Figure 8. Sensitivity of Equation 4 over a range of conditions.



Design Parameter	Change In Parameter	Change In Millions of 80-kN ESALs
Modulus of rupture, MPa	4.1 to 5.5	9.2 to 16.5
k-value of foundation, MN/m^3	27.1 to 136	9.2 to 14.5
Slab thickness, cm	23 to 28	9.2 to 20.0
Climatic region	WF to D	9.2 to 15.3

A variation over the typical range of each parameter produces a significant change in the number of 80-kN ESALs. (A typical lane loading for a heavily trafficked pavement is 1 million 80-kN ESALs/year, which means that these are equivalent to years of pavement life.)

A graphical solution to Equation 4 is shown in Figure 9 for a terminal serviceability index of 3.0. The use of this chart in design is illustrated below. Assume the following parameters and solve for the required slab thickness:

Parameter	Value
Climatic region	WF
Millions of 80-kN ESALs	20.0
k-value on top of subbase, MN/m^3	27.1
F28, MPa	4.1

Therefore, the required PCC slab thickness is 279 mm (11 in). The total number of 80-kN ESALs is not reduced because the project is located in a WF climate, and the CF = 1.0.

If the pavement were located in a D-climate, the required slab thickness would be calculated as follows: Millions of 80-kN ESALs = 20.0 x 0.6 = 12.0, and slab thickness = 241 mm (9.5 in). Here, the traffic computed in the design lane is reduced by the recommended CF = 0.6 for a D-climate. [Additional recommendations on obtaining design input values are given by Darter (1)].

JOINTS, SHOULDERS, AND SUBBASE

Joint design is also a very important consideration. Most of the projects used in the development of the design procedure had 4.6-m (15-ft) contraction-type joint spacing with dowel bars. Field studies have shown that dowel bars must be used in all climates, except possibly D. If dowels are not used, serious faulting may develop under heavy traffic as discussed in detail by Darter (2) and Packard (9). Joint spacing must also be limited to about 4.6 m because field studies have shown that longer slabs show much greater transverse cracking. Adequate high-durability sealants must be used to prevent water and other materials from infiltrating. Shoulders must be designed for low maintenance. This requires either PCC-tied shoulders or full-depth asphalt concrete shoulders with sealed longitudinal joints (2). The subbase should be of high-quality, stabilized nonerodable material.

CONCLUSIONS

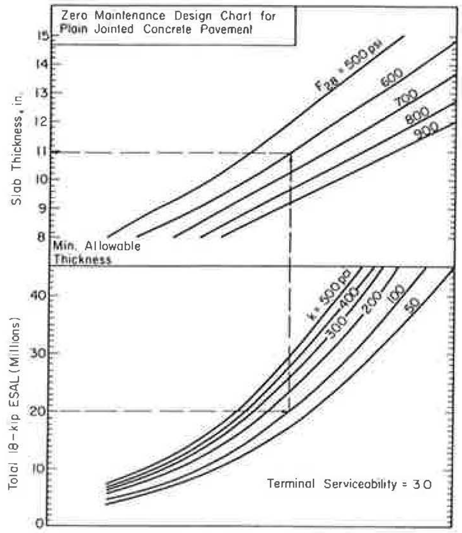
1. A new structural design procedure has been developed for heavily trafficked jointed concrete pavements. The procedure is based on the serviceability-performance concept and a fatigue-damage analysis. Only the development of the serviceability-performance analysis is described here, which was derived from long-term performance data from 25 pavement sections in service on I-80 in Illinois. A major objective of the procedure is to provide zero-maintenance (very-low structural maintenance) performance.

2. The new design equation represents an improvement over the existing AASHO equation and fits the long-term 16-year data much better. Performance data were obtained from 12 projects located in various climates in the United States and compared with the performance of the Illinois pavements. Significant differences were found, and a climatic factor was introduced to adjust the design for varying climates.

3. A terminal serviceability index of 3.0 is recommended for design of heavily trafficked pavements to keep structural maintenance requirements at a minimum throughout the design period.

4. Although this paper emphasizes the determination of adequate slab thickness, several other design factors—including shoulders, subbase, joint spacing and load transfer, subdrainage, and joint sealant—are of equal importance.

Figure 9. Graphical solution of Equation 4 for use in design of low-maintenance, heavily trafficked, plain-jointed concrete pavements.



ACKNOWLEDGMENTS

This investigation was conducted at the University of Illinois, Urbana-Champaign. I wish to thank the sponsor, the U.S. Department of Transportation, Federal Highway Administration, and the project monitors, Thomas Pasko, William Kenis, and Floyd Stanek. Special thinks are due Huey-Shin Yaun for assistance with the statistical analysis. I also thank the Illinois Department of Transportation for providing data from the 25 sections at the AASHO Road Test site and Donald Schwartz, Phillip Dierstein, and Robert J. Little for their assistance.

The contents of this paper reflect my views; I am responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. M. I. Darter and E. J. Barenberg. Design of Plain-Jointed Concrete Pavements: Vol. 2—Design Manual. Federal Highway Administration, Technical Rept. FHWA-RD-77-111, 1977.
2. M. I. Darter. Design of Zero-Maintenance Plain-Jointed Concrete Pavement: Vol. 1—Development of Design Procedures. Federal Highway Administra-

tion, Technical Rept. FHWA-RD-77-112, 1977.
3. Interim Guide for the Structural Design of Rigid Pavement Structures. Committee of Design, AASHO, April 1962.
4. Interim Guide for Design of Pavement Structures. AASHO, 1972.
5. AASHO Road Test: Report 5—Pavement Research. TRB, Special Rept. 61E, 1962.
6. B. F. McCullough and others. Evaluation of AASHO Interim Guides for Design of Pavement Structures. NCHRP, Rept. 128, 1972.
7. Engineering and Design: Pavement Design for Frost Conditions. U.S. Army Corps of Engineers, Rept. EM-1110-345-206, 1958.
8. M. I. Darter and E. J. Barenberg. Zero-Maintenance Pavement: Results of Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems. Federal Highway Administration, Technical Rept. FHWA-RD-76-105, April 1976.
9. R. G. Packard. Design Considerations for Control of Joint Faulting of Undoweled Pavements. Proc., International Conference on Concrete Pavement Design, Purdue Univ., West Lafayette, IN, Feb. 15-17, 1977, pp. 121-135.

Publication of this paper sponsored by Committee on Rigid Pavement Design.

Abridgment

Pavement Design Characteristics of In-Service Portland Cement Concrete

Thomas W. Kennedy, Department of Civil Engineering, University of Texas, Austin

The primary objectives of this investigation were (a) to determine the modulus of elasticity and the fatigue life of in-service concrete under repeated loads, (b) to estimate the variations in these properties, and (c) to determine the changes in the modulus of elasticity caused by repeated loads. Cores from four recently constructed portland cement concrete pavements in Texas were tested by using both the static and the repeated-load indirect tensile tests (Table 1), and the tensile strengths, fatigue lives, and moduli of elasticity and the variations of these properties were estimated.

EXPERIMENTAL PROGRAM

The indirect tensile test involves loading a cylindrical specimen with either static or repeated compressive loads that act parallel to and along the vertical diametral plane. To distribute it and to maintain a constant loading area, the compressive load is distributed through 13-mm (0.5-in) wide steel loading strips that are curved at the interface to fit the specimen.

This loading configuration develops relatively uniform tensile stresses that are perpendicular to and along the vertical axis and are fairly uniform over approximately the center 70 percent of the specimen. Failure generally occurs by splitting along the vertical axis. Estimates of the modulus of elasticity were obtained by using the applied load, the corresponding horizontal deformations (H_R), and an assumed Poisson's ratio.

The test specimens were cut from the cores and were 51 mm (2 in) high and approximately 102 mm (4 in) in diameter. A capping compound of high-strength gypsum plaster was applied to smooth the irregularities produced by coring. The apparatus used in the capping produced a radius of curvature that was the same as the radius of the specimens. All specimens were from the lower portion of the core.

In the static tests, a preload of 89 N (20 lbf), which corresponds to a tensile stress of about 11 kPa (1.5 lbf/in^2), was applied to the specimen to prevent impact loading and to minimize the effect of seating the loading strip. The specimen was then loaded at a rate of 13 mm/min (0.5 in/min). Loads and vertical deformation were continously recorded on an X-Y plotter.

In the repeated-load test, a preload of 89 N was used to prevent impact loading and to reduce movement of the specimen. Then, repeated total loads that produced total tensile stresses ranging from 2.17 to 3.56 MPa (315 to 516 lbf/in^2) were applied in the form of a haversine at a frequency of 1 Hz with a 0.4-s load duration and a 0.6-s rest period.

All tests were conducted at 24°C (75°F) and continued until failure, which was considered to occur when the specimen fractured completely. The recoverable (or resilient) horizontal deformations were measured at 25, 50, and 100 cycles and then periodically monitored during the remainder of the test.

The properties analyzed were the indirect tensile strength, the resilient modulus of elasticity, and the fatigue life, i.e., the number of load applications required to completely fracture the specimen. Values of the indirect tensile strength and the resilient modulus of elasticity were calculated by using the equations

Table 1. Summary of project data.

District Project Identification	Type of Aggregate	Cement Factor (kg/m^3)	Water-to-Cement Ratio[a]	Beam Strength[b] (MPa)
2E	Limestone	240	0.58	448
17B	Gravel	252	0.59	396
17M	Gravel	252	0.59	396
19B	Iron ore, slag, and gravel	279-330	0.43-0.53	396

Note: 1 kg/m^3 = 0.018 sack/yd^3 and 1 MPa = 145 lbf/in^2.
[a]Either by mass or by mass to volume.
[b]7-d field curing, center-point loading, using cast beams.